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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of:	)	Docket No. 72-22-ISFSI
PRIVATE FUEL STORAGE, LLC	)	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel Storage Installation)	)	September 5, 2002

STATE OF UTAH'S PROPOSED FINDINGS OF FACT AND CONCLUSIONS OF LAW ON UNIFIED CONTENTION UTAH L/QQ

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I. Introduction

The license at issue in this proceeding would allow Private Fuel Storage, LLC ("PFS"), to store in the open 4,000 unanchored casks containing spent nuclear fuel on three foot thick concrete slabs which are embedded a couple of feet into the soil. Instead of foundations or other mechanisms for supporting the concrete slabs, PFS's seismic design calls for adding Portland cement to native soil in an effort to improve the strength and stiffness of the soils that are expected to take the brunt of forces from a local earthquake. A capable fault only six tenths of a mile away could produce a magnitude 6.5 or larger shock dips directly under the site. The Stansbury fault – capable of a magnitude 7.0 or larger earthquake and also dipping toward the site – is located little more than 5 miles away. Consolidated Safety Evaluation Report, March 2002 ("Con-SER") at 2-46 to 2-27 (Staff Exh. C); Staff Exh. Q[1] at § 2.1.5. On top of this, PFS has requested NRC to lower the earthquake standard against

[1]Stamatakos, et al, *Seismic Ground Motion and Faulting Hazard at Private Fuel Storage Facility in the Skull Valley Indian Reservation, Tooele County, Utah -- Final Report* (September 1999.)

which PFS must demonstrate the seismic performance and adequacy of its design to such a low ebb that the requested standard appears to be even more lax than that required for the design of Interstate bridges in Utah and the design of some buildings under the International Building Code.

The decision facing the Board in Unified Contention Utah L/QQ is whether the meager design PFS has proposed and the relaxed seismic standard the Staff has endorsed provide an adequate margin of safety, protect public health and safety, and are in the public interest.

Unified Contention Utah L/QQ[2] presents five major issues: (1) a unique, unprecedented and untested seismic design for the storage facility; (2) lack of characterization and poor strength of soils at the PFS site; (3) PFS's novel and untested use of soil cement to overcome foundation sliding during an earthquake; (4) total reliance on a nonlinear computer analyses to demonstrate the seismic stability of 4,000 unanchored storage casks; and (5) an exemption from the standard that establishes the ground motions for the design of the storage facility.

Following the Board's cue, the State structures its Findings and Conclusions by discussing first the logic and concepts of the decision and the applicable legal standards, then in sequentially numbered paragraphs enumerating the findings of fact and conclusions of law.

II. Decisional Framework and Applicable Legal Standards

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[2] *See* PFS Exh. 237.

A. Decisional Framework

The touchstone in reaching a decision in this proceeding is to ensure protection of public health and safety. Advanced Medical Systems, Inc. (One Factory Row, Geneva, Ohio 44041), CLI-94-6, 39 NRC 285, 314 (1994) (*internal quotation omitted*) ("[t]he fundamental principle guiding all Commission licensing actions is the paramount consideration of public safety."). The contest here is focused on the design basis earthquake for the PFS facility and whether there is a sufficient built-in margin of safety in the facility design. As the Commission noted, the State's geotechnical contention squarely relates to critical safety issues that are material to licensing. CLI-01-12, 53 NRC 459, 465-467 (2001).

The gravamen of the State's claim is that the PFS facility is of such an ill conceived and meager design that it compromises safety. PFS argues that its design is safe enough. One critically important factor in reaching a decision in this case is whether the Atomic Energy Act ("AEA") permits the Board to give any consideration or weight to economic costs in determining whether the design of the PFS facility provides an adequate margin of safety to protect public health and safety. As the following discussion demonstrates, when dealing with basic safety regulations, the Board cannot give any consideration to costs.

PFS has applied to the NRC for a specific license to "receive, transfer, and store spent nuclear fuel (SNF) from commercial nuclear power plants at a privately owned independent spent fuel storage installation (ISFSI) which it proposes to construct and operate...." FEIS at xxix (Staff Exh. E). Under Part 72 "no person may acquire, receive, or possess – (1) Spent fuel for the purpose of storage in an ISFSI" without obtaining a specific license from the Commission. 10 CFR § 72.6(c).

Nowhere in the Atomic Energy Act is there a clear statement of the Commission's authority to issue a license for storage of spent nuclear fuel in an ISFSI. The principles to guide the Board in its decision-making must, therefore, be gleaned from the interstices of the Act.

The Commission's source of authority to regulate storage of spent nuclear fuel in an ISFSI is pertinent to the standards this Board must use to make its decision. In its brief to the Commission in response to CLI-02-11,[3] the Staff claims the Commission derives its authority under the Atomic Energy Act to issue a license to PFS from the Domestic Distribution of Special Nuclear Material (AEA § 53, 42 USC § 2073), Source Material (AEA § 63, 42 USC § 2093) and By Product Material (AEA § 81, 42 USC § 2111) because, argues the Staff, these materials form the constituent materials of spent nuclear fuel. Staff Brief at 3. These sections of the AEA generally relate to the distribution and use of special nuclear, source and by-product materials and not to their storage.[4] Even if the Commission derives its authority from these sections of the AEA, it cannot issue a license if there is undue risk to public health and safety.[5]

The general provision of the AEA – Sections 161 and 182 – also provide guidance. Turning first to the more specific section on License Applications – Section 182 – it

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[3] NRC Staff's Brief in Response to CLI-02-11 (May 15, 2002).

[4]The Commission is authorized (1) under § 53 to *inter alia* issue licenses to transfer, possess, own, receive possession of or title to certain specified quantities of SNM; (2) under § 63 to issue licenses for and to distribute source material; and (3) under § 81 to issue licenses for and distribute by-product material.

[5]*See* AEA §§ 53(e)(7); 69; 81, 42 U.S.C. §§ 2073, 2099, and 2111.

authorizes the Commission, by rule or regulation, to require each license applicant to submit technical, financial and other information. In addition, Section 182 authorizes the Commission to enact regulations to obtain information "to enable it to find that the utilization or production of special nuclear material will be in accord with the common defense and security and will provide adequate protection to the health and safety of the public..." 42 USC § 2232(a).

Section 161, entitled General Provisions, authorizes the Commission to:

> (b) establish by rule, regulation, or order, such standards and instructions to govern the possession and use of special nuclear material, source material, and byproduct material as the Commission may deem necessary or desirable to promote the common defense and security or to protect public health or to minimize danger to life or property. . . .
>
> . . .
>
> (i) . . . prescribe such regulations or order as it may deem necessary . . . (3) to govern any activity authorized pursuant to this chapter, including standards and restrictions governing the design, location, and operation of facilities used in the conduct of such activity, in order to protect health and to minimize danger to life or property[.]

42 USC § 2201(a)and (i)(3).

Union of Concerned Scientists v. U.S. Nuclear Regulatory Commission, 824 F.2d 108 (D.C. Cir. 1987), reviewed the question of whether and to what extent a regulatory agency may consider economic costs in carrying out its health-based statutory provisions -- specifically it considered whether NRC could consider economic costs when making backfit decisions (decisions about safety-enhancing modifications to previously licensed nuclear power plants). The Court analyzed both AEA sections 182(a) and 161(b) and (i). The Court found that section 182(a) commands the Commission to ensure that any use or production of nuclear materials "provide[s] adequate protection to the health or safety of the public" and thus the Commission cannot consider economic costs of safety measures. 824 F.2d at 109, 114. In contrast, the Court found that section 161 empowers – but does not require – the Commission to establish safety requirements. Because section 182(a) requires the Commission to impose adequate protection standards, section 161 cannot be read to impose those same standards. Therefore, reasoned the Court, under section 161 the Commission has the discretion to impose additional safety precautions on nuclear power plants already satisfying the adequate protection standard. In making this decision, the Commission may take economic costs into account. 824 F.2d at 118.

Union of Concerned Scientists illustrates the dilemma facing the Board when there is no clear statutory authority to license an ISFSI. The rationale of the court when applied to an ISFSI does not stand if the statutory authority to license an ISFSI is derived from section 161 because that singular health and safety based requirement cannot be read to be discretionary.[6]

When the drafters of the Atomic Energy Act wanted to have the Commission consider economic costs, they knew how to use explicit language.[7] However, assuming *arguendo* that the Commission may take economics into account in setting a health and safety

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[6]Only by finding that section 161 was a discretionary provision, did Union of Concerned Scientists find that the Commission may consider economic costs when imposing additional safety requirements on nuclear power plants. 824 F.2d at 118.

[7]*See* AEA § 84, 42 USC 2114 (the Commission may consider economic costs, as well as risks to public health safety and the environment in the management of by-product material, as defined under 11e.(2) of the Act.).

standard for an ISFSI, that does not mean that a decision-maker can interpose cost considerations as a factor in deciding whether an applicant or licensee has met a health and safety standard. In declining to accept a cost-based argument to delay delivery of fuel shipments, the Board in Cincinnati Gas & Electric Co. (William H. Zimmer Nuclear Station), LBP-79-24, 10 NRC 226 (1979), held: "All that is relevant under the [Atomic Energy] Act is whether, in undertaking their planned shipments and storage of fuel, the Applicants will abide by applicable regulatory requirements and the terms of their materials license." 10 NRC at 231. Furthermore, "[u]nder the Atomic Energy Act, economic costs become relevant only in terms of an applicant's financial qualification." Id.

In summary, the Board cannot consider financial costs in determining whether PFS has met the health and safety standards in Part 72. The standards must simply be met. In arriving at its decision on these fundamental safety provisions, the Board should give no consideration to whether PFS may have to go back to the drawing board to find an acceptably safe design for the Skull Valley site.

B. Legal Standard

Relevant to Utah L/QQ, the Commission may issue an ISFSI license only upon a determination that "[t]he proposed site complies with the criteria in subpart E" (Siting and Evaluation Factors), and "[t]here is reasonable assurance that ... [t]he activities authorized by the license can be conducted without endangering the health and safety of the public ... and [that] issuance of the license will not be inimical to the common defense and security." 10 CFR § § 72.40(a)(2), (13) and (14).

In a formal adjudicatory proceeding, 10 C.F.R. § 2.732 provides that the applicant has the burden of proof, and "in order for the applicant to prevail on each contested factual issue, the applicant's position must be supported by a preponderance of the evidence." Louisiana Energy Services, L.P. (Claiborne Enrichment Center), LBP-96-7, 43 NRC 142, 144 (1996), *citing* Philadelphia Electric Co. (Limerick Generating Station, Units 1 and 2), ALAB-819, 22 NRC 681, 720 (1985); Pacific Gas and Electric Co. (Diablo Canyon Nuclear Power Plant, Units 1 and 2), ALAB-763, 19 NRC 571, 577 (1984). *See also* Duke Power Co. (Catawba Nuclear Station, Units 1 and 2), CLI-83-19, 17 NRC 1041, 1048 (1983), *citing* Consumers Power Co. (Midland Plant, Units 1 and 2), ALAB-283, 2 NRC 11, 17 (1975) (applicant carries burden of proof on safety issues). Furthermore, while 10 C.F.R. § 2.714 imposes the burden of going forward on the intervenor, it does not shift the ultimate burden of proof from the applicant to the intervenor. Yankee Atomic Electric Co. (Yankee Nuclear Power Station), LBP-96-15, 44 NRC 8, 31 (1996).

The licensing board "must evaluate the staff's evidence and arguments in the light of the same principles which apply to the presentation of the other parties." Consolidated Edison Company of New York (Indian Point, Units No. 1, 2, and 3), ALAB-304, 3 NRC 1, 6 & n. 14 (1976) (*citing* Vermont Yankee Nuclear Power Corporation (Vermont Yankee Station), ALAB-138, 6 AEC 520, 532 (additional views of Mr. Farrar) (1973); ALAB-229, 8 AEC 425, 440-441, reversed on other grounds, CLI-74-40, 8 AEC 809 (1974). "[S]taff views 'are in no way binding upon' the boards; they cannot be accepted without passing the same scrutiny as those of the other parties." Indian Point, ALAB-304, 3 NRC at 6 (*citing* Southern California Edison Co. (San Onofre Units 2 and 3), ALAB-268, 1 N.R.C. 383, 400 (1975)); *see also*, Texas Utilities Generating Co. (Comanche Peak Steam Electric Station, Units 1 and 2), LBP-82-87, 16 NRC 1195, 1200 (1982), *vacated on other grounds*, CLI-83-30, 18 NRC 1164 (1983).

The burden is on PFS to show by a preponderance of the evidence that it meets all the following regulations prior to license issuance:

1. Site characteristics have been investigated and assessed that may directly affect the safety or environmental impact of the proposed ISFSI.. 10 CFR § 72.90

2. Site specific soil stability investigations and laboratory analyses have been performed to demonstrate adequacy of foundation loading. 10 CFR § 72.102 (c) ("Sites other than bedrock sites must be evaluated for ... other soil instability due to vibratory ground motion") and (d) ("Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading").

3. Structures, systems, and components ("SSCs") must be designed to accommodate the effects of and to be compatible with site characteristics. 10 CFR § 72.122(b)(1). SSCs must also be designed to withstand the effects of earthquakes 10 CFR § 72.122(b)(2) (SSCs must be designed to withstand the effects of earthquakes without impairing their capability to perform safety functions. The design bases for these SSCs are the most severe reported natural phenomena. The ISFSI should also be designed to prevent massive collapse of building structures or the dropping of heavy objects on the spent fuel or SSCs as a result of building structural failure).

4. An exemption from 10 CFR § 72.102(f)(1) is authorized by law, if it will not endanger life or property or the common defense and security and is otherwise in the public interest. 10 CFR § 72.7.

To obtain a license under Part 72, PFS needs to do more than show that there will be no unacceptable release of radiation. PFS has applied for an exemption from only one provision of the Part 72 siting criteria. Consequently, even if an exemption is granted, PFS must prove that it meets the other provisions of the siting and evaluation criteria and that its structure, systems and components are designed to withstand the effects of earthquakes.

Under Part 72, SSCs must provide "reasonable assurance that spent fuel ... can be ... stored, and retrieved without undue risk to the health and safety of the public." 10 CFR § 72.3. In Union of Concerned Scientists the Court found that the Commission uses the standard "without undue risk" interchangeably with the "adequate protection" standard in AEA section 182(a). 789 F.2d at 109. When considering whether spent nuclear fuel can be safely stored at PFS, the Board should not be swayed that any benefit to health and safety is outweighed by the economic cost of reducing the residual risk to an acceptable level.

1. Exemption

In addition to presenting an unconventional, untested and spare design, PFS has requested and the Staff has endorsed an exemption from the regulation that establishes the design basis earthquake.[8] If PFS were to design its facility to meet current regulatory requirements, its facility would have to withstand peak ground accelerations of approximately 1.15 g, whereas under the 2,000-year return period exemption standard, the

[8]The reason articulated for the exemption is that the ground motion values of 1.15g (horizontal) and 1.17g (vertical) peak ground acceleration estimated using a deterministic seismic hazard analysis ("DSHA") methodology "exceed the SAR proposed design values" and that in order to "resolve the issue of seismic design, the applicant submitted to NRC, a request for an exemption to the seismic design requirement of 10 CFR 72.102(f)(1)...." Con-SER at 2-34.

design ground-motion accelerations would be approximately 0.7g. Con-SER at 2-34, 2-48. PFS must show that such an exemption "will not endanger life or property or the common defense and security and [is] otherwise in the public interest." 10 CFR § 72.7. What standard must PFS meet to show that a design basis earthquake based on a 2,000-year return period earthquake will not endanger life or property? It is not axiomatic that the standard should be the accident standard in 10 CFR § 72.106(b). Under the regulations as enacted, PFS must prove that in the event of a worst case earthquake the release of radiation will not exceed 5 rems. 10 CFR §§ 72.102(f)(1), 72.106(b). PFS is not requesting an exemption from 10 CFR § 72.106(b) but the effect of the exemption from the design basis regulation is to dilute the margin of safety the Commission has built into 10 CFR § 72.106(b).

The exemption standard also requires consideration of the public interest. While the term "public interest" is frequently used, it is rarely defined. The Atomic Energy Commission, in a footnote to a decision, stated, "[t]he determination to be made if '... otherwise in the public interest' is not limited merely to safety considerations, since the word 'otherwise' is defined as 'in other respects.' It is concluded that 'public interest' is not needless repetition to the safety factors in the term 'endanger life or property, but constitutes a distinct and separate aspect to be resolved." Connecticut Yankee Atomic Power Co., 2 AEC 393, n.2 (1964).

Apart from safety considerations, a distinct and separate aspect for the Board to resolve is in what respects is an unconventional and untested nuclear facility in Utah, with a relaxed seismic design earthquake standard, in the public interest. The fallout legacy on citizens of southern Utah from 121 atmospheric nuclear weapons tests at the Nevada test site conducted by Atomic Energy Commission ("AEC") from 1951 through 1962 cannot readily be ignored when the Board is considering the public interest in an untested nuclear facility to be sited in Utah. *See e.g.*, "The Forgotten Guinea Pigs: A Report on Health Effects of Low-Level Radiation Sustained as a Result of the Nuclear Weapons Testing Program Conducted by the United States Government," Staff of the House Committee on Interstate and Foreign Commerce, 96th Cong. (1980); Allen v. U.S., 588 F.Supp. 247 (D. Utah 1984) (bellwether case to determine federal government liability under discretionary function of the Federal Tort Claims Act for death/injury claims proximately caused by exposure to radioactive fallout from weapons testing), *reversed by* Allen v. U.S., 815 F.2d 1417 (10th Cir. 1987), *cert. denied*, 484 US 1004 (1988) (whether the AEC or its employees were negligent in failing to adequately protect the public is irrelevant to discretionary function under the Torts Claim Act); and Radiation Exposure Compensation Act of 1990, 42 U.S.C. § 2210, note (congressional finding that fallout emitted during the Government's above-ground nuclear tests in Nevada exposed individuals who lived in the downwind affected area in Nevada, Utah, and Arizona to radiation that is presumed to have generated an excess of cancers among these individuals). Using Utah as a test case to determine whether 4,000 spent nuclear fuel storage casks will perform adequately under seismic conditions smacks as another untested experiment that will again cause undue risk to Utah citizens. Any public interest in awarding PFS an exemption because design ground motions using a DSHA methodology exceed the proposed design values in PFS's SAR pales in comparison to the moral obligation of NRC – the successor to the AEC – to ensure that the federal government gives due consideration to who should bear the burden of the risks created by a

4,000 cask high level nuclear waste storage facility whose bold seismic design philosophy is parsimonious, unconventional and untested.

2. Guidance Documents, Expert Witness Testimony, Hearsay Evidence Standards

The standards for evaluating guidance documents, expert witness testimony, and hearsay evidence are fully set forth in State of Utah's Proposed Findings of Fact and Conclusions of Law Regarding Contention Utah K/Confederated Tribes B, August 30, 2002, Section III Subsections B-D; pgs 7-11. In sum:

(1) An NRC Guidance Documents "consistent with regulations and at least implicitly endorsed by the Commission is entitled to correspondingly special weight." Long Island Lighting Co. (Shoreham Nuclear Power Station, Unit 1), ALAB-900, 28 NRC 275, 290, *review denied*, CLI-88-11, 28 NRC 603 (1988).

(2) Expert testimony based upon insufficient facts or data should be given little weight. Moreover, testimony not founded on reliable principles and methods should be disregarded or given no weight. Daubert v. Merrell Dow Pharmaceuticals, Inc., 509 U.S. 579, 592 & n.10 (1993); FRE 702 Advisory Comm. Notes, 2000 Amendments.

(3) While hearsay evidence is admissible in NRC adjudicative proceedings, "only relevant, material, and reliable evidence which is not unduly repetitious will be admitted." 10 CFR §2.743(c)

III. Background

The proposed PFS ISFSI site is located on the Skull Valley Band of Goshute Indians Reservation in Skull Valley, Utah, about 50 miles southwest of Salt Lake City. Skull Valley lies within a tectonically active part of the eastern Basin and Range physiographic province and along the western margin of the Intermountain seismic belt. Staff Exh. Q at 2-1 to 2-4; Staff Exh. 62[9]; Con-SER at 2-29. The Skull Valley basin is bounded on the east by the Stansbury Mountains and the Stansbury fault and on the west by the Cedar Mountains and the East Cedar Mountain fault. Con-SER at 2-29. During geological and geophysical investigations of its proposed ISFSI site, PFS discovered two close, formerly unknown, capable faults, termed the East fault and the West fault. The four closest fault sources that pose a serious ground-shaking hazard at the PFS site are: the East fault, 0.6 miles away; the West fault, 1.2 miles away; the Stansbury fault, 5.6 miles away; and the East Cedar Mountain fault, 5.6 miles away. Con-SER at 2-47. The mean maximum magnitudes for these fault sources are 6.5, 6.4, 7.0, and 6.5, respectively. Id. The dominating contributions to the probabilistic seismic hazard at the PFS site for return periods greater than about a thousand years come from the Stansbury, East-Springline, and East Cedar Mountain faults. Con-SER at 2-46; State's Exh. 185 at Figure 6-12. When the total probabilistic hazard is deaggregated, the controlling ground motions are shown to be from nearby magnitude 6 to 7 earthquakes. Con-SER at 2-46; Tr. (Arabasz) at 9321.

In response to PFS's NRC June 1997 license application, the State filed a petition to intervene and was admitted as a party to the PFS licensing proceeding. *Private Fuel Storage, LLC* (Independent Spent Fuel Storage Installation), LBP-98-7, 47 NRC 142, 157, *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). In addition to its intervention petition, the State

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[9]Map, *Historic Seismicity and Nuclear Facilities in the United States*, prepared by J Stamatakos (June16, 2002), with insert of the Intermountain Seismic Belt.

filed a number of contentions, including Utah L, Geotechnical, which the Board admitted in its entirety. LBP-98-7, 47 NRC at 191, 247, 253.

The State's seismic exemption contention, Utah L part B[10], originated in response to PFS's April 2, 1999 request to use a probabilistic seismic hazard analysis ("PSHA") methodology rather than use a deterministic seismic hazard analysis ("DSHA") as required by current regulations, 10 CFR § 72.102. PFS initially requested to use a PSHA and a 1,000-year return period earthquake for design basis ground motions (*i.e.*, a 1,000-year design basis earthquake or "DBE")[11] but later amended the request to allow use of a 2,000-year DBE.[12] The State's contention Utah L Part B, filed April 30, 1999, was amended twice, first on January 26, 2000 in response to the Staff's original SER dated December 1999, and then on November 9, 2000, after the Staff issued its September 2000 final SER.[13] On January 31, 2001, the Board found the contention admissible, in part, and referred its ruling to the Commission, certifying the question of whether the contention should be further litigated. LBP-01-03, 53 NRC 84. In confirming the Board's ruling, the Commission stated,

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[10] State's Motion Requiring Applicant to Apply for Rule Waiver Under 10 CFR § 2.758(b) or in the Alternative Amendment to Utah Contention L (April 30, 1999).

[11] Applicant Exh. 247 [proposed], April 2, 1999 Letter from John D. Parkyn (PFS) to Mark Delligati re: Request for Exemption to 10 CFR § 72.102(f)(1), Seismic Design Requirement.

[12] Applicant Exh. 248 [proposed], August 24, 1999 Letter from John D. Parkyn to Mark Delligati re: Request for Exemption to 10 CFR72.102(f)(1), Seismic Design Requirement.

[13] The Board rejected Utah L Part B in LBP-99-21, 49 NRC 431 (and denied the State's motion to require the Applicant to apply for a rule waiver) (May 26, 1999), and in LBP-00-15, 51 NRC 313 (June 1, 2000), ruling in both cases that the Staff had not actually taken a final position and thus the State's challenges were premature.

> . . . what Utah proposes to litigate is whether PFS's ISFSI design, which is dependent on an exemption from otherwise controlling seismic regulations, is adequate to withstand plausible earthquake risks. Viewed this way, Utah's proposed revised Contention L (geotechnical) plainly puts into play safety issues that are material to licensing and suitable for consideration at an NRC hearing.

CLI-01-12, 53 NRC 459, 466 (2001).

In early 2001, PFS performed another geotechnical investigation after determining that it had not fully integrated all test data into its geotechnical characterization of the site. Its geotechnical investigation showed an increase in the design basis ground motions of about 35 percent above the design basis PFS was previously relying upon, and as a consequence, all dynamic analyses previously conducted had to be revised and re-evaluated. This resulted in a major amendment in March 2001 to the seismological, geotechnical, and structural design chapters of its Safety Analysis Report. In May 2001 the State filed Contention Utah QQ, addressing PFS's Canister Transfer Building ("CTB") design changes (including use of soil cement), revisions to storage pad analyses, soils analyses, soil-cement design calculations/analyses, and Holtec site-specific cask analyses.[14] The State modified Utah QQ in June and August of 2001 in response to PFS's several revisions of its stability analyses for the Canister Transfer Building and the cask storage pads.

The Board denied in toto PFS's December 2000 motion for summary disposition of Utah L Part A (the original geotechnical contention) and admitted Utah QQ and its modifications. LBP-01-39, 54 NRC 497 (2001). The Board also denied in toto PFS's

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[14]State of Utah's Request for Admission of Late-filed Contention Utah QQ (Seismic Stability) (May 16, 2001); *see also* April 26, 2001 Board Order at 3.

November 2001 motion for summary disposition of Utah L Part B (seismic exemption) in LBP-02-01, 55 NRC 11 (2002), and bound over for hearing, Utah L, Parts A and B and Utah QQ, with its modifications. Shortly thereafter, these two contentions were consolidated.[15] *See* PFS Exh. 237.

Unified Contention Utah L/QQ consists of five subparts. The parties resolved parts A (Surface faulting) and B (ground motions) by stipulation.[16] In Part C, the State alleges that: PFS has not conducted sufficient soil sampling and analysis; PFS has not performed physical property testing for engineering analysis to demonstrate that the soils and soil cement have an adequate margin against potential failure during a seismic event; and PFS has not proven its unprecedented soil cement (cement-treated soil) design concept or that by using those materials the CTB and storage pads can meet the 1.1 factor of safety against sliding.

Part D challenges PFS's failure to prove the storage pads, the CTB, their foundation systems, and the storage casks have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake. It also challenges PFS's failure to prove that, under design basis ground motions at the PFS site, the free standing HI-STORM 100 casks will not experience excessive sliding, uplift, collision, or tip over. In Part E the State's allegations relate to PFS's seismic exemption request, and asks that instead of a 2,000 year

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[15]Joint Submittal of Unified Geotechnical Contention, Utah L and Utah QQ (January 16, 2002).

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

DBE, PFS be required either to use a probabilistic methodology with a 10,000-year return period, comply with the existing deterministic analysis as required by 10 CFR § 72.102(f), or use a return period significantly greater than 2000 years.

The Licensing Board convened evidentiary hearings in Salt Lake City on April 8, 2002 on Contentions Utah K (Aircraft Crashes), Utah L/QQ (Geotechnical), Utah O (Hydrology) and SUWA B (Rail Alignment). Hearings in Salt Lake City continued until May 17, re-convened again June 3 through 8 and finally concluded in Rockville, Md. June 17 through July 3. The adjudicatory hearing relating specifically to Parts C, D, and E of Unified Contention Utah L/QQ were conducted on April 29-May 9, May 11-13, May 16-17, June 3-8 in Salt Lake City and on June 17-27 at NRC headquarters in Rockville, Maryland.

IV. Witnesses

During the direct case presentations, the parties called a total of twenty-two panels of witnesses. Generally, discussion of witness qualifications, where appropriate, can be found in the Findings that follow. But as PFS's case relies on two prime witnesses who have an economic self interest in promoting their product – the HI-STORM 100 storage cask – we digress at this point and discuss those witnesses.

PFS intends to procure its 4,000 cask facility spent fuel stainless steel canisters, storage casks and transportation casks, from one vendor: Holtec. PFS's key witnesses in the seismic hearings are the two principals from Holtec, its CEO and President, Dr. Krishna P. Singh, and its only executive Vice President, Dr. Alan I. Soler. Tr. (Singh/Soler) 5907-08. Under a memorandum of understanding with PFS, Holtec is to provide technical information and technical assistance to PFS, as requested, during the NRC application proceeding. Id. at 5910. Holtec started out in 1986 in the spent fuel business and only in mid-1990s did Holtec enter into the dry cask sales business. Id. at 5915-17. To date, only 27 HI-STORM 100 storage casks are in use or planned to be used at nuclear reactor sites. Id. at 5918-19. Sale of 4,000 HI-STORM 100 casks to PFS will enrich Holtec to the tune of hundreds of millions of dollars and reap enormous economic benefits for the Holtec principal owners Singh and Soler. Id. at 5920. Not to be overlooked by the Board in assessing the credibility of the two Holtec witnesses is the extreme bias and self-interest these two cask vendors have in the favorable outcome of the PFS proceeding. *See* Houston Lighting and Power Co (South Texas Project, Units 1 and 2) LBP-79-30, 10 NRC 594 (1979) (a witness' bias may reduce the weight of the witness' testimony); Power Authority of the State of New York (James A. FitzPatrick Nuclear Power Plant; Indian Point, Unit 3), CLI-01-14, 53 NRC 488, 516, n.41 (2001) (financial interests of the witnesses are taken into account in evaluating expert testimony).

In contrast, the State's experts opposing the Holtec witnesses – Dr. Farhang Ostadan, Dr. Steven Bartlett, Dr. Mohsin Khan and Dr. Marvin Resnikoff – have no personal financial stake in the outcome of this proceeding.

V. FINDINGS OF FACT AND CONCLUSIONS OF LAW

CONTENTION PART C: Characterization of Subsurface Soils

A. Issue: Should PFS be required to conduct additional sampling and analysis as well as physical property testing for engineering analysis to demonstrate that the soils, have an adequate margin of safety against potential failure during a seismic event.

B. Regulations/Guidance

10 CFR § 72.90: Evaluation of site characteristics that may directly affect the safety or environmental impact of the proposed facility.

10 C.F.R. § 72.102(c) "Sites other than bedrock sites must be evaluated for ... soil instability due to vibratory ground motion."

10 C.F.R. § 72.102(d) "Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading."

10 C.F.R. § 72.122(b)(1), structures, systems, and components important to safety ("SSCs") must be designed to accommodate the effects of, and be compatible with, site characteristics and environmental conditions associated with normal operation, maintenance and testing of the ISFSI, and to withstand postulated accidents.

10 C.F.R. § 72.122(b)(2) requires that SSCs be designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions.

Reg. Guide 1.132, Site Investigations for Foundations of Nuclear Power Plants, Appendix C (Sampling) gives recommended spacing and depth of borings.

C. Findings of Fact - Characterization of Subsurface Soils

Witnesses

1. As the issues presented in the soils characterization portion of the proceeding will frequently require the Board to give more credence to one person's testimony over another, we discuss here the relative strength, experience, and training of the parties' witnesses: PFS witness Mr. Paul J. Trudeau; Staff witness Dr. Goodluck I. Ofoegbu and State witness Dr. Steven F. Bartlett.

2. Mr. Trudeau is employed by PFS's ISFSI contractor Stone & Webster. He has a masters degree in civil engineering and has experience in geotechnical engineering. Trudeau/Wissa Tstmy, Post Tr. 10834, Resume at 2. Other than work at the PFS site, however, Mr. Trudeau has no geotechnical experience working in the Basin and Range Province and no experience in working with the Lake Bonneville sediments. Tr. (Trudeau) at 11740. Nor does Mr. Trudeau have any experience working at sites with strong ground motions, such as the PFS site. Id. at 6161.

3. Dr. Ofoegbu is employed by the Center for Nuclear Waste Regulatory Analyses as a principal engineer and his training is in geology and geotechnical engineering. Ofoegbu Tstmy, Post Tr. 11001 at 1 and Resume. Dr. Ofoegbu has no direct experience with the Bonneville clays and apparently has not personally correlated CPT data with other test data. Tr. (Ofoegbu) at 11787-90. Further Dr. Ofoegbu is not a registered engineer in the United States. Id. at 6550.

4. Dr. Bartlett holds a Ph.D. in Civil Engineering, and a B.S. in Geology. Bartlett Tstmy (Soils), Post Tr. 11822 at 1. Dr. Bartlett currently teaches graduate and undergraduate courses and conducts research in geotechnical earthquake engineering and related areas. He has 15 years of professional and research experience assessing the seismic behavior and stability of soils. Id. Apart from Dr. Bartlett's work at the DOE South Carolina Savannah River site, which pertained to seismic design of nuclear and hazardous waste facilities, much Dr. Bartlett's professional work has taken place in Utah giving Dr. Bartlett a solid background and knowledge in local soil conditions, especially the upper Lake Bonneville sediments. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 1. While working on the

Interstate 15 reconstruction project and other projects in Salt Lake City, Dr. Bartlett conducted soils investigations of the upper Bonneville clays and while working as a senior engineer at the DOE Savannah River, he was involved in the correlation of cone penetrometer test ("CPT") data with laboratory shear strength testing. Bartlett C.V. (State Exh. 92); Tr. (Bartlett) at 11881, 12010.

5. When it comes to professional experience and judgment in ascertaining the behavior of the upper Bonneville clays under seismic conditions, and correlation of the CPT data, the Board finds that Dr. Bartlett has a greater level of direct experience than do Mr. Trudeau and Dr. Ofoegbu and gives substantial weight to his testimony.

Background and Purpose of Characterizing Subsurface Soils

6. In general, competent soils have a high capacity to carry seismic loads without failure or excessive deformation. At the PFS site, the soil layers directly under the CTB and pad foundations are silty and clayey soils, which are generally considered less desirable soils due to their compressibility, deformability and relatively low strength. In recognition of the weakness of the supporting soil at the PFS site, PFS has introduced soil cement and cement-treated soil as an "engineering mechanism" in an attempt to improve generally poor soil conditions. Bartlett/Ostadan Tstmy (Part D, dynamic analysis), Post Tr. 7268 at 9; PFS Exh. JJJ, excerpts from PFS SAR.

7. The compressibility of soil at the PFS site is important because foundations overlying compressible soils will settle with time. Even more important to PFS's seismic analysis is soil deformation. Soil by its nature is a deformable body, which will strain or deform during an earthquake. Deformation of the soil can have many consequences to the

dynamic response and interaction with the foundation and supported structures. The dynamic response of the structure is affected by the response and deformation of the soil and the interaction of the masses of the foundations and supported structures. This type of interaction is called soil structure interaction. Bartlett/Ostadan Tstmy (Part D, dynamic analysis), Post Tr. 2268 at 9-10.

8. To show that the storage pads and the Canister Transfer Building will be adequately supported on a stable foundation during a seismic event, PFS's subsurface soils characterization must show that the site soils have adequate margins against potential failure during a seismic event. Bartlett Tstmy (Soils), Post Tr. 11822 at 2. The potential foundation failure modes that PFS has assessed in its seismic design calculations are sliding, overturning and bearing capacity. PFS Exhs. UU and VV.

9. 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. Bartlett Tstmy (Soils), Post Tr. 11822 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.

10. For extreme environmental events, such as earthquakes, a factor of safety of

at least 1.1 is considered inviolable. Tr. (Bartlett) at 11845-48. A factor of safety of 1.1, or 10 percent, is widely used in the engineering profession (id.; Tr. (Ebbeson) at 10802; (Trudeau) at 6163-64); it is the acceptance criterion in NUREG-0800, § 3.8.5, Section II, Subpart 5, Structural Acceptance Criteria for Seismic Category I Structures, p. 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. PFS Exhs. UU (at 22-26) and VV (at 21-24); Tr. (Trudeau) at 6163, 6169; Bartlett Tstmy (Soils), Post Tr. 11822 at 3. Even so, there is uncertainty in a 1.1 factor of safety and the closer one gets to 1.1 the higher the potential for failure of the foundation system. Tr. (Bartlett) at 11847.

11. The State challenges PFS's demonstration that the dynamic forces and the capacity of the soils have been properly described and used in PFS's calculation of a 1.1 factor of safety against sliding, bearing capacity and overturning of the pads and the CTB. Tr. (Bartlett) at 11845; *see also* Contention Part D. PFS is relying primarily on the shear strength of the soils to resist earthquake forces. Tr. (Bartlett) 11849. In its seismic calculations PFS computes factors of safety against sliding of 1.17 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 Tstmy (Soils), Post Tr. 11822 at 5-6; Tr. (Bartlett) at 11843. Based on PFS's calculated design values there is only a 6 to 15 percent margin in PFS's assumed capacity of the soils used in its design calculations before it would reach unacceptable performance. Bartlett Tstmy (Soils), Post Tr. 11822 at 5-6; Tr at 11843-49. Therefore, the soundness of PFS sampling, characterization, analysis and testing program of site soils is critical to PFS's demonstration that the site soils are adequate for the proposed foundation loadings and that

to show adequate an adequate margin of safety against potential failure during an earthquake.

Shear Strength of the Soils

12. Soil strength is composed of two main components, friction and cohesion. Mr. Trudeau testified that in the CTB and pads sliding stability calculations, PFS relies on the cohesive strength of the upper Bonneville clays to provide seismic stability. Tr. (Trudeau) at 6139.

13. The Board finds that the soil layer upon which PFS is relying to resist all the seismic loads is the upper Bonneville clays, and the primary mechanism PFS uses to calculate the resistance to sliding and bearing capacity failure of the pads and CTB is the shear resistance (*i.e.*, shear strength) of those soils. The Board further finds that an accurate and adequate characterization of the upper Bonneville clays is essential to PFS's demonstration that the pads and CTB will be supported on a stable foundation during a seismic event. Consequently, PFS must show that both the seismic performance and the shear strength characteristics of these soils throughout the pad emplacement area and foot print of the CTB are well-defined and understood.

14. The soil's shear strength is a function of its resistance to shear or sliding along a plane and the orientation of the potential failure plane. The dependency of shear strength upon the direction of shear is known shear anisotropy. When two planes – vertical, subvertical or horizontal, depending on the direction of the stresses – try to slide or shear, the amount of the soil's resistance is a function of its shear strength and can be measured by various laboratory shear tests (*e.g.*, triaxial compression, triaxial extension and direct shear). Tr. (Bartlett) 11839-40; Trudeau/Wissa Tstmy, Post Tr. 10834 at 17. The

sliding resistance of the pads and the stresses parallel to the base of the pad create a potential failure mechanism parallel to the fabric of the soil; this is typically measured by the direct shear test. Tr. (Bartlett) at 11933. Also, because earthquake shaking causes several cycles of stress reversal, these test are usually done in a cyclic manner.

15. The State's position is that there are fatal flaws in PFS's testing program because, *inter alia*, PFS has not adequately sampled and tested the upper Bonneville clays or established their stress-strain behavior under the range of cyclic strains imposed by the design basis earthquake. Bartlett Tstmy (Soils), Post Tr. 11822 at 5; Trudeau/Wissa Tstmy, Post Tr. 10834 at 17. At bottom, the State claims that by not adequately defining the lateral and vertical variability of the upper 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. The Staff and PFS do not share the State's concerns.

Soil Variability and Upper Bonneville Clays

16. There is inherent variability in soil shear strength because soils are deposited and influenced by natural processes. Variability results from vertical and horizontal changes in soil type and is strongly influenced by other geological factors and processes, such as soil density, void ratio, degree of consolidation, in situ moisture content, dessication (drying) and degree of natural cementation. Bartlett Tstmy (Soils), Post Tr. 11822 at 4.

17. The PFS site is a layered soil site. Below the surface of the thin eolian top soil are Lake Bonneville deposits consisting of an approximate three to thirteen foot layer of silty clay/clayey silt upper Lake Bonneville deposits; below that is an eight to ten foot layer

of Lake Bonneville sediments that are a little less clayey than the upper Lake Bonneville clays; and at a depth of about thirty feet, a sand layer. PFS Exh. 233a; Tr. (Trudeau) 11748; (Bartlett) 11830-32. All parties agree that for seismic analysis the upper Lake Bonneville clays are the critical soil layer at the PFS site.[17] Tr. (Trudeau) 11748; Tr. (Bartlett) at 11834.

18. The upper Bonneville clays are part of the Bonneville lake basin, which occupies northwestern Utah and small parts of adjacent Nevada and Idaho. Solomon Tstmy, Post Tr. 8965 at 5. It consists of a number of topographically closed structural basins in the northeastern Basin and Range province that were hydrologically connected during major lacustral episodes. Id. Lake Bonneville, the most recent major lake to have formed in the Bonneville lake basin, was essentially coincident with the last major ice age. Although other Quaternary lakes existed in the basin at various times prior to the Bonneville Lake cycle, Lake Bonneville was the deepest and most extensive lake in the series. Id. The lake level varied throughout its existence because of climate changes, changes in the relative proportion of inflow to the lake versus evaporative outflow, and the catastrophic failure of the lake threshold in southern Idaho. Id. Seasonal variation in climatic conditions affected particle size deposition of Lake Bonneville soils – during quiet times when there was little run off, finer-grained particles (clays) were deposited at the base of the lake but during runoffs silt sized particles predominate. Tr. (Bartlett) at 11831-32. This has resulted in the

[17]In this proceeding the State's soils witness, Dr. Bartlett, uses the term upper Lake Bonneville clays; PFS's soils witness, Mr. Trudeau, generally refers to the upper Lake Bonneville clays at the PFS site as Layer 2 soils; and the Staff's soils witness, Dr. Ofoegbu, refers to them as Layer 1B soils. *See e.g.*, Bartlett Tstmy (Soils), Post Tr. 11822 at 7; Trudeau/Wissa Tstmy, Post Tr. 10834 at 8; Ofoegbu Tstmy, Post Tr. 11001 at 7. The more descriptive term, upper Bonneville clays, is used in this document.

soils in the upper Bonneville sediments being finely bedded clays interbedded with silts and clayey silts and ones that consist of a microfabric of distinct layering. Id. In the aggregate the upper Bonneville sediments can be described as clayey silt/silty clay but they are finely bedded, plastic and compressible soils. Id.

19. Mr. Trudeau claims that the upper Bonneville clays are uniform but he did admit there is some variability in those soils. Tr. (Trudeau) at 11750. However, from the soils laboratory data PFS has not determined what percentage of the upper Bonneville clays are a plastic soil using the soil classifications (*i.e*, a CH or MH material).[18] Tr. (Trudeau) at 11751. Plastic (CH and MH) soils are of concern because of the generally low shear strength and high compressibility. As discussed *infra* the cone penetrometer testing results show significant variability across the site.

20. The Board finds that because PFS has failed to classify the plasticity of the upper Bonneville clays, it cannot claim that there is uniformity across the site.

Sampling at the PFS Site

21. The PFS site is one of complex layering and large size – the pad emplacement area covers approximately 51 acres. Tr. (Trudeau) at 11754. Also, PFS's design has small calculated margins against seismic failure and, what is more, the State has challenged those design calculations. *See* Unified Contention, Part D, PFS Exh. 237. Even if PFS's calculations are accepted, the storage pads have a 1.27 factor of safety against sliding and a 1.17 factor of safety against bearing capacity failure; and the CTB has a 1.26 factor of

[18]Plasticity relates to the stickiness of clay soils; non-plastic soils lack the cohesion that is caused by the clay sized particles. Tr. (Trudeau at 10874)

safety against sliding. Bartlett Tstmy (Soils), Post Tr. 11822 at 5. Given these margins of safety that are very close to minimum acceptable standards, the Board finds PFS must demonstrate that it has sufficient data and statistical analyses of critical layers to ensure that design soil properties have been conservatively selected and are supported by site-specific data.

22. In its seismic stability calculation, PFS Exhs. UU and VV, PFS relies on laboratory test data obtained from soil samples taken from boreholes drilled in or near the CTB and the pad emplacement area. Later PFS conducted 37 CPT soundings in the pad area but CPT data were not directly used in PFS's seismic stability calculations. Tr. (Bartlett) at 11874-75.

Density of Borings

23. The State claims that in many respects PFS's sampling of the pad emplacement area is grossly deficient. One of the State's claims is that PFS's sampling program does not conform to the density of borehole spacings recommended in Reg. Guide 1.132,[19] Appendix. C. Bartlett Tstmy (Soils), Post Tr. 11822 at 5-7; Tr. (Bartlett) at 11853, 11855, 11862-76. PFS admitted that it used Reg. Guide 1.132 to plan its field and laboratory investigations for the CTB. Bartlett Tstmy (Soils), Post Tr. 11822 at 6; State Exh. 98. Unlike borehole spacing used in the pad emplacement area, there is no disagreement that PFS has met the density of boreholes recommended in Reg. Guide 1.132, Appendix. C, for the Canister Transfer Building.

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[19] *Site Investigations for Foundations of Nuclear Power Plants*, Rev. 1 (March 1979), excerpts included as State Exh. 97

24. Appendix C of Reg. 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, Reg 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.[20] Even though this Reg. Guide is specific to nuclear power plants, the Board finds it appropriate guidance at the PFS site unless PFS has devised a more conservative sampling plan. This position is reinforced by the fact that PFS makes analogies to nuclear power plant ("NPP") guidance in arguing for the grant on its seismic exemption. *See* Contention Part E: Seismic Exemption Request.

25. PFS 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. Bartlett Tstmy (Soils), Post Tr. 11822 at 7; State Exh. 94 (SAR Fig 2.6-19, Rev. 22). 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. Bartlett Tstmy (Soils), Post Tr. 11822 at 6. Mr. Trudeau claims that seven borings, additional 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, and D3). Trudeau Rebuttal Tstmy (Soils), Post Tr. 11954 at 7-8; PFS Exh. 235 (SAR Fig. 2.6-19, Rev. 22).[21] Reviewing SAR Fig. 2.6-19, the Board notes 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

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[20]Reg. Guide 1.132, p. 1.132-3, 1.132-21, 1.132-22, State's Exh. 97; Bartlett Tstmy (Soils), Post Tr. 11822 at 6.

[21]PFS Exh. 235 and State Exh. 94 both contain the same SAR Fig. 2.6-19 (Rev. 22).

pads. 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. Reg. Guide 1.132 recommends borings be spaced one every 100 linear feet; therefore, the Board cannot consider those borings to meet the intent of Reg. Guide 1.132.

26. The Board finds that PFS used an approximate borehole and cone penetrometer spacing of about 221 feet for the pad area. SAR Figure 2.6-19, Rev. 22; State's Exh. 94.

27. The Board recognizes that some professional judgment is involved in PFS's sampling program. *See* Tr. (Bartlett) 11855; Trudeau Tr. 11774. Nonetheless, the Board finds PFS has significantly undersampled the pad emplacement area when compared with both the Canister Transfer Building sampling density and with the borehole spacings recommended by Reg. Guide 1.132. The undersampling is even more acute considering that only nine boreholes were drilled for the purpose of retrieving samples for laboratory testing and analysis of the approximate 51-acre pad emplacement site.

No Continuous Sampling at Depth to Establish Engineering Properties of the Upper Bonneville Clay

28. The State also claims that PFS has not continuously sampled the upper Bonneville clays with depth. Bartlett Tstmy (Soils), Post Tr. 11822 at 7. The importance of continuous sampling is to ascertain whether there are any zones of weak or unstable soils in the upper Bonneville clays. Id.; Trudeau Rebuttal (Soils), Post Tr. 11954 at 8-9.

29. Again we start with Reg. Guide 1.1.32, which recommends continuous sampling in a single boring or when that is not possible, then from adjacent closely spaced

borings in the immediate vicinity to represent the material in the omitted depth intervals.[22]

30. At the PFS site, undisturbed vertical sampling was conducted on five foot intervals and submitted for laboratory testing, the results of which became the design basis for the facility. Tr. (Bartlett) at 11864. The sampling program was completed in its entirety before PFS conducted cone penetrometer testing. Id.

31. Both PFS and the Staff attempt to rely on the CPT data in lieu of collecting samples continuously throughout the upper Bonneville clays to confirm that no weak layers are present. Trudeau Rebuttal (Soils), Post Tr. 11954 at 8; Ofoegbu Tstmy, Post Tr. 11001 at 9.

32. Cone penetrometer testing is an in-situ test that indirectly measures soil property; it is not a technique for obtaining undisturbed samples for laboratory testing. Tr. (Bartlett) at 11868-69. Cone penetrometer testing measures 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. (Trudeau) at 11728. PFS did not, however, conduct any statistical analysis of the CPT data to determine the variability of the upper Bonneville clays. PFS did

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[22]Reg. Guide 1.132, pp. 1.132-5 and 1.132-6 (State's Exh. 97) recommends:

> Relatively thin zones of weak or unstable soils may be contained within more competent materials and may affect the engineering characteristics or behavior of the soil or rock. Continuous sampling in subsequent borings is needed through these suspect zones. Where it is not possible to obtain continuous samples in a single boring, samples may be obtained from adjacent closely spaced borings in the immediate vicinity and may be used as representative of the material in the omitted depth intervals. Such a set of borings should be considered equivalent to one principal boring.

not analyze the range or standard deviation of the tip resistance across the site. Tr. (Bartlett) at 11939-40; (Trudeau) at 11771-72.

33. At the PFS site the CPT measured the relative stiffness of the soils, and this data would need to be correlated back to obtain engineering soil properties, such as shear strength. Tr. (Bartlett) at 11948-49. The visual representation of the CPT data on PFS Exh. 233a is a depiction of the relative stiffness of the tip resistance of the cone penetrometer, not of the shear strength of that layer. Id., and at 11868-73.

34. The purpose of continuous sampling at the PFS site is to ascertain through the recovery of samples and lab testing whether there are any weak zones in the depth of the upper Bonneville clays. Often, when it is difficult to conduct continuous sampling in a single bore hole an adjacent bore hole about five feet away may be used to stagger the investigation. Tr. (Bartlett) at 11865.

35. In the pad emplacement area, the bore holes and CPT are not adjacent to each other; they are spaced tens if not hundreds of feet apart. PFS Exh. 235; Tr. (Bartlett) at 11863-66.

36. We have noted that the three to thirteen thick upper Bonneville clays is the critical layer that may affect the engineering properties that PFS is relying upon. As we read it, and as applied to the PFS site, the intent of Reg. Guide 1.132 is to determine whether there are relatively thin zones of weak or unstable soils in the upper Bonneville clays. Sampling at five feet intervals in a three to thirteen foot layer does not constitute continuous sampling. In addition, the CPT testing was performed after the laboratory samples had been taken, therefore, the CPT data could not have been used to select the weakest zone for the

laboratory shear strength test program. Further, 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 Bonneville clays. *See* Tr (Bartlett) at 11864.

37. The Board finds that the upper Bonneville clays have not been continuously sampled and this introduces additional uncertainty into PFS's estimate of the shear strength of the upper Bonneville clays and subsequently into the factors of safety calculated for sliding and bearing capacity of the storage pads.

Extreme Undersampling to Measure Undrained Shear Strength of the Upper Bonneville Clays

38. PFS's design philosophy is to take the inertial forces caused by the movement or potential movement of the casks and pads and transfer those forces directly to the top of the upper Bonneville clays via the cement-treated soil. Ultimately, PFS relies upon the undrained shear strength of the upper Bonneville clay to resist the seismic motions. Tr. (Bartlett) at 11936.

39. To obtain a design value for the undrained shear strength, PFS first calculated the normal stress (i.e., the vertical stress at the base of the pads) and then developed a Mohr-Coulomb failure envelope to describe the shear strength resistance for the direct shear failure mode. PFS Exh. UU at Attachment C, p. C2; Tr. (Bartlett) at 11936-37; (Trudeau) at 11756-57.

40. The laboratory data used to develop the Mohr-Coulomb failure envelope was obtained from a single sample from one borehole in the pad area (Sample U-1 from Boring C-2) on which PFS conducted undrained direct shear strength testing. The envelope from

this single sample was then used to represent the undrained shear strength for the entire 51 acre pad emplacement area. Bartlett Tstmy (Soils), Post Tr. 11822 at 8. For the CTB, PFS obtained samples from two boreholes, both outside the CTB footprint, on which it conducted undrained direct shear strength testing. Id. at 9-10.

41. The direct shear sample for the pad area was collected from borehole C-2 in an approximately two foot long, three inch diameter Shelby tube. At the lab three approximately 2 inch long samples were selected from the Shelby tube for the direct shear test. Tr. (Bartlett) at 11934-35.

42. The sample for the one borehole in the pad emplacement area, resulted in a shear resistance for a vertical stress of 2 ksf of about 2.1 ksf. Tr. 11937. 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. 11938. Comparing the CTB and pad data, this strongly suggests 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 Bonneville clays at the PFS site.

43. PFS does not dispute that it collected a sample from only one borehole in the pad emplacement area. Instead PFS attempts to explain away its reliance on one sample from an approximate 15.5 million cubic foot volume of soil to ascertain the engineering properties of the upper Bonneville clays for the entire pad area by having the Board assume that the one borehole sample contained the weakest soil in the pad area. Trudeau Rebuttal (Soils), Post Tr. 11954 at 9.

44. PFS'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. Tr. (Trudeau) at 11774-76; Trudeau Rebuttal (Soils), Post Tr. 11954 at 9. The Board has already found that PFS has an insufficient density of borings in the pad area, and as discussed below, has not continuously sampled the upper Bonneville clays. Furthermore, 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. Tr. (Trudeau) at 11775-76.

45. PFS also argues it may use CPT data to predict the undrained shear strength of the upper Bonneville clays. Tr. (Trudeau) at 11955-62; PFS Exh. 238. Dr. Bartlett on the other hand argues that the CPT data may be used to predict a two fold variability in the upper Bonneville clays across the pad emplacement area. Bartlett Tstmy (Soils), Post Tr. 11822 at 9; Tr. at 11895-98; State Exh. 99.

46. To predict undrained shear strength from CPT data, a specific N sub K factor in the following equation from EPRI, State Exh. 100, Eq. 4-61 at p 4-55, 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. Id. at 4-57. *See also* Bartlett Tstmy (Soils), Post Tr. 11822 at 9; Tr. at 11939-40.

47. Mr. Trudeau relied on a N sub K factor developed by Cone Tec to predict undrained shear strength from the CPT data. This factor was developed from triaxial

compression test, which measure shear in a subvertical direction but the mode of failure for sliding and the PFS site is horizontal *i.e.,* direct shear. Tr. (Trudeau) at 11955-62; PFS Exh. 238; Tr (Bartlett) at 11972. The Board finds that the ConTec N_K factor is inappropriate for sliding calculations and likely overestimates the available shear resistance provided by the clayey soils.

48. Dr. Bartlett developed some hand drawn plots of the CPT data published in the ConTec report to show there is horizontal and vertical variability in the upper Bonneville clays. Bartlett Tstmy (Soils), Post Tr. 11822 at 9; 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. (Bartlett) 11898-99. Dr. Bartlett, relying on his past experience in testing 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 Bonneville clays across the pad emplacement area. Bartlett Tstmy (Soils), Post Tr. 11822 at 7, 9; Tr. at 11874-81.

49. As described in Dr. Bartlett's prefiled testimony, if one were to assume that one undrained shear strength value, 2.1 ksf, that PFS has obtained for the pad area, 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. Bartlett Tstmy (Soils), Post Tr. 11822 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. Id.

50. Based on the evidence, the Board is unconvinced that PFS has, in fact, conducted direct shear tests from samples obtained in a sufficient number of boreholes in

the pad area. While not as egregious as for the pad area, the shear strength testing of the upper Bonneville clays under the CTB from samples taken from only two boreholes outside the footprint of the building is insufficient to characterize the engineering properties of those soils.

51. As to the potential that the upper Bonneville clays may vary by a factor of two across the pad area, the Board finds that while the State's CPT plots are somewhat unrefined, it has nonetheless met its burden of going forward. Dr. Bartlett testified that this issue could easily be put to rest if such plots were developed electronically either by him or by PFS. No such evidence is in the record. The Board finds that PFS has not met its burden of characterizing the horizontal variability of the upper Bonneville clays.

52. Further because of the competing evidence of potentially a two fold variability in the clays across the pad area and PFS's unavailing attempt to correlate the CPT data to undrained strength, the Board is unconvinced that such a correlation can substitute for gross undersampling in the pad area. Such undersampling could introduce severe bias and potentially lead to overestimation of the shear strength capacity of the upper Bonneville clays to resist earthquake forces.

53. The Board finds that PFS that not demonstrated that it has developed engineering properties for its seismic stability analysis based on upper Bonneville clay soil samples that are representative of the pad emplacement area or the CTB footprint.

Other tests to determine engineering soil properties

<u>Cyclic Triaxial Tests</u>

54. The State claims that PFS has not adequately described the cyclic stress-strain

behavior of the upper Bonneville clays, and 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. Unified Contention Utah L/QQ, Parts 2.b & 3.a, at 4 (PFS Exh. 237; Bartlett Tstmy (Soils), Post Tr. 11822 at 10-12. For its part, PFS claims that instead of strain-controlled cyclic triaxial tests, it can extrapolate the results of its resonance column tests to establish the high-strain behavior that the PFS site soils may experience during an earthquake. Trudeau/Wissa Tstmy, Post Tr. 10834 at 18.

55. Earthquake loadings are cyclic in nature with several reversals in the direction of loading during a large earthquake. 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 Tstmy (Soils), Post Tr. 11822 at 10.

56. Dr. Bartlett expressed his concern that PFS had not tested the strength behavior of the upper Bonneville clays at a range of high strain levels, which is important because if their shear strength degrades due to earthquake cycling then there is an additional unconversatism introduced into PFS sliding calculations. Tr. (Bartlett) at 11992-93.

57. We next address Mr. Trudeau's claim that the resonance column test is a form of strain controlled cyclic triaxial testing. Trudeau/Wissa Tstmy, Post Tr. 10834 at 17.

58. The resonance column testing is a low strain dynamic test that measures a low strain shear moduli to shear strain of not more than about 0.1 percent. Tr. (Bartlett) at 11989-90. Mr. Trudeau testified that one can extrapolate data from the resonance column tests results. He compared a sample taken at 8 feet in the CTB area and tested to a shear

strain of 0.07% with a sample taken at a depth of 20 feet in the CTB area and tested to a shear strain of 0.15% to claim that the damping versus shear strain from these tests are very similar. Trudeau/Wissa Tstmy, Post Tr. 10834 at 18.

59. The modulus degradation and damping curves referred to above were developed by Geomatrix for analyzing ground motions that develop in the free field. Id. at 19; Tr. (Ostadan) at 7515 Geomatrix used the resonance column testing and published curves to extrapolate estimates of shear modulus and damping to higher strain levels for its free field ground response analyses. Tr. (Bartlett) 11992. However, from a shear strength perspective for foundation design, the upper Bonneville clays will have to resist strong ground motions including the forces resulting from the masses of the casks and pad. Tr. (Ostadan) at 7515; (Bartlett) at 11992. Attempting to extrapolate resonant column data from strains developed in the free field does not answer the question of whether the cyclic shear strength capacity of upper Bonneville clays is sufficient under the pads to resist seismic loads, or if the cyclic shear strength may in fact degrade at the higher levels of strain expected under the pads. If such shear strength degradation is possible, this could affect the sliding stability of the pads and CTB. Tr. (Bartlett) at 11993.

60. When strain controlled cyclic testing is absent, it is common practice to reduce the monotonic peak shear strength by about 10 to 20 percent to conservatively account for any potential strain softening of the soil due to cycling. Bartlett Tstmy (Soils), Post Tr. 11822 at 11; Bartlett Surrebuttal, Post Tr. 11982 at 2. This PFS has not done. Trudeau/Wissa Tstmy, Post Tr. 10834 at 18.

61. The Board is not convinced that PFS has demonstrated an acceptable level of

conservatism in its seismic stability calculations for the storage pads and CTB because PFS has not conducted strain-controlled cyclic triaxial testing or, alternatively, reduced the shear strength it estimated from monotonic shear strength testing for use in those sliding stability calculations. The Board also finds that the extrapolation from resonance column tests are no substitute for actual strain-controlled cyclic triaxial testing.

Anisotropy

62. The upper Lake Bonneville sediments have anisotropic shear strength properties, *i.e.*, the shear strength is a function of the direction of shear. Bartlett Tstmy (Soils), Post Tr. 11822 at 11. As described previously, the Bonneville clays have a layering or microfabric to them where there are alternating clays and more silty materials and it is this microfabric that creates the anisotropy. Tr. (Bartlett) at 11983-84. If, for example, the principal shearing stresses are applied in the lab soil sample in the vertical direction, by pushing on the sample it will reach a failure state and shear planes will develop at some angle to that principal direction of stress. If stress is placed in the horizontal direction so that the sample is sheared in a pure horizontal direction, then it will exhibit another strength. These characteristics of the upper Bonneville clays are a function of the microfabric of these soils. Tr. (Bartlett) at 11984.

63. The upper Lake Bonneville sediments are strongest in triaxial compression and weakest in triaxial extension. They have intermediate shear strength values when tested in direct shear. Previous studies performed on Lake Bonneville sediments have shown that the undrained shear strength in triaxial extension is approximately 60 percent of the undrained shear strength in triaxial compression. Bartlett Tstmy (Soils), Post Tr. 11822 at

11.

64. PFS has conducted laboratory tests monotonic (*i.e.*, non-cyclic) unconsolidated-undrained (UU) and consolidated undrained (CU) triaxial compression tests that measure the shear strength in a subvertical direction. Trudeau/Wissa Tstmy, Post Tr. 10834 at 9, 12-13. These tests do not measure horizontal failure planes. Tr. (Bartlett) at 11984-85.

65. 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. If significant anisotropy is present, then the use of triaxial compression tests overestimates the average shear resistance along the potential failure plane. Bartlett Tstmy (Soils), Post Tr. 11822 at 11-12.; State's Exh. 103, Figure 9 e. This issue has the greatest significance in analyzing the bearing capacity of the storage pads, due to their relatively narrow width (30 feet) and the small margin (*i.e.*, 5 percent) against seismic bearing capacity failure estimated by the Applicant. Bartlett Tstmy (Soils), Post Tr. 11822 at 11-12.

66. The Board find that the monotonic tests PFS has performed are not appropriate for horizontal failure planes and these tests will overestimate the available shear strength. The Board further find that PFS's calculations of the bearing capacity of the storage pads are potentially unconservative because PFS has not accounted for the anisotropy of the upper Bonneville clays.

D. <u>Summary</u>

PFS's pad design does not use deep foundations to transfer seismic loads and vibratory ground motions to deeper, more competent soil at the PFS site. PFS relies on the shear strength of the upper Bonneville clays to resist earthquake forces. With this design philosophy in mind, the Board finds there are too many uncertainties in PFS's characterization of the upper Bonneville clays. Our prime concern is the extreme undersampling and testing in the pad area – and to a lesser extent around the CTB footprint – where PFS relies on samples taken from one boring to determine the undrained shear strength of the upper Bonneville clays in the pad area. There are also uncertainties in PFS's program because PFS has an inadequate density of borings in the pad area to determine whether there are thin zones of weaker soils in the upper Bonneville clay layer; PFS has not continuously sampled the upper Bonneville clays at depth to determine vertical variability in that layer; and PFS has not conducted strain-controlled triaxial testing to adequately determine the stress-strain and strength behavior at the magnitude of cyclic strains imposed by the design basis earthquake from the inertial loadings resulting from the masses of the casks and pads. Because this test data is lacking, PFS has not demonstrated that the soil is capable of resisting the seismic forces at larger strain without a decrease in its cyclic shear strength. All of these shortcoming undermine PFS calculated factors of safety in its design calculations.

E. <u>Conclusions of Law:</u>

Based on the evidence presented, PFS's characterization of subsurface soils (*i.e.*, the upper Bonneville clays):

1. Is inadequate to determine the horizontal variability of the upper Bonneville clays.

2. Is inadequate to determine the vertical variability of the upper Bonneville clays.

3. Is inadequate to determine the shear strength properties of the upper Bonneville clays.

4. Is inadequate to ensure there will be no significant degradation of shear strength at shear strain levels caused by the design basis earthquake.

5. Is inadequate to determine whether significant anisotropy in the upper Bonneville clays could decrease the bearing capacity of those soils.

6. Is inadequate to demonstrate that PFS has met a factor of safety of 1.1 in its design calculations for the storage pads and CTB.

7. Does not comply with 10 C.F.R. §§ 72.90, 72.102(c), 72.102(d), or 72.122(b).

CONTENTION PART C: PFS's Proposed Use of Soil-Cement

A. Issue: Has PFS proven its soil cement/cement-treated soil design concept through qualified physical property testing and engineering analyses and shown by a preponderance of the evidence that the CTB and storage pads can meet the 1.1 factor of safety against sliding by relying on soil cement to provide dynamic stability to the CTB and storage pad foundation systems from a design basis earthquake?

B. Regulations/Guidance: *See* "Soils Characterization" above.

C. Findings of Fact - Soil Cement

Background on PFS's Use of Soil Cement

67. PFS intends to add Portland cement to soils in an effort to improve soils under and around the storage pads and around the CTB. PFS SAR at 2.6-108 to 2.6-120

(PFS Exh. JJJ, State Exh. 106).[23] PFS's purpose of treating soil with cement is, during an earthquake, to aid the storage pads in resisting sliding and to provide passive resistance to sliding of the CTB foundation. Trudeau/Wissa Tstmy, Post Tr. 10834 at 23-24.

68. The surficial layer of soil at the PFS site consists of eolian slits that vary in depth but, in general, this layer is on the order of about three feet. Underlying the eolian silts are the upper Bonneville clays. The surficial soil layer consists of silts from windblown deposit, some plastic material (*i.e.*, clays), and loosely formed soil grains that may vary in size. The eolian silts are an unsuitable subgrade for foundations and must be excavated. Trudeau/Wissa Tstmy, Post Tr. 10834 at 23; SAR Rev. 22 at 2.6-108 (PFS Exh. JJJ). PFS proposes to mix the eolian silts with a certain yet-to-be determined percentage of Portland cement. PFS's design concept is to place a layer of soil cement around the pads and CTB and a weaker layer of cement-treated soil under the pads. SAR at 2.6-108-108b (PFS Exh. JJJ).

69. Each of the 500 concrete storage pads at PFS has a dimension of 30 feet by 67 feet and a maximum thickness of three feet. SAR Fig. 1.2-1 (PFS Exh. 84). The storage casks are placed in a two by four array per pad, the pads are lined up five feet apart in the longitudinal direction and there is 35 foot spacing between the rows. Id. Below the three foot thick concrete storage pads is no more than two feet of cement-treated soil with a minimum compressive strength of 40 psi. The five feet layering around the pads will consist of an 8" top layer of aggregate, 2' 4" layer of 250 psi soil cement, underlain by a 2' layer of 40

[23]PFS Exh. JJJ and State Exh. 106 contain basically the same excerpts from the PFS SAR.

psi cement-treated soil. Tr. (Trudeau) at 10868-69; SAR Fig. 4.2-7 (State Exh. 212). Below the five foot layering under and around the pads are the upper Bonneville clays. State Exh. 212.

70. In its cask-tip over analysis, Holtec assumed certain properties of the concrete pad and underlying cement-treated soil. Consequently, there are certain constraints on the properties of those layers. The thickness of the concrete pad is a maximum of three feet and the compressive strength of the concrete storage pad is about 3,600 psi. Tr. (Soler) at 10575; Soler Rebuttal, Post Tr. 10557 at 3-4. The thickness of cement-treated soil is a minimum of one foot and a maximum of two feet and its compressive strength a minimum of 40 psi. The modulus of elasticity – also referred to as Young's modulus – of the cement-treated soil must be less than 75,000 psi. Tr. (Trudeau) at 10868-71; State Exh. 212.

71. The CTB basemat is approximately 240 feet by 280 feet and soil cement will extend that same basemat dimension around the building (*i.e.*, 240 feet to the east and west and 280 feet to the north and south). SAR 2.6-108a-108b (PFS Exh. JJJ). The soil cement around the CTB will be 5 feet thick and, in order to provide an adequate factor of safety against sliding, will have a minimum compressive strength of 250 psi. Id. at 2.6-108b.

72. State witness Dr. James K. Mitchell estimated that based on PFS's description of its soil cement design concept in the SAR, PFS Exh. JJJ, it will require about a one percent mix of cement to meet the target compressive strength of 40 psi for cement-treated soil under the pads and about 6 percent for the 250 psi soil cement around the pads and CTB. In addition to attainment of a designated compressive strength, durability testing is also required before the material to be classified as soil cement. Mitchell/Bartlett Tstmy,

Post Tr. 11033 at 7.

73. The Board notes that Dr. Mitchell has over 40 years' experience in geotechnical engineering, was on the University of California, Berkeley, Department of Civil Engineering faculty for more than 35 years, and has authored hundreds of publications, including two editions of "Fundamentals of Soils Behavior" – a graduate level text and reference. *See* Id. at 1-3; Mitchell C.V. (State Exh. 105). Specific to soil cement, Dr. Mitchell has spent many years researching the properties of cement stabilized soils and use of soil cement in pavement structures. His current work involves deep soil mixing and his past work included use of soil cement at the Koeberg nuclear power plant project in Cape Town, South Africa. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 3. With this wealth of professional experience, the Board gives particular deference to Dr. Mitchell's opinions on the topic of soil cement.

PFS's Soil Cement Testing Program to Date

74. PFS's soil cement[24] testing program is described in two documents: SAR at 2.6-108 through -121 (PFS Exh. JJJ), and Engineering Services Scope of Work for Laboratory Testing of Soil-Cement Mixes between Stone and Webster and Applied Geotechnical Engineering Consultants, Inc. ("AGEC"), ESSOW No. 05995.02-G010 (Rev. 0), dated January 21, 2001 (PFS Exh. GGG).[25]

[24]This term, at times, is used generically to also encompass cement-treated soil.

[25]The State's pre-filed testimony originally referred to State Exh. 107 which, apart from the confidentiality stamp, is the same as PFS Exh. GGG. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 5, 13 and fn. 1.

75. PFS, through Stone and Webster, retained AGEC to conduct laboratory testing of PFS site soils. PFS Exh. GGG; Trudeau/Wissa Tstmy, Post Tr. 10834 at 5. Under the contract's schedule and deliverables for the testing, premised on AGEC receiving notification to proceed no later than February 1, 2000, AGEC was to complete testing and delivery of a draft laboratory testing report to Stone and Webster by March 30, 2001. PFS Exh. GGG at 12 (section 5.5).

76. The contract called for AGEC to perform index property testing (water content, liquid limit, plastic limit, sieve analysis and hydrometer analysis) to determine the basic properties of the site soils. PFS Exh. GGG at 5; Trudeau/Wissa Tstmy, Post Tr. 10834 at 26-27. The next phase after index property testing is to conduct moisture-density testing on each soil-cement mixture to establish the relationship between the moisture content of the mixture and the resulting density when the mixture is compacted. Trudeau/Wissa Tstmy, Post Tr. 10834 at 27. AGEC has provided PFS preliminary results of those tests. Id. at 30.

77. AGEC soil classification results showed that there are some plastic soils in the eolian silts. State Exh. 213. The AGEC results listed a number of soils classified under the uniform system of soil classification as "CH" or "MH."[26] Soils designated as CH (high plastic clay) or MH (high plastic slit) are plastic soils. Tr. (Trudeau) at 10879. At the PFS site, the significance of the top layer of soils having some plasticity is that typically, the higher the degree of soil plasticity, the more cement needed to achieve a certain compressive

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[26]CH designates a high plastic clay and MH a high plastic silt (the "M" in MH is derived from "Moh" – the Swedish word for silt). Tr. (Mitchell) at 11214.

strength. Tr. (Trudeau) at 10875. And, generally, the more cement added to soil, the higher the modulus of elasticity. Tr. (Trudeau) 10876, 10939-40; (Wissa) 10986-87.

78. Under the contract, AGEC is also to conduct durability testing (wet-dry and freeze-thaw tests), compressive-strength testing, permeability testing and splitting tensile strength testing. PFS Exh. GGG at 6-7.

79. AGEC conducted durability tests but Mr. Trudeau determined those tests were unsatisfactory. Trudeau/Wissa Tstmy, Post Tr. 10834 at 30; Tr. (Trudeau) at 10976. Work has now come to a standstill on the PFS's soils testing program. Trudeau/Wissa Tstmy, Post Tr. 10834 at 30.

80. PFS has hired Dr. Anwar E. Wissa for litigation assistance. According to Mr. Trudeau, Dr. Wissa has the expertise to conduct more sophisticated testing – such as bond strength testing – required under PFS soil-cement testing program. Tr. (Trudeau) at 10977.

81. Dr. Wissa testified that PFS's soil cement laboratory testing program is adequate, and we emphasize, if properly implemented. Trudeau/Wissa Tstmy, Post Tr. 10834 at 30. Dr. Wissa also testified that if he were to conduct PFS's soil cement testing program, he would basically need to start all over so that he could vouch for the quality of his work. Tr. (Wissa) at 10980. He would use the AGEC test results only as a check on his results. Id. Dr. Wissa is talking with PFS about taking over the soil cement testing program, but to date, it is unknown who will be conducting PFS's testing program. Id.

82. It is apparent to the Board that the quality and success of PFS's 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 PFS's testing program.

PFS's Testing Program

83. After index property testing, moisture density testing and durability testing, PFS must then conduct compressive tests to ascertain what mix of Portland cement PFS needs to add to the silts to obtain 250 psi for the soil cement around the pads and around the CTB and 40 psi for cement-treated soil under the pads, as well as moduli testing of the cement-treated soil to determine whether PFS can achieve a mix that complies with the limitations of less than 75,000 psi Young's modulus. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 13.

84. Once PFS establishes a "recipe" for the cement-treated soil that will have a minimum compressive strength of 40 psi and not exceed a modulus of 75,000 psi, PFS will then have to conduct a bonding study and interface strength tests. Id.

85. The cement-treated soil will be constructed in six inch lifts. SAR at 2.6-118 (PFS Exh. JJJ). To show that it can meet a factor of safety of 1.1 in its pad sliding calculations, PFS is relying on the cohesion of the various interface layers to provide shear strength to resist sliding. Id. at 2.6-119-20.

86. PFS has yet to determine how to obtain sufficient bonding and adhesion between the cement-treated soil and the underlying upper Bonneville clays; between the various lifts of cement-treated soil; and between the cement-treated soil and the underside to the concrete storage pad. Trudeau/Wissa Tstmy, Post Tr. 10834 at 31; SAR at 2.6-118 (PFS Exh. JJJ).

87. Dr. Wissa expects the bond testing program would take about 2 to 3 months

and that it would take somewhere between six to nine months to complete the whole testing program. Tr. (Wissa) at 10865-66. Only then will PFS have proven the design. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 13 (*citing* Trudeau deposition, Tr. at 81).

PFS SAR Commitment to Shear Strength Testing

88. PFS makes the following commitment in the SAR:

> The on-going laboratory testing program will also include additional tests to confirm that the bond at the interfaces between concrete and soil-cement, soil-cement and soil-cement, and soil-cement and the site soils will exceed the strength of the in situ clayey soils. These tests will include direct shear tests, performed on specimens prepared from the site soils at various cement and moisture contents, in a manner similar to that used by DeGroot in his testing of bond along soil-cement interfaces.

SAR at 2.5-111 (PFS Exh. JJJ).

89. One of the issues confronting the Board in the soils portion of this proceeding is whether PFS has conducted adequate shear strength testing of the in situ clayey soils – the upper Bonneville clays. In that case PFS has already conducted direct shear strength tests, albeit from three soil samples taken from one borehole. PFS argued that the one set of direct shear tests on the upper Bonneville clays provides a minimum strength value for PFS's pad sliding analysis to meet a factor of safety of at least 1.1. Trudeau Rebuttal, Post Tr. 11954 at 9. Here PFS argues that for the eolian silts with an approximate one percent mix of Portland cement (*i.e.*, the soil layer that will be placed on top the upper Bonneville clays) there is no need to similarly demonstrate to the Board that they too will have adequate shear strength. PFS is relying on the shear strength of both the upper Bonneville clays and the cement-treated soils to meet a minimum factor of safety of 1.1 in the pad sliding analysis. Tr. (Trudeau) at 10838-39; 10928-29.

Construction of Soil Cement and Field Testing

90. First, the Board notes that PFS will construct the storage pads in stages over a number of years. PFS anticipates that it will receive 100 to 200 casks annually and that it will construct about one quarter of the storage pads in stage one, one quarter in stage two, and the remaining half in stage three. PFS FEIS at xxx, 2-3 to 2-4, 2-19 (Staff Exh. E).

91. PFS Exh. JJJ and testimony primarily by Dr. Wissa provide a window into how PFS will construct soil cement. *See e.g.*, Tr. (Wissa) at 10888-95. First the surface vegetation will be cleared, but as to the actual construction details, that would be left to the discretion of the contractor. According to Dr. Wissa, the bottleneck in the production of soil cement will depend on the equipment and facilities available to the contractor. Tr. (Wissa) at 10888-90.

92. One uncertainty during the construction stage is the effect construction and exposure of the subsurface layer will have on the upper Bonneville clays. 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). Tr. (Wissa) at 10889. When plastic soils tend to dry and shrink they tend to crack, and cracked soils loose their strength. Tr. (Mitchell) at 11119. If the clays gain moisture, they soften and decrease their undrained shear strength. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 12; Tr. (Mitchell) at 11151. PFS is relying on undrained shear strength ascertained from samples already collected at the site. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 12. The Board finds the samples of the upper Bonneville clays that PFS has used for testing may not be representative of actual field

conditions.

93. Another uncertainty is how much of the upper Bonneville clays will be removed along with the eolian silts during excavation of the site. There is insufficient evidence in the record to establish that the eolian silts have a uniform depth across the site and that the depth is three feet. PFS cannot rely on varying the thicknesses of cement-treated soils if the eolian silts are at depths greater than three feet because of Holtec's constraint that the cement-treated soil may only be one to two feet thick. If the upper Bonneville clays are excavated or disturbed, PFS intends to replace them with compacted clay fill. Tr. (Trudeau) at 10899-900. There is currently no analysis on whether the remolded upper Bonneville clays, consisting of compacted clay fill, will have the same shear strength as the undisturbed upper Bonneville clays that form the basis of PFS's pad sliding analysis. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 11-12. This has significance to PFS's 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. PFS, however, says it may possibly conduct rapid loading tests on compacted clay specimens. Tr. (Trudeau) at 10902. The Board finds that if such tests were, in fact, conducted, they would not be part of the record for this proceeding. Tr. (Trudeau) at 10982-83 ("I don't know what would drive the need to revise that calc [G(B) 04 (PFS Exh. UU)] again").

94. A further uncertainty is whether contractors employed by PFS to make soil cement will be able to identify plastic from non-plastic soils during excavation. PFS intends to separately stockpile non-plastic surficial soil to use in making cement-treated soil. Tr. (Trudeau) 10875-76. Dr. Wissa testified that a trained person could identify non-plastic soils

from plastic soil visually and by touch. Tr. (Wissa) at 10883-84.

95. If licensed, PFS may construct 500 pads, and this construction will take place over a number of years. The Board finds that if any of the various soil-cement contractors who work on the PFS project fail to correctly identify plastic from non-plastic soils, this failure could lead to a higher Young's modulus in the constructed cement-treated soil than analyzed in the Holtec cask tip over analysis.

96. Yet another uncertainty is the quality assurance/quality control measures that need to be instituted in order for the application of cement-treated soil to attain the qualities in the field that PFS aspires to demonstrate in the lab. To date, there is an inadequate description of the method or rate of production of cement-treated soil. Those details would probably be left to the bidding process. Tr. (Wissa) at 10884-90. Dr. Wissa testified that while not a pre-requisite to the award of a contract for the production and placement of cement-treated soil, it would be most efficient to have a centralized mixing plant at the site. Id. at 10890. Attaining the target cement-treated soil properties in the field will be affected by the quantity and timing of cement-treated soil production and placement as well as by the competency of the contractors in ascertaining what measures they will take to ensure adequate adhesion between interface layers.

97. The Board finds that to provide cohesive strength to the foundational soils under the pads, PFS is relying on the cohesive strength of the bond between interface layers at the bottom of the pad down through the lifts of cement-treated soil to the native soils. SAR at 2.5-114 (PFS Exh. JJJ). If the soil cement production cannot keep pace with the efficient placement of cement-treated soil lifts, this will negatively effect interface bonding,

thereby decreasing bond strength. Id. at 2.6-116. Further, the record illustrates that based on research by DeGroot there are many factors that can decrease bond strength. Id. at 2.6-116 (items 1 through 7). These factors, such as curing conditions, smoothness of compaction plane, etc., suggest to the Board that unless elaborate and detailed specifications are spelled out for contractors to follow, there is no assurance that laboratory test results can be achieved in the field.

Young's modulus

98. One of the most difficult tasks confronting PFS's 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. Tr. (Mitchell) at 11217; Trudeau/Wissa Tstmy, Post Tr. 10834 at 24-25.

99. For soils that have been studied, there is a limited amount of data indicating that low Young's modulus value may be achievable. Tr. (Mitchell) at 11159-60; 11200. Tests reported in the literature are based on site specific soils, and testing to establish a Young's modulus is a function of the site soils. Tr. (Wissa) at 10985. That is why testing by PFS to achieve such a low modulus is very important. Tr. (Mitchell) at 11159-60; 11200.

100. The cement-treated soil beneath the storage pads will need to endure for the lifetime of the facility (*i.e.*, 40 years). Cement-treated soil will continue to cure with time. Tr. (Mitchell) at 11216. As a result, the compressive strength and stiffness are likely to continually increase for some time period – months or years – after the material is first formed. Id. at 11216-17. Therefore, PFS may initially need to achieve a Young's modulus much lower than 75,000 psi to obtain a 75,000 psi modulus over time. Tr. (Mitchell) at

11216-18. PFS may, in fact, need to initially achieve a Young's modulus as low as 40,000 psi. Tr. (Mitchell) at 11222.

101. Dr. Wissa testified that moduli testing is done through trial and error by making up various proportions of site soils and cement. Tr. (Wissa) at 10914. To measure the static Young's modulus, Dr. Wissa proposes taking the initial tangent modulus from a compression test of soil cement. Tr. (Wissa) at 10914-15. Dr. Mitchell testified that one may use a static Young's modulus "[i]f you are able to correctly take into account the effect of the dynamic loading as opposed to the slower so-called static loading and if you are able to account for any strain dependence that there might be on the modulus itself." Tr. (Mitchell) at 11218.

102. There is a difference in modulus under dynamic loads than under static loads. Dr. Mitchell anticipated the static modulus would be lower than the dynamic modulus, maybe by one hundred percent or more. Tr. (Mitchell) at 11218-19.

103. The Board finds that the State has presented evidence casting significant doubt on the value ascribed to Young's modulus and how it may change over time and whether testing done under dynamic or static loads will also change the outcome of the moduli. Part of the Board's concern is that as PFS's modulus testing is intended to be done at some time in the future, it will likely require an exercise of discretion on the part of the Staff to determine whether PFS has, in fact, met a 75,000 psi over time and under dynamic loading conditions. This process would grant Staff post-licensing decision making authority that is much more than ministerial in nature.

Precedent for the Use of Soil Cement to Resist Sliding from Strong Ground Motions

104. The parties do not agree on whether PFS's use of soil cement to resist sliding during an earthquake is a unique application of soil cement.[27] Both the Staff and the Applicant point to the Koeberg nuclear power plant project in South Africa and the Boston Tunnel as examples of the use of soil cement to provide a buttressing or lateral resistance. 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 Tstmy, Post Tr. 11033 at 5-6; Tr. (Mitchell) 11051.

105. PFS defines "precedent" as not the specific use made of soil cement but "whether the application draws upon the same mechanical properties of the soil cement." Trudeau, Wissa Rebuttal, Post Tr. 11232 at 1. Precedent so defined leads PFS to propose that the Board should consider that the CTB and storage pads are relying on the shear and compressive strengths of soil cement and cement-treated soil to resist founding sliding just like other well known applications dating as far back as the early 1900s that rely on shear and compressive strength properties of soil cement. Id.

106. Commenting on PFS's postulate that a new precedent is defined by whether the application draws on different mechanical properties, Dr. Mitchell elucidated thus:

> If we think about that for a minute, it would seem to me that all soil cement projects have had a precedent, in that the properties, the mechanical properties that you're concerned about are the strength, the compressibility, the stiffness or modulus, and the permeability of the material. Every project involving soil cement that was constructed since that first road in, was it Sarasota, Florida in 1906 or 1908, draws on one or more of those properties.

[27] Again, "soil cement" is used generically to encompass cement-treated soil.

> Therefore, there have been, according to their [Wissa/Trudeau] definition, no precedents in the use of soil cement in nearly 100 years. And it's difficult for me to accept that.

Tr. (Mitchell) at 1255-56. Turning to the example of the two fold increase in concrete high rise structures, where both short and tall buildings rely on the compressive strength and modulus of concrete, Dr. Mitchell found it to be precedent setting going from 50 to 60 stories to those over 100 stories. Tr. (Mitchell) at 11256. Dr. Mitchell concluded, "to say you can only have a precedent by using different properties doesn't really hold much water." Id. at 11256-57; *see also* Tr. (Mitchell) at 11262.

107. Both PFS and the Staff refer to a paper by Lambrechts, *et al* to argue that deep soil mixing used in the Boston Tunnel project to resist lateral forces, is essentially the same use as PFS proposes. *See e.g.*, Trudeau, Wissa Rebuttal, Post Tr. 11232 at 2, Ofoegbu Tstmy, Post Tr. 11011 at 14.

108. Deep soil mixing for the Boston Central Artery Tunnel project has a similarity to PFS project – both projects rely on the strength and stiffness of the material but that is where the similarity ends. The Boston project deals with deep mixing soft clays with a very thick layer of treated material, whereas the treated soil at PFS – an eolian silt – is relatively thin; the Boston project involves a high water content material – in Utah the water content is much lower. At PFS the material is compacted whereas the Boston project is a deep soil mix (which is not a soil cement at all) with considerably different properties than the treated material at PFS. Tr. (Mitchell) at 11257-58.

109. The Board finds deep soil mixing, and the Boston Central Artery Tunnel project in particular, do not provide a precedent for the use of soil cement at the PFS site.

110. In the SAR, PFS relies on the Koeberg, South Africa nuclear power plant project as precedent. SAR at 2.6-113 (PFS Exh. JJJ). The Koeberg project involved removing a large volume of potentially liquefiable sand and replacing it with a sand-cement mix. Tr. (Mitchell) at 11258. At the PFS site there are no liquefiable soils and the site soils for mixing with cement are silts. Further, the Koeberg project, where the plant was designed as a base isolated structure, was not an attempt to mitigate sliding and did not rely on the buttressing effect of soil cement as will be the case with the CTB. Tr. (Mitchell) at 11258. Again, the Board finds that the liquefaction mitigation measures in South Africa do not provide precedent for PFS's use of soil cement.

111. Another example offered in the hearing was the use of lateral walls as providing resistance to lateral stress. At the PFS site, however, the dynamics are different in that the PFS project involves the horizontal potential sliding of the whole stabilized mass . Tr. (Mitchell) at 11258-59. The Board finds that lateral walls do not provide precedent for the use of soil cement at the PFS site.

112. Every new application of soil cement is subject to significant failure in the early stages of its use. When soil cement was first used as slope protection for dam embankments, for example, there were critical problems to overcome in bonding between soil cement layers. Tr. 11195. Subterranean use of soil cement lateral wall constructed through deep soil mixing has had at least one significant failure – the Bird Island Flats project in Boston, a project not mentioned in the Lambrechts' paper. Tr. (Mitchell) at 11193-94, 11219-20.

113. The Board is cognizant that there must always be a first new application of a

concept, but in such instances the Board expects that there will be sufficient testing prior to project approval to prove the concept.

114. The Board finds 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 favor of the State. The State convincingly shows, through its expert, Dr. Mitchell, that PFS cannot rely on the properties of the material as defining precedent. To accept PFS's position would turn the clock back to the early 1900s for the precedent setting use of soil cement in road construction.

115. The Board finds that PFS's intended use of soil cement is precedent setting. PFS has made many assumptions about the properties and behavior of soil cement but has not demonstrated them in this proceeding.

Degradation and Environmental Effects

116. The State maintains that even if PFS can prove that it design concept can meet target performance goals, there are still significant unresolved problems with PFS's use of soil cement. In addition to the problems, discussed *supra*, that may occur during construction, such as remolding of the upper Bonneville clays, the State claims there are potential failure mechanisms, such as the material cracking or being adversely affected by moisture, and from the debonding at interface layers, during the expected 40 year longevity of the soil cement and cement-treated soil. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 8-9.

117. The State has raised particular concerns about cracks in the soil cement and cement-treated soil, which if such were the case would lead to a loss of tensile strength in those materials. Id. The loss of tensile capacity is important because it will decrease some of

the structural competency of the soil cement layer. Tr. (Mitchell) at 11112. 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. Id. at 11111.

118. The water infiltration into the soil cement or cement-treated soil layers will potentially degrade those materials. Id. at 11147-49. 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. Id. at 11137-38.

119. The State raised the potential for the concrete slab to crack. For unknown reasons, concrete slabs can crack (e.g., garage floors, bridges). Tr. (Mitchell) at 11130; (Bartlett) at 11134-35. The concrete slab may also crack from a cask tip over or seismic event. Tr. (Mitchell) at 11130-32, (Bartlett) at 11133. Or 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. (Bartlett) at 11134-36.

120. There is a 8 inch layer of aggregate on top of the soil cement around the pads. SAR Gig. 4.2-7 (State Exh. 212). In the north-south direction there will essentially be a 30 foot wide gravel trench, and if there is no rapid drainage of water from the aggregate, it will create a bathtub effect. 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. (Mitchell) at 11137-44; (Bartlett) at 11141. Increased moisture could lead to the weakening of the upper Bonneville clays. Tr. (Mitchell) at 11147. 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. Tr. (Mitchell) at 11148-49.

Pad-to-pad interaction

121. Reinforcing bar in concrete allows that material to overcome its otherwise relative weakness in tension. Mitchell/Bartlett Tstmy, Post Tr. 11033 at 7; Tr. (Bartlett) at 11208. As 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 Tstmy, Post Tr. 11033 at 7-8. The casks, pads, soil cement and soils have very difference masses, and these masses will have different frequencies of vibration and behave differently during the cyclic forces from an earthquake. Id. at 8; Tr. (Bartlett) at 11206-07. 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.

122. PFS 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. Trudeau Tstmy (Dynamic Analysis), Post Tr. 6135 at 5, 10; Mitchell/Bartlett Tstmy, Post Tr. 11033 at 8; Tr. (Bartlett) at 11207. 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; 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. (Bartlett) at 11206-07.

123. 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 Tstmy, Post Tr. 11033 at 7-8; Tr. (Bartlett) at 11209.

124. PFS is relying on the shear strength of both the upper Bonneville clays and the cement-treated soils to meet a minimum factor of safety of 1.1 in the pad sliding analysis. While PFS has attempted to demonstrate the shear strength of the upper Bonneville clays, no such demonstration has been attempted for the shear strength of the cement-treated soil.

Summary

PFS has not shown that use of cement-treated soil and/or soil cement will have adequate shear strength, will assist PFS in meeting a factor of safety of at least 1.1, or provide an acceptable seismic design for storage of spent nuclear fuel at the Skull Valley site.

D. Conclusions of Law

125. Here the Board is confronted with the situation of whether PFS's soil cement program is merely procedural or whether there are substantive requirements that PFS must meet prior to the grant of a license. The Staff's position is that testing of soil cement to show that PFS has met its design requirements is procedural and does not need to be accomplished in order for NRC to issue a license. Tr.(Ofoegbu) at 6650 ("I've been told that this [PFS's future soil cement testing] is consistent with the NRC process."). The State's position, contrary to that of the Staff, is that PFS's future soil cement program is part of the demonstration PFS must make to prove its design concept and to satisfy 10 CFR § 72.102. That demonstration, says the State, must be satisfied before PFS earns the privilege of

obtaining a license from the Commission.

126. 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 CFR § 72.102(d).

127. The items that PFS has yet to demonstrate in its soil cement program include the following: adequate classification of surficial soils; selection of soils for further testing including durability (wet-dry, freeze-thaw); 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 testing to determine bonding and adhesion between interface layers. Then there are uncertainties about implementing PFS's laboratory testing program, whether confirmatory tests can be successfully carried out in the field and also whether construction techniques or activities will degrade the strength of the upper Bonneville clays.

128. An ISFSI license is a combined construction and operating license; for a nuclear power plant, licensing is separate for the construction and the operation of the plant. *Cf* 10 CFR Part 72 *with* 10 CFR Part 50. The issue here is whether PFS can rely on future actions to meet its burden for the construction of 500 storage pads, *i.e.*, a significant safety element in the construction of the ISFSI. The Board turns to Consumers Power Co. (Midland Plant, Units 1 & 2), ALAB-315, 3 NRC 101 (1976), in which the Appellate Board analyzed the evidentiary burden in a show cause hearing for alleged violation of a NPP construction permit. The Appellate Board held, as the company or its contractors are the ones most likely to possess the requisite information and be aware of the relevant

construction details, the construction permit holder has the burden of proving that its nuclear power plant is being built in conformity the Commission's safety regulations. 3 NRC at 109. The Board finds that State has the burden of going forward but the ultimate burden is on PFS to demonstrate during this licensing proceeding that there is sufficient evidence in the record that PFS will, in fact, meet the target performance goals for cement-treated soils and soil cement with an adequate factor of safety.[28]

129. The record does not contain any hard and fast numbers – other than cement-treated soil with a 40 psi compressive strength and less than 75,000 psi Young's modulus – against which the Staff may carry out an inspection. Potential defects could occur in the cement-treated soil that will be placed under the 500 pads because of the yet to be achieved cement-treated soil properties and bonding from samples tested in the lab, the construction techniques used to implement the PFS soil cement program, and the fact that the cement-treated soil will be subgrade and only be visible during construction. Tr. (Ofoegbu) at 6591, 6650. Given the importance of these factors to the overall safety of the facility, the Board finds that the level of judgment that would be required of the Staff during its inspections and in its review of PFS's testing program is inappropriate, and that judgment is more appropriately exercised in the course of licensing.

130. The Board finds that neither PFS nor the Staff can rely on NRC's inspection program to ensure that placement of cement-treated soil under the 500 pads at PFS will

[28]As the Appellate Board stated: "A rule that places the burden of proving a fact on the party who presumably has peculiar means of knowledge enabling him to prove its truth or falsity is neither novel nor untoward, particularly when the ultimate issue is one of public safety." Consumers Union, 3 NRC at 109 (citation omitted).

meet PFS's target performance goals. The NRC's inspection program is not geared to detect latent defects or to be a watchdog at every step of construction. Consumers Power Co. (Midland Plant, Units 1 & 2), CLI-74-3, 7 AEC 7, 11 (1974). In Consumers Power the Commission stated: "cadwell[29] deficiencies represent potential latent defects in the structure housing the reactor" and ". . . present[ ] an immediate threat because this stage of construction is the only one at which such deficiencies can be detected. . . Our inspection system is not designed to and cannot assume such tasks . . ." Id.

131. The State has met its burden of going forward by presenting evidence that PFS's use of soil cement is precedent-setting, that PFS's testing program should be conducted prior to license issuance, and that PFS's sliding stability calculations for the storage pads and the CTB are not based on site-specific investigations and laboratory analyses that show that soil cement and cement-treated soil are adequate for foundation loadings.

132. PFS has not met its burden to show in its sliding analyses, PFS Exh. UU, that the shear strength of both the upper Bonneville clays and the cement-treated soil will meet a minimum factor of safety of 1.1.

133. Neither PFS nor the Staff have demonstrated that PFS can meet the requirements of 10 CFR § 72.102(d). Also, the Board finds that PFS has not met the requirements of 10 CFR §§ 72.90, 72.102(c), or 72.122(b).

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[29]"Cadwelling" is a process of fusing together metal bars used in reinforced concrete construction. Consumers Power Co. (Midland Plant, Units 1 & 2), ALAB-315, 3 NRC 101, 108 (1976) (citation omitted).

134. Further the State's evidence raises the potential that even if PFS could prove its soil cement design concept, there may be significant degradation of soil cement over time from such causes at shrinkage cracks, debonding along interface layers, and moisture infiltration. PFS has not produced evidence to overcome the concerns presented by the State.

135. Finally, the State raises the concern about pad-to-pad interaction. There is some overlap between the concerns specific to use of soil cement and the way in which soil cement will react to dynamic forces during an earthquake. Because of the overlap with the dynamic analysis portion of the contention, the Board defers ruling on this issue until Contention Part D.

CONTENTION PART D: Seismic Design and Foundation Stability

A. Issue: Has PFS met its burden of showing that the storage pads, the CTB, their foundations systems, and the storage casks have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake?

B. Regulations/Guidance

10 CFR § 72.90: Evaluation of site characteristics that may directly affect the safety or environmental impact of the proposed facility.

10 C.F.R. § 72.102(c): "Sites other than bedrock sites must be evaluated for ... soil instability due to vibratory ground motion."

10 C.F.R. § 72.102(d): "Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading."

10 CFR § 72.120(a): general design criterial for the proposed storage facility.

10 C.F.R. § 72.122(b)(1): structures, systems, and components important to safety ("SSCs") must be designed to accommodate the effects of, and be compatible with, site characteristics and environmental conditions associated with normal operation, maintenance, and testing of the ISFSI, and to withstand postulated accidents.

10 C.F.R. § 72.122(b)(2) requires that SSCs be designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions.

NUREG-1567, *Standard Review Plan for Spent Fuel Dry Storage Facilities* (March 2002).

NUREG-1617, *Standard Review Plan for Transportation Packages for Spent Nuclear Fuel* (March 2000).

NUREG-0800, Sections 3.7.1 (Seismic Design Parameters), 3.7.2 (Seismic System Analysis), 3.8.4 (Other Seismic Category I), 3.8.5 (Foundations).

C. Findings of Fact

Witnesses

136. Dr. Farhang Ostadan has considerable expertise in the dynamic analysis of nuclear facilities. An expert in soil structure interaction from the effects of earthquakes, he has more than 15 years experience in the dynamic analysis and seismic safety evaluation of above and under ground structures and subsurface materials. State Exh. 110; Tr. (Ostadan) at 7369. He co-authored and implemented the computer program "SASSI" used by worldwide as the industry practice for modeling soil structure interaction. State Exh. 110; Tr. (Ostadan) at 7352. Dr. Ostadan has a PhD in civil engineering. State Exh. 110; Tr. (Ostadan) at 7259.

137. Dr. Bartlett, an expert in soil behavior under seismic loading, has performed seismic analyses for the Department of Energy High Level Waste Facilities at Savannah River. State Exh. 92; Tr. (Bartlett) at 7818. As a project engineer for Woodward-Clyde and a research project manager at the Utah Department of Transportation, he researched and analyzed the construction and performance of lime-cement-treated soil and the consolidation, shear strength, and seismic response of the Lake Bonneville deposits, the same deposits that are found at the PFS site. State Exh. 92; Tr. (Bartlett) at 7725. Dr. Bartlett has a PhD in civil engineering (geotechinical emphasis); in his present position he conducts research and teaches graduate and undergraduate courses in geotechnical engineering at the University of Utah.

138. For this part of the proceeding, PFS presented the following witnesses: the Holtec cask vendor CEO/president, Dr. Singh and its executive vice president, Dr. Soler; from the project architect and engineering firm, Stone and Webster, Mr. Ebbeson and Mr. Trudeau; other subcontractors who have worked on the PFS project, Dr. Weng Tseng from ICEC and Dr. Robert Youngs from Geomatrix.

139. As described in more detail *supra* and in Cask Stability *infra*, Dr. Singh and Dr. Soler have an extreme economic bias and self-interest in the outcome of this proceeding. The HI-STORM 100 casks is the only storage presently contemplated for the PFS project. Tr. (Singh/Soler) 5907-08. The potential to sell 4,000 HI-STORM 100 casks and other related items, such as the HI-TRAC transfer casks and HI-STORM transportation, is yield significant financial rewards to these two witnesses.

140. The Board finds that Dr. Singh and Dr. Soler have a bias and interest in the

outcome of this case. Accordingly, we find it appropriate to consider those biases and interest in determining the weight to be given to their testimony and other the evidence relevant thereto.

141. The Applicant's other witnesses rely heavily on the fragmented and biased reports prepared by Holtec. Mr. Paul Trudeau, responsible for the CTB and storage pad stability design calculation, relies on reports prepared by Holtec for significant portions of his testimony (Tr. (Trudeau) at 6201, 6203, 6297-98, 6305-060), as does Dr. Weng Tseng who is primarily responsible for the design of the storage pad. Tr. (Tseng) at 10760, 70762. Both Mr. Bruce Ebbeson and Dr. Robert Youngs rely on the data from Holtec. Tr. (Ebbeson) at 6376, 6380; Tr. (Youngs) at 10483-84. The Board finds that the reliance by these witnesses on Holtec's reports affects these witnesses' opinions to the extent that the same biases, errors, or unconservative assumptions are found in the Holtec documents.

142. The Applicant relies on Mr. Trudeau's testimony relating to the sliding, overturning and bearning capacity failure of the CTB and the storage pads. Of note, Mr. Trudeau has no previous experience in working at facilities in high seismic areas or with the upper Bonneville clays. Tr. (Trudeau) at 6270. Further Mr. Trudeau no experience at all in soil structure interaction analysis. Id. at 6163.

143. The Staff presented two non-NRC employee witnesses: Dr. Goodluck Ofoegbu from the Southwest Research Institute and Mr. Daniel Pomerening from the Center for Nuclear Waste Regulatory Analyses. Ofoegbu/Pomerening Tstmy, Post Tr. 6496 at 1-3.

Background

144. The Board's starting point is that PFS's ISFSI design is not exemplary of a design for a site, such as the Skull Valley site, that has relatively soft soils and strong ground motions. The State goes so far as to claim that it is difficult to contemplate a design that is as cheap and unsafe as the one PFS proposes. As an example of a rigorous design, the State points to the proposed Diablo Canyon ISFSI where casks are anchored into the storage pad; the pads are 7 feet thick; and are anchored into bedrock. Tr. (Ostadan) at 7383-84; (Bartlett) at 7385-86. In contrast to the Diablo Canyon ISFSI, PFS intends to use unanchored casks sitting on three foot thick unsupported concrete storage pads, bonded to a layer of yet-to-be tested cement-treated soil, which in turn will be bonded to a layer of soft soils. Still, the Board must deal with the design PFS has presented but, nonetheless, it keeps in mind the meagerness of PFS's design.

145. The heart of the dispute between the State and PFS/NRC Staff is the premise underlying PFS's design philosophy and the adequacy of PFS's design calculations to support the seismic performance of the Canister Transfer Building, the casks, the storage pads, and underlying soils at peak horizontal and vertical ground accelerations of 0.7g or greater.

146. 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. Seismic forces estimated in this one design calculation propagate throughout other seismic and engineering calculations that PFS is relying upon to demonstrate the performance of its ISFSI during an earthquake. Tr. (Ostadan) at 7344-45. Also, the Holtec nonlinear analyses are being used as the basis to

predict cask movement atop the pads; to predict seismic loads transferred to the soil cement[30], cement-treated soil, and soils; to perform the structural design of the storage pads; to predict pad sliding; and to analyze the effects of soil structure interaction on the response of the pads and the casks. Tr. (Ostadan) at 7341, 7384-85; Youngs/Tseng Tstmy, Post Tr. 5529 at 10, 15.

147. In seismic engineering, the concern is with both the capacity of the structures, foundations and soils, and the demand placed on that capacity by an earthquake. Ostadan/Bartlett Tstmy, Post Tr. 7268 at 6. Knowledge of input parameters is often limited – there are uncertainties in the prediction of ground motion and the seismic performance of structures and foundations. Tr. (Ostadan) at 7340. Seismology and earthquake-resistant design evolves every time there is an earthquake by analyzing the seismic performance of structures and foundations and using this design experience in the future. Id. This has led to a significant knowledge base about conventional designs, their design capabilities, and how they perform during an earthquake. Id. at 7343.

148. Performance data from real earthquakes, however, do not assist the Board in evaluating the design for the PFS ISFSI. As the State points out, PFS's design is unconventional and unprecedented. The unproven features at the PFS site (where there are known major active faults under or near the site and where high, strong ground motions are expected) include: unanchored cylindrical casks; 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

[30]At times "soil cement" is used generically to encompass cement-treated soil.

the storage pads and their foundations; shallowly embedded storage pads founded on a compressible clay with the potential for several inches of settlement; 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. (Ostadan & Bartlett) at 7724-29; 10287; 7351, 10286; Contention Part C, Soils and Soil Cement *supra*. On top of this, there is relatively little margin for error in PFS's design. For example, if the Applicant has under-predicted dynamic loads by only twenty percent or over-predicted capacity by twenty percent then it is questionable whether PFS's design will perform during an earthquake. Tr. (Ostadan) at 7342-43.

149. The Board concludes that there is no engineering precedence or seismic performance data from previous earthquakes the Board can rely upon to give it an indication of the performance of the PFS ISFSI under earthquake conditions.

150. In addition to no performance data, the Board has no test data it can rely upon to evaluate the performance of the casks. The HI-STORM Holtec casks intended for PFS have not been tested either experimentally or during an actual earthquake to determine their performance under earthquake conditions. Holtec only entered the dry cask business in the mid-1990s. About twelve HI-STORM 100 casks are in use and about fifteen others have been delivered to nuclear reactor sites. Tr. (Singh) at 5915-16; 5918-19.

151. The Board received testimony on the potential to acquire experimental data from shake table tests for the design of the casks on the pad. *See e.g.*, Ostadan/Bartlett Tstmy, Post Tr. 7268 at 9, Tr. (Ostadan) at 7404-14; (Luk) 11569-72; (Singh) 9733, 9738-48. Apparently, shake table tests are routinely conducted in earthquake engineering practice. Tr.

(Ostadan) at 7406, 7412-13.

152. A shake table is a steel surface table, underneath which are specially designed hydraulic jacks that can react in three directions to the acceleration time histories fed into the system by a computer. Tr. (Ostadan) at 7412-13. Dr. Luk testified that the University of California at San Diego recently received funding to build an outdoor state-of the art shake table. The facility, expected to be ready next spring, should be able to handle a full sized cask. Tr. (Luk) at 11569-72.

153. Frequently, shake table tests are done with scaled down models. Dr. Ostadan testified that a scale model of the cask and pad could be used and sliding of the cask simulated on a shake table. Tr. (Ostadan) at 7412-13. By modeling sliding of the casks on the pad, the data from the shake table tests could be used to calibrate Holtec's nonlinear model for prediction of cask movement and the sensitivity of unknown parameters (such as contact stiffness). Tr. (Ostadan) 7407–08.

154. Dr. Luk was of the opinion that a shake table test would be useful in confirming his analysis and because of the uncertainties in nonlinear analysis there is an incentive to push for such testing. Tr. (Luk) at 11569, 11572. In fact, Dr. Luk even approached Holtec to donate a cask for a shake table test. Tr. (Luk) at 11568-69. On a second round of redirect rebuttal testimony, Dr. Luk equivocated somewhat on the use of shake table tests. He still maintained that shake table test results would help confirm his analysis model and guide him in improving his model but he confessed that if questionable output data were obtained from shake table testing, then it would be of no use. Tr. (Luk) at 11680–81.

155. In correspondence from Holtec to PFS during 1997 and 1998, Dr. Singh recommended that Holtec perform scale model shake table testing as part of the seismic qualification of the free standing HI-STORM 100 casks. State Exh. 197A[31] at bates No. 47928; *see also* Tr. (Singh) at 9733, 9738-48. An internal PFS memo and testimony by Dr. Singh make it clear that NRC was reluctant to approve the HI-STAR and HI-STORM generic certificate of compliance for high seismic areas. State Exh. 197A at bates No. 47925 ("I think it is clear from the NRC that they do not want to review the high seismic situation on the initial HI-STAR/HI-STORM dockets. Whether the NRC will allow amending the initial C of C, or treat seismic issues on the high seismic sites only on the site specific docket isn't clear."). Following NRC instructions, Holtec secured a docket for a general license for high seismic scenarios and requested funding for seismic qualification testing from Pacific Gas & Electric for anchored HI-STORM casks and from PFS for free standing HI-STORM casks. State Exh. 197 at bates No. 47928; Tr. (Singh) at 9744-45. As to NRC's statement from a July 1997 meeting with Holtec that NRC suggested decoupling the high seismic issue from the generic Certificate of Compliance ("CoC") and instead endorsed Holtec's proposal to experimentally confirm the seismic analysis approach, Dr. Singh testified: "The statements made here are politically correct. You're dealing with the NRC." Tr. (Singh) at 9748.

156. The Board finds that it was possible for the Applicant to have acquired experimental test data on the performance of the HI-STORM 100 casks. The Applicant

[31] State Exh. 197A consists of PFS discovery documents marked confidential; the confidentiality claim was removed per *Joint Report on Status of Utah Contention L/QQ Exh.s and Other Open Items from Hearing Concerning Utah Contention L/QQ* (July 31, 2002).

could have conducted shake table tests on a scaled model cask or, in the United States by next spring, may acquire such data by conducting shake table tests on a full size cask. The Applicant, however, has chosen not to do so.

157. What the Board is left with is Holtec's design calculation and engineering judgment. The Board makes particular note, here, 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. Texas Utilities Electric Co. (Comanche Peak Steam Electric Station, Units 1 and 2), LBP-84-10, 19 NRC 509, 518 (1984). Further, 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. Virginia Electric and Power Co. (North Anna Nuclear Power Station, Units 1 and 2 ALAB-555, 10 N.R.C. 23, 27 (1979).

158. Holtec's design calculation is based entirely on a nonlinear computer program. Nonlinear analyses, however, 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. (Ostadan) at 7335-36; 7551-52; *see also* (Khan) at 9358; Tr. (Luk) at 11572. As is evident from the Cask Stability section *infra*, small changes in an input parameter, such as contact stiffness or damping, could dramatically change the result of nonlinear analyses. Tr. (Ostadan) at 7336, 7352; Khan/Ostadan Tstmy, Post Tr. 7123 at 11-12. Also, PFS has no performance data or experience data to calibrate its design calculation. With this in mind, we turn now to the specifics of the PFS project, its seismic design, and foundation stability

analyses.

The PFS Facility

159. If licensed, PFS could store up to 4,000 casks of spent nuclear fuel containing 40,000 metric tons of uranium ("MTU"). Con-SER 4-8. Each cask may store up to 24 pressurized water reactor fuel assemblies or 68 boiling water reactor assemblies. Con-SER at 1-2 (Staff Exh. C). PFS anticipates a 40 year design life for the ISFSI. Con-SER at 4-8.

160. The casks will be stored in a 2 x 4 array on three foot thick concrete pads. SAR Fig 4.2-7 (State Exh. 175 and 212). There will two quadrants of 250 pads, *i.e.*, a total of 500 pads. SAR Fig. 1.2-1 (Staff Exh. X). Each pad is separated by five feet in longitudinal (north-south) direction and 35 feet in the east-west direction. Id. As described in more detail in Contention Part C *supra*, the storage pads will be supported by cement-treated soil and compressible clays and surrounded by soil cement. SAR Fig 4.2-7 (State Exh. 175 and 212). PFS's design model anticipates that earthquake forces will be transmitted down from the casks, pad and cement-treated soils to the underlying upper Bonneville clays. Tr. (Bartlett) at 10294-96; *see also* Contention Part C, *supra*.

161. Under the PFS license application, the only cask that PFS will employ for storage of spent nuclear fuel at the Skull Valley site is the HI-STORM 100 cask system. The walls of the storage cask, about 30 inches thick, consist of concrete encased within a thin inner and outer steel layer; the spent nuclear fuel rods are stored upright in a multi-purpose canister that sits in the center of the storage cask. Singh/Soler Tstmy, Post Tr. 5750 at 7; Fig. 5.3.11, HI-STORM 100 overpack (Staff Exh. W).

162. A HI-STORM 100 cask loaded with spent nuclear fuel weighs about 180 tons, is about 20 feet tall and 11 feet in diameter. Loaded casks are moved from the Canister Transfer Building onto the storage pad by a cask transporter. Singh/Soler Tstmy, Post Tr. 5750 at 7. The CTB is founded on a 240 feet by 280 feet reinforced concrete foundation mat that is 5 feet thick; soil cement will extend around each side of the building, 240 feet to the east and west, and 280 feet to the north and south. SAR 2.6-108a-108b (PFS Exh. JJJ).

163. Fuel is shipped to PFS in a HI-STAR transportation cask, inside which is the multi-purpose canister containing spent nuclear fuel. Transfer of the canister containing the spent nuclear fuel from the HI-STAR to the HI-STORM casks takes place in the Canister Transfer Building. A transfer cask (HI-TRAC) is positioned above the HI-STAR cask; the canister is raised into the HI-TRAC, which is then re-positioned above the HI-STORM, and the canister placed therein. Lewis Tstmy, Post Tr. 8968 at 2-3.

Board Findings

164. The Board finds that the storage casks, the storage pads, the Canister Transfer Buildings and their foundations are structures, systems and components as that term is defined and used in 10 CFR Part 72.

Transfer Operations

165. PFS maintains that the entire transfer operation in the CTB for a single transfer, as listed in PFS SAR Table 5.1-1 (State Exh. 188; PFS Exh. ZZ)[32] can be completed in 20 hours. Lewis Tstmy, Post Tr. 8968 at 4. Mr. Lewis testified that SAR Table 5.1-1 is

[32]The times in each exhibit, State Exh. 188, Rev. 0 and PFS Exh. ZZ, Rev. 6, are identical.

based primarily on estimated on-site worker dose assessments from Table 10.3.3(a) in Holtec's generic safety analysis report (PFS Exh. AAA); Tr. (Lewis) at 8987-90; State Exh. 190. The Holtec worker dose assessment "addresses only the operators that perform work on or immediately adjacent to the cask." State Exh. 190 at 10.3-1. Mr. Lewis testified that the times in the Holtec Table (PFS Exh. AAA), were based on experience in loading Holtec casks. Tr. (Lewis) at 9059, 9078-80. The evidence does not support this claim. 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 multi-purpose canister occurred on July 6, 2000. Tr. (Lewis) at 8993, 9079; State Exhs. 191, 194. Further, the Board notes that Table 5.1-1, Rev. 0, is part of PFS's original 1997 license application. The Board finds that the table from which PFS estimates the canister transfer operations time is not derived from actual Holtec cask transfer operations.

166. The 20 hour transfer time listed on SAR Table 5.1-1 (Rev. 0) is an operational worker exposure time; the actual time of those operations would occur over a three day period. Tr. (Lewis) at 9039. Also, PFS will employ a three person crew to conduct a transfer operation and does not contemplate that its transfer crew will work night shifts. Tr. (Lewis) at 9038, 9073, 9077.

167. Mr. Lewis testified that from the time the MPC is in the transfer cask (*i.e.*, the MPC is not in a sealed cask) until it is placed in the storage cask (Table 5.1-1, Steps 17 to 23) is 2.8 hours. However, there is evidence in the record from the cask manufacturer that suggests this operation could take 8 to 12 hours. Tr. (Lewis) 9029; Singh/Soler Depo. Tr. excerpts (State Exh. 193). The continuous part of the transfer operation (Table 5.1-1, Steps

7 to 25) occurs over an estimated operational time of 9 hours. Mr. Lewis was unaware of any OSHA or NRC restrictions on maximum hours a worker could continuously work or the breaks required during that period.[33] Tr. (Lewis) at 9043-45. Mr. Lewis also acknowledged that there are no license conditions or commitments in the SAR that require PFS to complete Steps 7 to 25 continuously. Tr. (Lewis) at 9077. The Board notes that there is no regulatory prohibition on the MPC remaining for a long period, such as overnight, in the unsealed HI-TRAC transfer cask Tr. (Lewis) at 9077-78.

Board Findings

168. The Board finds that the transfer times listed in SAR Table 5.1-1 (Rev. 0) are operational times. The State has presented evidence of serious shortcomings in PFS's presentation of actual and operational transfer times. The Applicant has not responded with evidence that its time presentations are reliably based on experience or conservative assumptions. The State has met its burden of going forward to which PFS has not adequately responded.

Ground Motions at the PFS Site

169. As part of its original license application, PFS estimated ground motion at the site had a peak ground acceleration of 0.72g in the horizontal direction and 0.80g in the vertical direction using a deterministic seismic hazard analysis ("DSHA"). Bartlett/Ostadan Tstmy, Post Tr. 7268 at 4. PFS later revised and significantly increased the strong ground motion estimates for the DSHA, which now have peak ground acceleration (pga) values of

[33] Mr. Lewis conceded there will be a one hour lunch break thereby increasing his estimate to 10 hours. Tr. (Lewis) at 9043-45.

1.15g in the horizontal direction and 1.17g in the vertical direction. Id. For the 2,000-year return period event, PFS first estimated pga values of 0.53g (horizontal) and 0.52g (vertical), using a probabilistic seismic hazard analysis. Id. But, after a further seismic investigation, pga values were significantly increased to 0.711 g (horizontal) and 0.695 g (vertical), which are the latest peak ground accelerations for the design. Con-SER at 2-34.

Board Findings

170. The Board further finds that design basis ground motion for a 2,000 year mean return period event are 0.711 g in the horizontal direction and 0.695 g in the vertical direction; ground motions based on a deterministic analysis are 1.15 g (horizontal) and 1.17 g (vertical).

PFS's Seismic Design

171. The State maintains that the many disparate pieces of PFS'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. *See e.g.*, PFS Exh. 210; Tr (Soler) at 10609 ("[W]e were tasked ... to get forces from the casks on the pad and transmit them to ICEC. We were not party to the calculations being done by ICEC, nor were we party to the calculations being done by Stone & Webster in Boston. We wrote the reports... but we were not part to what use he [Wen Tseng] was going to make of those forces.") Emblematic of this is the lack of independent verification or checks and balances of the input parameters to the various design calculations; Tr. (Trudeau) at 6247-49; Tr. (Soler) at 10610; Tr. (Ostadan) at 7350. Many of the input parameters for the design calculations are derived from Holtec – the cask manufacturer who stands to gain millions of dollars from the PFS project and who

is providing technical assistance from its president and vice-president to PFS for the hearings as part of its sales package. Tr. (Ostadan) at 7350; Tr. (Singh/Soler) at 5915-20.

172. PFS'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. PFS Exh. 210, internal memo Trudeau to Macie, April 3, 1997; Tr. (Bartlett) at 10293-94. Soil cement was first introduced to stabilize the eolian silts in place and to provide a stable platform for the cask transporter. Tr. (Bartlett) at 10295. Only when there was a significant increase in ground motions, did PFS introduce using cement-treated soil as an mechanism for resist seismic loading. Tr. (Bartlett) at 10295.

173. Rather than bypass the weaker and more compressible zone of the upper Bonneville clays either by treating the clays or embedding the pads into deeper, stiffer, and stronger soil, PFS 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* Contention Part C, *supra*. PFS'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 Bonneville clays. Tr. (Barlett) at 10287.

174. The foundation design of the CTB changed in response to an estimated thirty-five percent increase in ground motions at the PFS site. Tr. (Bartlett) at 7313-14; Bartlett/Ostadan Tstmy (Dynamic Analysis), Post Tr. 7268 at 4; PFS Exh. VV at 5-6. Existing soils around the footprint of the CTB will be excavated to a depth of about five feet, mixed with cement and placed a distance of about 240 feet to 280 feet out from the basemat of the building. Ebbeson Tstmy, Post Tr. 6357 at 5; Tr. (Bartlett) at 7313. A

reinforced concrete key will also be constructed around the perimeter of the foundation mat. Ebbeson Tstmy, Post Tr. 6357 at 5.

175. The soils at the PFS site have limited capacity to carry loads and thus PFS 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. (Bartlett and Ostadan) at 7313-16. 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. (Ostadan) at 7316. If the passive resistance of soil cement is not used then the calculated factor of safety against sliding will be less than 1. Tr. (Ebbeson) at 10798; Bartlett/Ostadan Tstmy, Post Tr. 7268 at 4-5.

Board Finding

176. The Board finds that in its seismic design PFS is relying upon soil cement and cement-treated soil under and around the pads and soil cement around the foundation of the CTB to increase the capacity of the soft clays (*i.e.*, upper Bonneville clays). Referring to Contention, Part C, at this time, testing the properties and performance of these cement materials has not been conducted.

177. We pause here to reflect on the complexities in evaluating foundation design, especially under seismic condition. Unlike fabricated material such as concrete and steel where the boundary conditions in the design are reasonably well-defined, that is not the case with soils. Tr. (Bartlett) at 10300. Soils are naturally deposited materials that are heterogenous and isotropic, 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, there are huge uncertainties in soil properties that affect their strength and compressibility, both with

time and during an earthquake. Id. at 10301. The usually 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, it is only when there is sufficient precedence and actual experience, that geotechnical engineers have confidence in their models. Accordingly, there is 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 10301-02.

PFS's Seismic Design Calculations

178. Central to PFS's design calculations is Holtec cask stability analysis. The purpose of this design calculation was to analyze cask displacement as well as to determine dynamic forces. The Holtec design calculation is central because evaluating the adequacy of the foundation design is a function of the dynamic forces that will be imparted to them. In order to evaluate the response of the foundation or the soil cement to resist seismic loads and to evaluate the stability of the casks, it is critical to understand the seismic loads and the assumptions made in calculating the seismic loads. Bartlett/Ostadan Tstmy, Post Tr. Xx at 11. With no experience or test data to rely upon, the Holtec nonlinear analysis is the linchpin in PFS's package of design calculations for seismic stability of the casks, pads and foundations system.

179. Holtec's early decision to treat the pads as rigid for its cask stability analysis apparently came from Stone & Webster, the contractor responsible for the overall design of the ISFSI. Tr. (Trudeau) 6186-87. As inputs into the cask stability analysis, Holtec used earthquake time histories, and dynamic soil properties developed by Geomatrix. Tr.

(Ostadan) at 7570-71. Of particular note, the ground motion time histories developed by Geomatrix are in the free field, *i.e.*, away from the influence of the storage pads, the casks and the soil cement. Tr. (Ostadan) at 7513. As a consequently, the 0.7g peak vertical and horizontal ground acceleration estimated by Geomatrix does not include any effects on ground motion from the underlying soils or overlying structures. Those effects must be analyzed through soil structure interaction.

180. The outputs from Holtec's cask stability analysis included the seismic loads (*i.e.*, force time histories) from the casks and the pads. These output were used by Dr. Weng Tseng (ICEC) as an input into the structural design of the storage pads and in part by Mr. Paul Trudeau as an input into the seismic pad stability design calculations. Also, Dr. Youngs (Geomatrix) used the Holtec force time histories from the top of the cask as an input into his analysis of the effects of non-vertically propagating waves. However, there is no record in Holtec's calculation of computation of seismic load without cask sliding. Tr. (Ostadan) at 10291-92.

181. Holtec also conducted a cask tipover analysis from which it determined the maximum deceleration of a cask falling onto the pad is a limit of 45 g. As part of this analysis, Holtec placed constrains on the depth of cement-treated soil under the pads to a maximum of two feet and on the modulus of elasticity of that material to be no more than 75,000 psi. In addition, Holtec limited the stiffness of the concrete storage pad, thereby constraining the design of the pad to a maximum of three feet thick.

182. The soils characterization, testing and analysis results were used in Mr. Trudeau's seismic stability design calculations for the storage pads and for the CTB.and also

by Mr. Tseng in the structural design of the storage pads.

Sliding as a Design Concept and Base Isolation Systems

183. PFS relies entirely on cask sliding as a mechanism to reduce seismic loads. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 5. For example, in his review of the forces that Holtec has provided, Dr. Ostadan found at different times during the duration of shaking that there is separation in the contact points between the cask and the pad. Tr. at 10435-36. 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. (Ostadan) at 10291. As such, the forces transmitted to the pad and the underlying soils will be significantly greater if the casks are not allowed to slide. Tr. (Bartlett) at 10292.

184. Sliding of SSCs in earthquake resistant design is not common and is rarely used. Tr. (Ostadan) at 7345. One accepted exception called a base isolation system is engineered and designed to reduced seismic loads. Id. at 7345-46, 10290. Still, with a base isolation system the Uniform Building Code places requirements on the maximum reduction in seismic forces that can be used. Id. at 10290-91.

185. The Uniform Building Code recognizes the advantage of a base isolation system, but with no performance data from real earthquakes for base isolation systems, the UBC only allows 20% maximum credit for reduction in seismic load from these systems. Also, the UBC first requires test plans and laboratory testing, then computation of the seismic loads without the isolators. A reduction of 20 percent of the seismic load without the isolators is the maximum allowed, or in other words, 80 percent of seismic loading without isolators must be used. Id. at 10290.

186. The State claims 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. Id. at 7349-50, 10290.

Board Findings

187. During the hearings the Board used the analogy of a cliff and the distance from it as a way to conceptualize uncertainty and factors of safety. Tr. (Judge Lam) 6274. Here, the Board finds that the Applicant's reliance on a system that absorbs some of the seismic energy and reduces the seismic loads to the pads and foundation soils approaches the edge of that cliff. PFS has not proposed a conventional engineered base isolated system; instead it is willing to accept happenstance during an earthquake as to the movement of the casks. The Board, however, is not willing to make this leap. PFS has not shown that its seismic design and estimation of seismic loadings are suitable for the site conditions at the Skull Valley site.

Pad Flexibility/Rigidity

188. As a design concept for the storage pads, PFS has conflicting requirements. The storage pads need to be rigid enough to allow smooth sliding of the storage casks but somewhat flexible for cask drop or tipover. Further, the stiffness of the cement-treated soil directly under the pads cannot be too stiff because of Holtec's cask drop and tipover condition but must be stiff enough to provide resistance to pad sliding during an earthquake. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 11-12.

189. An initial premise underlying Holtec's design calculations is that the pad will

act as a rigid body. As stated in the original Holtec *Multi-Cask Seismic Response at the PFS ISFSI* (at 3-4), dated May 19, 1997: "the characteristics of the pad are based on the assumption that the 30′ by 64′ 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." *See* Tr. (Trudeau) 6186-87; State Exh. 170; PFS Exh. 85. While Mr. Trudeau admitted that he did not make this recommendation to Holtec, he acknowledged that Stone and Webster are the project engineers responsible for the design of the PFS facility. Id. Further, Dr. Tseng responsible for the structural design of the pads, did not complete his design calculation until October 1999; so he could not have given this material property of the pad to Holtec. 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. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 11. The assumption of pad rigidity has also guided Holtec's selection of soil springs and damping values. Tr. (Ostadan) at 7451; *see* Cask Stability *infra*.

Rigidity for purposes of dynamic loading

190. Holtec has calculated damping that is associated with a rigid pad and ICEC has used those soil springs and damping values in its design calculation. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 14. The Applicant has not shown the pads are rigid, yet it takes full credit for radiation damping. Tr. (Ostadan) at 7459. If the pads are, in fact, rigid there would be significant soil structure interaction effects with the soils playing a major role in dissipating energy through radiation damping. Id. at 7451, 7455-57.

191. Radiation damping occurs under earthquake conditions from the engagement

of the structure with the soil. If the foundation is rigid, it resists engagement with the soil whereas a flexible foundation moves back and forth as the soils move. Id. With a rigid foundation there is a greater difference between its mass and stiffness than those of the soil (especially as compared with a flexible foundation), and because the rigid foundation tries to negate the motion of the ground, there are significant soil structure interaction effects. In this case, the soil acts as a continuing medium and has the beneficial effect of dissipating a great deal of seismic energy. Tr. (Ostadan) at 7449-50; 7457. Material damping can range from 5 to 7 percent whereas radiation damping can be as much as 20 to 30 percent. Id. at 7458. If the pad is flexible then there is much less radiation damping than if it is rigid. Id. at 7456.

192. ICEC was never asked to determine the appropriate damping under rigid or flexible conditions. Id. at 7467. Instead, ICEC was given the dynamic forces that came out of Holtec's nonlinear time history analysis. Id. 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 concrete properties. Id. at 7466, 7471. 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.

Flexibility for purposes of cask drop and tip over

193. We turn now from flexibility of the pad relating to dynamic analysis to flexibility or deformation of the pad from physical cask impact. Physical impact on the pad

could be from cask tipover or cask drop. The conditions under which the cask contacts the pad are complex and dynamic. For example, if the cask drops vertically, the whole area of the bottom of the cask would impact the pad, but the cask could also drop and make initial contact at one point. Id. at 7450. The pad, cement-treated soil, and soil all contribute to the stiffness or flexibility that would engage in this drop/tipover condition.

194. The contact condition in Holtec's analytical calculation for cask tip over and drop requires that the pad and underlying cement-treated soil be somewhat flexible to be able to absorb energy from cask impact. Tr. (Ostadan) at 7449. Here, 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. (Ostadan) at 7422-23; 7450-54; 10398-99.

195. In response to Dr. Ostadan's claim that the ICEC calculation (Table D-1(d)), using the forces obtained from Holtec, evidenced pad flexibility because it showed vertical deformation or displacement occurring, Dr. Tseng testifies that the maximum displacement in Table D-1(d) is 3/8th of an inch. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 12-13; Tseng/Youngs Tstmy, Post Tr. 5529 at p. 24; ICEC Calc. Table D-1(d) (PFS Exh. 85 at 234). Dr. Tseng further asserted, without any basis for his conclusion, that the maximum deviation of local displacement from a rigid body is 1/8 of an inch. Tseng/Youngs Tstmy, Post Tr. 5529 at 24; Tr. (Ostadan) at 7463. Dr. Ostadan countered this proposition; what is important is not necessarily the amplitude of displacement but the movement of the different points on the pad with respect to each other. Tr. (Ostadan) at 7460, 7465. The larger the relative movement over the pad the less damping. Less damping will result in an increase in the

calculated seismic loadings. Id. at 7460, 7469-70. 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, an expert in soil structure interaction, 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. (Ostadan) at 7471.

Board Finding

196. PFS attempts to present the case that the pad can be rigid for some purposes but flexible for others. The question for the Board to address 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.

197. The Board finds that PFS has had ample opportunity to put this matter to rest. It could have conducted a half day analysis with SASSI and determined the appropriate dynamic properties of the pad for the PFS site compared to a rigid pad. Instead, we have this tortured post-hoc justification of why the pad can be somewhat flexible under dynamic cask drop and tip over yet still retain its rigid properties when it comes to cask sliding and computation of soil springs and radiation damping. PFS is asking the Board to accept that the same pad-foundation system is flexible enough for the cask drop and tip over constraint and, given that condition, allow Holtec and ICEC to claim full credit for radiation damping and assume a smooth surface for cask sliding. Also, as discussed in Soil Cement *supra*, whether PFS can meet the constraints placed on cement-treated soil and make the pad and

the underlying cement-treated soil somewhat flexible is still a big unknown that, had PFS already conducted testing, would not be at issue in this proceeding. PFS is asking too much of this Board to agree to these potentially conflicting requirements, especially when its entire design concept is not based on any experimental, test or performance data but relies entirely on a nonlinear analysis with assumed inputs. The Board finds that PFS had the ability to satisfy the claims advanced by the State. It has not, however, credibly or consistently demonstrated the dynamic properties and behavior of the storage pad.

Storage Pad Foundation System and Soil Structure Interaction Effects.

198. Geomatrix performed a soil column analysis to obtain the strain compatible soil properties in the free field using a common industry program, SHAKE. Tr. (Ostadan) at 7570-71. Soil material is highly nonlinear during earthquake shaking; 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, 7571. Although the SHAKE soil properties are used in soil structure interaction analysis, the SHAKE analysis cannot in any way be considered a soil structure interaction analysis. Tr. 7513.

199. 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 soil structure interaction effects. Tr. 7515, 7570. Soil-structure interaction is a very complex analysis. If one were analyzing a rock site where the rock does not deform, there would be virtually no soil structure interaction effects. But on a relatively soft and layered soil site like the PFS site, one needs to account for deformation of the soil; the additional amplification of the seismic

motion that could be caused by the soil; take credit for radiation damping in the soil; and then realistically predict the seismic loads and seismic response of the structure. Tr. (Ostadan) at 10312-13. 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. Id. at 10418.

200. Dr. Ostadan has written and reviewed numerous soil structure interaction reports. Tr. 7516. 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 soil structure interaction effects. What is the frequency response of the pad system? How does it change? If soil property is changed from lower bound to best estimate to upper bound, is there any rocking? Is there any torsional response on the pad? Holtec is focused only on the displacement of the cask and these questions remain unanswered. Tr. (Ostadan) at 7517; (Soler) at 10610.

201. Holtec estimated the force time histories from the casks on the pad and transmitted them to ICEC; Holtec was not asked to – nor did they – provide the acceleration of the pad to ICEC or to Stone and Webster. Tr. (Soler) at 10609; Tr. (Ostadan) 10338. Holtec's focus was on "what the casks do." Tr. (Soler) at 10610.

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

203. Seismic loads from the cask are exerted on the pad and additional seismic loads are due to the mass of the pad, itself. Tr. (Ostadan) 7533. 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. First, ICEC, in the design of the pads, and SWEC, in its pad sliding analysis, had to guess at the inertial load of the pad itself. Second, SWEC also had to guess at the loads coming from the cask. Id. at 7529-36.

204. ICEC was not asked to perform a complete soil structure interaction analysis or to analyze damping. Tr. (Ostadan) 7466-67. 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, such to estimate the amount of steel reinforcement. Id. at 7467. At best, ICEC's analysis constitutes about ten to twenty percent of what is needed for a complete soil structure interaction analysis. Id.

205. To understand the inertial load of the pad requires knowledge of the acceleration of the pad. Tr. (Ostadan) 10338. Further, from Holtec's analysis it is unknown how much shear force is going to be generated based on the pad itself. The dynamic loads from Holtec do not include acceleration of the pad or shear forces. Tr. (Soler) 10609-10.

206. Turning to the dynamic forces for pad stability, instead of obtaining the acceleration of the pad from Holtec in the cask stability design calculations, PFS Exh. UU, Mr. Trudeau assumed a number — peak ground acceleration (0.7g) – for a design input into the pad sliding analysis. Tr. (Ostadan) at 7624-25; Bartlett/Ostadan Tstmy, Post Tr. 7268 at 18. Peak ground acceleration ("pga") is the ground motion in the free field and does not account for soil structure interaction effects. Use of pga for the seismic loads for the pads has nothing to do with the response of the pad. Tr. (Ostadan) 7480-81. The pad, the soil and the foundation have their own natural frequency; it ranges from 5 to 11 hz. Id. at 7481. Using peak ground acceleration as the input motion to estimate the seismic loads for the pad

is only appropriate for rock sites and that is not the case at the PFS site. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 12-13; Tr. (Ostadan) at 7480-81.

207. Mr. Trudeau attempts to justify his use of peak ground acceleration by assuming that there is a tremendous amount of radiation damping – 48 to 52 percent – and with these high damping values, the design motion in the free field (0.7g) is a fairly close fit. Trudeau (Dynamic Analysis) Tstmy, Post Tr. 6135 at 14-16; Tr. (Trudeau) at 6199-6200; Tr. (Ostadan) at 7623. However, Dr. Ostadan, an expert in soil structure interaction – compared to Mr. Trudeau who has no expertise whatsoever in soil structure interaction[34] – found Mr. Trudeau's assertion of high damping values "unusual" for such a foundation system. Tr. (Ostadan) at 7623. Moreover, the "simple" calculation that Mr. Trudeau performed to arrive at about fifty percent damping was not the method he used in his CTB stability calculations. Tr. (Trudeau) at 6200-01. While 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. (Ostadan) at 7530.

208. For the sliding and stability analysis of the CTB, PFS Exh. UU, Stone & Webster obtained the structural response and dynamic response from other calculations and used them as inputs into the sliding and overturning analysis of the CTB. Tr. (Ostadan) at 7530, 7551. The amplified response of the CTB mat is in excess of 1 g. Tr. 7545. Dr. Ostadan found this significant because the foundation pressure for the mat is about the same for the pads as it is for the CTB (2ksf). Tr. at 7545. Accordingly, there is an anomaly

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[34]Tr. (Trudeau) at 6163.

between the more than 1 g response in the CTB and 0.7 g response in the pads. Tr (Bartlett/Ostadan) at 7544-45.

209. Because the Luk Report is about the only place in the record that comes close to discussing pad accelerations, Dr. Ostadan resorted to Figures 17 and 20b of the Luk Report as an indicator of pad acceleration. Tr. (Ostadan) 10339, 10342-44. Dr. Ostadan admitted that Dr. Luk omitted proportional damping from his model and that possibly his analysis tends to over-predict high frequency response, which may be responsible for the 3 g acceleration in Fig. 17; Figure 20b, however, has accelerations beyond 10 g. Id. at 10342-43. Even though Dr. Luk omitted proportional damping and Figures 20b and 17 apply to one node, one still can glean high pad acceleration from those figures – whether it is 2 g or 3 g, the pad acceleration is still very high. Id. at 10344. 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 10343.

210. The importance of the dispute on the response of the pad is that the Applicant has only a 1.27 factor of safety against sliding of the storage pads. In that calculation, Mr. Trudeau did not take into consideration the potential amplification of the acceleration of the pads in the horizontal direction. Tr. (Trudeau) at 6201. Foundation sliding is a major concern to the State - expecting the foundation to remain stable under the large accelerations predicted for the PFS site is "very optimistic expectation, to say the least." Tr. (FO) at 10340.

Board Finding

211. There is ample evidence to suggest that the acceleration of the pads may be

greater than that estimated by PFS. The Board finds that Holtec did not provide the acceleration of the pad to ICEC or to Stone and Webster. The Board further finds that neither ICEC in the design of the pads nor Stone and Webster in the pad stability analysis design calculations used the correct input parameters for the dynamic response of the pad. Finally, the Board finds that the 1.27 factor of safety for the pad sliding analysis is not based on correct input assumptions.

Pad-to-Pad Interaction

212. We briefly addressed pad-to-pad interaction in Contention Part C, Soil Cement, but deferred our ruling until Part D. As described in Soil Cement, *supra*, the casks, pads, soil cement and soils have very different masses, and the inertial effect – *i.e.*, fundamental frequencies at which these different masses want to vibrate – introduces compression and tension into the system and creates out-of-phase motion among the various masses.

213. Dr. Bartlett's and Dr. Ostadan's pre-filed testimony (answer 36) provides a detailed description of their concerns with pad-to-pad interaction. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 15-18. They testified 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, Stone and Webster assumed that during an earthquake the different masses of the entire system would be in-phase. Id. at 15-16.

214. Next, the State witnesses observed that the storage pads and surrounding soil cement are not structurally tied together, such as with reinforcing rebar and noted that in Dr.

Tseng's deposition testimony, he admitted that the pads and soil cement would not act as an integrated structure. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 16.

215. The crux of the State's testimony is that during the cycling of earthquake forces, there will be separation between the soil cement and the storage pads; the soil cement and pads will not act as an integrated unit; and the difference in modulus between the very stiff soil cement and the relative soft upper 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 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 Tstmy, Post Tr. 7268 at 17.

216. The soil structure interaction effects will cause the pads, which are only five feet apart from each other, to move differently from the free field motion of the soils. Tr. (Ostadan) at 10380; Bartlett/Ostadan Tstmy, Post Tr. 7268 at 17. 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 PFS's pad sliding analysis of the pads and stability analysis of the casks. Tr. (Ostadan) at 10380-81; Bartlett/Ostadan Tstmy, Post Tr. 7268 at 17.

217. 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. (Ostadan) at 10382; PFS Exh. 225, at 28. The resulting force transfer from that analysis is reported to be 1,900 kips. Id. This is a large force and has not been accounted for in the stability analysis of the pads. Tr. (Ostadan) at 10382. The State has

serious concerns about the already slim margin in PFS's pad seismic stability calculation, PFS Exh. UU, where the base case has only a 27 percent calculated margin of safety.

218. PFS attempts to present the pad-to-pad interaction effect as unrealistic because, according to PFS, the pads will not slide. Trudeau Rebuttal (Dynamic Analysis), Post Tr. 11275 at 6-7. Mr Trudeau assumes that there will be no sliding because PFS has calculated a 27 percent 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. This does not overcome the State's concern,s because as Dr. Bartlett testified, pad-to-pad interaction can occur without sliding. Bartlett Partial Surrebuttal (Dynamic Analysis), Post Tr. 11306 at 4; Bartlett/Ostadan Tstmy, Post Tr. 7268 at 17. The upper Bonneville clay underlying the pads is a relatively deformable body compared to the much stiffer soil cement plug between the pads. Bartlett Partial Surrebuttal (Dynamic Analysis), Post Tr. 11306 at 4. During earthquake cycling there will be soil structure interaction 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 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.; Tr. (Ostadan) at 7520-21. The Holtec report, discussed in the preceding paragraph, PFS Exh. 225, briefly analyzed a simple two pad system in the longitudinal direction and showed a significant transfer of lateral forces even without initial of pad sliding. Bartlett Partial Surrebuttal (Dynamic Analysis), Post Tr. 11306 at 4. Certainly, pad sliding will cause more severe pad-to-pad interaction effects than calculated by Holtec in Applicant's Exh. 225. Id.

Furthermore, the Holtec calculation did not include the effects of multiple pad interactions.

Board Findings

219. The Board finds that it is unrealistic to assume that a column or group of pads will act as an integrated unit. Moreover, PFS is relying on the bonding properties of the soils-cement-treated soil-pad-soil cement system, for which PFS has no testing or performance data, to make its case that the pads will not slide. Further, the Holtec analysis for a simple two pad system demonstrates that there can be significant forces transferred from pad-to-pad. The compounding effect in a row of pads can be very severe. In sum, the Board finds that PFS analysis is deficient because the seismic sliding stability calculations, PFS Exh. UU, stability analysis of the casks, and the design of soil cement and cement-treated soil do not take into consideration the horizontal load transfer from pad-to-pad interaction.

Stability Design Calculations

220. PFS witness Mr. Paul Trudeau is responsible for one of the major design calculations for the PFS facility, *Stability Analysis of the Cask Storage Pads*, Cal. No. G(B) 04, Rev. 9 (PFS Exh. UU). Tr. (Trudeau) at 6159-60. Mr. Trudeau is also responsible for a similar calculation, Cal. G(B) 13, Rev. 6, for the stability analysis of the CTB. *See* PFS Exh. VV. Mr. Trudeau confirmed that in PFS Safety Analysis Report (Rev. 21) at 2.6-45, PFS has committed meeting NUREG 75/078, Section 3.8.5 Foundations, II.5, "*Structural Acceptance Criteria*" and to the later superseded guidance, NUREG-0800, Section 3.8.5. Tr. (Trudeau) at 6169. These guidelines set a minimum factor of safety against sliding, overturning and bearing capacity failure of 1.1 or 10 percent. The base case in Cal. No. G(B) 04, Rev. 9 has a factor of safety and sliding of 1.27. Tr. (Trudeau) at 6164.

221. The Stone and Webster calculation for sliding, overturning and bearing capacity failure of the storage pads and the CTB, PFS Exhs. UU and VV, are based on a 2,000-year design basis earthquake. These two calculation are part of the design calculation in support of PFS's license application and are part of the basis on which the Staff has evaluated the facility and issued the Consolidated SER. Tr. (Ofoegbu) at 6529, 6580-81, 6597-98; Con-SER, Ch. 5.

<u>The Storage Pads</u>

222. As described in the preceding sections, the design calculation for the stability of the storage pads uses peak ground acceleration as an incorrect input parameter for the dynamic response of the pad and does not account for the horizontal load path from pad-to-pad interaction.

223. The other major concerns with the design calculation for the stability of the storage pads are whether there is any evidence to suggest that there are design calculations for a 10,000-year earthquake and whether there is a deficiency in the overturning analysis methodology.

224. Mr. Trudeau's pre-filed testimony alludes to the "conservatism" in PFS's design as being able to withstand ground acceleration greater than those for a 2,000-year DBE. *See* Trudeau Tstmy, Post Tr. 6135 at 5-13. Mr. Trudeau also testified that in his testimony he has reported results from some analyses for a 10,000-year mean return period event (MRP). Tr. (Trudeau) at 6166. Ground motions for a 10,000-year MRP would be greater than 1 g. <u>Id.</u> at 6343.

225. In response to questions by the Board, Trudeau admitted that his design

calculations for sliding, overturning and bearing capacity failure "do not address the 10,000 year earthquake" and that he would need to do more than is presented in his testimony to support a 10,000-year DBE analysis. Tr. (Trudeau) 6348.

226. Another other major concern in Cal. No. G(B) 04, Rev. 9, PFS Exh. UU, is Mr. Trudeau's analysis of pad overturning for the 2,000-year return period event. For overturning of the pads, the factor of safety against overturning is the sum of the resisting moment divided by the driving moment. Cal. No. G(B) 04, Rev. 9 at 13. In calculating the driving moment, Mr. Trudeau testified that he used the worst case for overturning by assuming that cask sliding had initiated when he calculated the horizontal driving force. Tr. (Trudeau) at 6243. He calculated a horizontal driving force of 696 kips, applied that force at the base of the cask and used a lever arm of 3 feet (the thickness of the pad) to arrive at an overturning moment of 2088 kip-feet. Id. at 6243-44. Mr. Trudeau agreed that for the case he analyzed, sliding is initiated after the horizontal driving force reaches 696 kips based on a coefficient of sliding friction of 0.8. Id. at 6245. When the horizontal driving force is slightly less than 696 kips and cask sliding has not yet initiated, then the lever arm is in the center of mass of the pads and the casks, *i.e.*, approximately 13 feet (½ of the 3 foot thick pad and 20 foot high cask). For the case where the cask has not yet initiated sliding, the driving moment is 8970 kip-feet; therefore, the factor of safety against overturning is significantly less than calculated on page 13 of Cal. No. G(B) 04, Rev. 9. Tr. (Trudeau) at 6245-47.

227. There are additional uncertainties in the overturning calculation. Mr. Trudeau did not validate many of the assumption that went into his analysis. He accepted from Holtec that sliding would be initiated at 0.8 times the normal stress at the base of the casks. Tr.

(Trudeau) at 6248. Further, Mr. Trudeau did not account for conditions that may interfere with initiating sliding as used in his calculation, such as deflections in the pad from long term settlement, concentration of stresses from partial cask uplift, or whether the casks do not slide at all, or have resistance to sliding due to cold bonding. Id. at 6250-51. Of critical importance to his analysis, Mr. Trudeau did not consider the potential amplification of the vertical or horizontal acceleration of the pad from soil structure interaction effects. Rather Mr. Trudeau used peak horizontal and vertical acceleration of 0.711g and 0.695g, respectively. Tr. (Trudeau) at 6251-53.

Board Findings

228. The Board finds that PFS has committed to follow NUREG-75/078, Section 3.8.5 Foundations, II.5, "*Structural Acceptance Criteria*" as updated by NUREG-0800, Section 3.8.5.

229. The Board finds that the only design calculations in the record for sliding, overturning, and bearing capacity failure of the storage pads and the CTB are PFS Exhs. UU and VV; they are based on a 2,000-year design basis earthquake; form the basis for licensing the PFS facility; and are relied upon by the Staff in the issuance of its Consolidated Safety Evaluation Report.

230. The Board finds there are no design calculations for a 10,000-year MRP for sliding, overturning, or bearing capacity failure.

231. Finally, the Board finds that the overturning calculations in Cal. No. G(B) 04, Rev. 9, PFS Exh. UU, are deficient and cannot be relied upon.

Pad settlement

232. Pad settlement was not considered in PFS's structural design of the pads or in Holtec's cask sliding stability analysis. Tr. (Bartlett) at 10332. Settlement from differential cask loading could cause dishing or tilting of the pads. Consequently, such an effect impacts Holtec's cask stability analysis because Holtec assumed a perfectly horizontal planer surface in its cask sliding and stability analyses. Tr. (Bartlett) at 10332-33.

233. PFS's estimations of pad settlement have spiraled downward from an initial five inches of settlement, to two inches to finally, in rebuttal testimony, half an inch. Internal Memo from Macie to Trudeau/Georges, dated April 2, 1997 (PFS Exh. 211); SAR (Rev. 22) at 2.6-50 (State Exh. 168); Trudeau Rebuttal (Section D), Post Tr.11275 at 4.

234. In 1997 PFS predicted total differential pad settlement of 5 inches in one month under full loads. State Exh. 211; Tr. (Bartlett) at 10334. 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). State Exh. 211; Tr. (Bartlett) at 10334-35. There is no known plan for PFS to do any pre-loading. Tr. (Bart;ett) at 10335. While the 1997 memo is a historical document, it does point out the long standing concerns about the settlement of the pads and its potential impact to the structural adequacy of the pads.

235. There are different types of settlement. Elastic settlement occurs as the pads are loaded and will be evident in days; consolidation settlement for the upper Bonneville clays occurs over about a two year period; and long term settlement or creep occurs over the design life of the facility. Tr. (Bartlett) 7495-96. Further, the science of predicting settlement is not so precise that one can estimate settlement in tenths of inches. Id. at 7496. Further, over the 51 acre pad emplacement area there is soil variability, and this variability will have an

effect on settlement of many of the pads. *See* Soils *supra*; Tr. (Bartlett) at 7497-7500.

236. The State agrees that two to three inches of settlement is a reasonable estimate of total settlement, but it points out that in geotechnical practice a few inches of settlement is a significant number in foundation design. Tr. (Ostadan) at 7501. 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, 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. (Ostadan) at 7763-64.

237. In rebuttal testimony Mr. Trudeau claims that rather than the settlement figure that PFS used to support its application – 1.7 inches – he suggests that settlement could be as small as half an inch. Trudeau Rebuttal (Section D), Post Tr. 11275 at 4. The Board is not inclined to accept this unsupported supposition. The Board takes note of Dr. Bartlett's experience and expertise in analyzing the upper Bonneville clays in contrast to Mr. Trudeau's lack of such experience. Also, the Board accepts Dr. Bartlett's testimony that settlement cannot be estimated in tenths of inches; therefore, it takes the two inches of settlement as reasonable. Further, as Dr. Ostadan testified, all 500 pads at the PFS will not be loaded at the same time. The sequence of loading of the casks on the pads, such as placing two casks or four casks on one side, may cause the pad to develop a concave or dishing shape, with the middle deforming more in the sides then at the two ends. Tr. (Ostadan) at 7486. Further, loading of several pads may compound the effect of settlement in one area causing the next rows of pads in a group to tilt. Id. at 7764.

238. During the hearing, Holtec used various coefficients of friction to predict pad

sliding. Even though Holtec used various coefficients of friction, it kept the coefficient of friction constant during a particular run. A pad with a dish shape will have an effect on cask sliding depending on whether the cask is climbing up the slope (harder) or down the slope (easier). Tr. (Ostadan) at 7501-02. Accordingly, there is a need to vary the co-efficient of friction depending on which direction the casks are sliding (up or down). Tr. (Bartlett) at 7504-06. In such a case, there is a bias in a particular direction so Holtec's run with a random coefficient of friction will not account for this bias. Tr. SB 7505-06. In sum, Holtec's analysis does not account for a non-planer pad surface.

Board Findings

239. The Board finds that while pad settlement relates to static loads those loads cannot be divorced from the analysis of dynamic loadings. *See* Tr. (Ostadan) 7482-83. To find otherwise would lead to a deficiency in the loads to be analyzed in the dynamic case and, as the State has repeatedly pointed out, the slim margins in PFS design make it imperative that we scrutinize all uncertainties in PFS's analyses. *See* Tr. at 7766. Further, the Board rejects PFS's unsupported claim of half an inch of pad settlement, especially when viewed from the perspective of the pads being placed on top of a relatively soft clay. Further, the Board finds that differential loading could cause a dishing shape in some of the pads and this in turn impacts Holtec's cask sliding analysis.

240. There is insufficient evidence in the record to show that all the pads will settle uniformly. Failure by PFS to support this proposition invalidates assumptions in Holtec's pad sliding analysis which in turn has the potential to underestimate cask movement atop the pads and the inertial forces transmitted to the pads and the foundation.