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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)	)	December 7, 2001

STATE OF UTAH'S RESPONSE AND OPPOSITION TO APPLICANT'S MOTION FOR SUMMARY DISPOSITION OF PART B OF UTAH CONTENTION L

The State of Utah files its response and opposition to Applicant's Motion for Summary Disposition of Contention Utah L, Part B, pursuant to 10 CFR § 2.749 and the Board's November 21, 2001 Order (Ruling on Motion to Supplement or to Extend Time for Filing Summary Disposition Response). The State's Opposition is supported by a Statement of Disputed and Relevant Material Facts; the Declaration of Dr. Walter J. Arabasz (Exhibit 1), Joint Declaration of Dr. Farhang Ostadan, Dr. Steven F. Bartlett and Dr. Mohsin R. Kahn ("Utah Joint Dec.") (Exhibit 2), and Declaration of Dr. Marvin Resnikoff (Exhibit 3).

BACKGROUND

In response to the Board's certified question in LBP-01-03, 53 NRC 84 (2001), the Commission in CLI-00-12, 53 NRC 459 (June 14, 2001), directed the Board to adjudicate the State's contention challenging the Staff's proposal to grant PFS an exemption from the seismic hazard analysis requirements of 10 CFR § 72.102(b) and (f). The contention, as admitted by the Board, and designated as Utah L, Part B, is set forth in the Board's Order dated June 15, 2001. *See also* PFS's Summary Disposition Motion at 1-3.

Template = SECY-041

SECY-02

The State filed Utah L, Part B in reaction to PFS's request to use a probabilistic seismic hazard analysis ("PSHA") methodology rather than use a deterministic seismic hazard analysis ("DSHA") as required by the regulations that to this day are still the current licensing requirement for ISFSIs. *See* 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") but later amended the request to allow use of a 2,000 year DBE. In LBP-01-03, the Licensing Board found that at the behest of the Staff, when PFS decided to change its request from a 1,000 year DBE to a 2,000 year DBE, it did not give any detailed reasons for the change. Rather, the only specific enumeration of reasons for the change was to the found in the Staff's Safety Evaluation Report ("SER"). LBP-01-03, slip op. at 19. Under this set of circumstances, the Board admitted the State's contention, which, in part, challenged the Staff's failure to conform to SECY-98-126, a rulemaking plan to amend 10 CFR § 72.102, which would allow only a 10,000 year DBE for NRC Category 1 structures systems and components ("SSC").[1] The contention also challenges the Staff's use of Department of Energy ("DOE") Standard 1020-94, reliance on a previous grant of an exemption to the Idaho Engineering and Environmental Laboratory ("INEEL") and lack of conservatism in PFS's design.

The contention, admitted as Contention Utah L, Part B, was filed on November 9, 2000.[2] Since that time, new events have unfolded. But DOE has issued a new draft

[1]SECY-98-126 refers to SSCs "whose failure would result in greater accident consequences" and requires those SSCs to be designed to "Frequency-Category 2 design basis events," i.e. a PSHA with a 10,000 year return period. SECY-98-126 at 4-5. In this Response, the State uses the term "NRC Category 1 SSCs" to denote those SSCs that must be designed to a 10,000 DBE.

[2] The State challenged PFS's exemption request first on April 30, 1999, then on January 26, 2000; the Board

Standard 1020-01 in which it still inextricably couples the probability of the occurrence of an earthquake with the performance goals of SSCs. DOE has moved away from a 2,000 year DBE and now intends to use a 2,500-year DBE for Performance Category 3 ("PC3") facilities.[3] In addition, the Staff has recommended to the Commission a modified rulemaking plan, SECY-01-0178, which would allow the use of a 2,000 year DBE, regardless of site location.[4]

LEGAL STANDARD

A party is entitled to summary disposition if "there is no genuine issue as to any material fact" and the party "is entitled to a decision as a matter of law." 10 CFR § 2.749(d). The burden of proving entitlement to summary disposition is on the movant and "the evidence submitted must be construed in favor of the party in opposition thereto, who receives the benefit of any favorable inferences that can be drawn."[5] If there is any possibility that a litigable issue of fact exists or any doubt as to whether the parties should be permitted or required to proceed further, the motion must be denied.[6] Expert testimony and opinion contained in affidavits in support of summary disposition must meet two legal requirements. First, the affidavit must contain a demonstration that the affiant is an expert,

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ruled the Staff had not actually taken a final position and thus the State's challenges were too early.

[3] PFS states ISFSIs are categorized as PC3. *See* PFS Motion at 7.

[4] The Commission gave negative consent to the modified plan but added the proviso that in proposing the rule, the Staff "should solicit comment on a range of probability of exceedance levels from 5.0E-04 through 1.0E-04" (*i.e.*, DBE ranging from 2,000 years to 10,000 years). Staff Requirements Memo, dated November 19, 2001, attached hereto as Exhibit 4.

[5] Sequoyah Fuels Corp. and General Atomics Corp. (Gore, Oklahoma Site Decontamination and Decommissioning Funding), LBP-94-17, 39 NRC 359, 361, *aff'd* CLI-94-11, 40 NRC 55 (1994).

[6] General Electric Co. (GE Morris Operation Spent Fuel Storage Facility), LBP-82-14, 15 NRC 530, 532 (1982).

qualified to express expert opinions on matters contained in his or her affidavit.[7] Second, the affidavit must contain analysis, facts and reasons supporting the expert's opinion.[8]

ARGUMENT

Utah L, Part B centers on the choice of the appropriate site specific design basis earthquake. Such a choice, however, is inextricably linked to the performance capabilities of structures, systems and components (SSCs) important to safety at the PFS facility[9] because it is the failure probabilities of these SSCs which are the primary concern. Another way of saying this is: given the probability of occurrence of the design earthquake ground motions, what is the probability of failure of the SSCs? If there is a relatively high probability of the design earthquake ground motion occurring (such as with the choice of a 1,000 or 2,000 year return period earthquake), then the failure probabilities of the SSCs *given these motions* must be small in order to reduce the annual probability of SSC failure to a suitably conservative level. Thus, there is a co-joining of the probability of strong ground motion associated with an earthquake and the probabilities of failure of SSCs. PFS wants to establish the design basis ground motions at an annual exceedance probability of 5×10^{-4} (*i.e.*, a 2000 year return period); however, it has attempted through unrealistic assumptions, omissions and gross generalizations to show that the failure probability of certain SSCs at PFS is much lower and

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[7] Public Serv. Co. of New Hampshire (Seabrook Station, Units 1 and 2), LBP-88-31, 28 NRC 652, 1988 WL 236205, 4 (1988), *aff'd*, ALAB-909, 29 NRC 1 (1989).

[8] Carolina Power & Light Co. and North Carolina Eastern Municipal Power Agency (Shearon Harris Nuclear Power Plant, Units 1 and 2), LBP-84-7, 19 NRC 432, 447 (1984); Mid-State Fertilizer Co. v. Exchange Nat'l Bank, 877 F.2d 1333, 1339 (7th Cir. 1989).

[9] The SSCs of concern for seismic analysis are Category I SSCs; at PFS these are: the CTB and certain components therein, the storage pads and the HI-STORM cask system. SAR at 3.4-3 to -4, Revs. 17, 9.

meets the "target performance goal" specified in DOE Standard 1020.[10] In both the 1994 and the (draft) 2001 versions of this standard, the performance goal for Performance Category 3 facilities is "set at an annual probability of exceedance of about 10^{-4} of damage beyond which hazardous material confinement and safety-related functions are impaired." DOE-STD-1020-94 at B-8. DOE-STD-1020-2001 at B-7.

PFS's Motion fails on a number of counts.[11] First, witnesses in support of PFS's Motion give opinions for which they have no expertise or make gross unsupported overstatements. Second, NRC as an agency is still grappling with the determination of the mean annual exceedance probability of an earthquake at an ISFSI; the 1998 Rulemaking Plan still provides a valid consideration in establishing the standard for the PFS site. Third, the Staff's grant of an exemption to an ISFSI at INEEL does not form a precedent for PFS's exemption request; there are significant differences in the facts underlying each request. Fourth, in discussing DOE Standard 1020, PFS has not followed its design philosophy - such as demonstrating that DOE's performance goals for SSCs are met. Fifth, in an attempt to claim that SSCs will survive a 2,000-year DBE as well as a 10,000-year DBE, PFS has made gross generalizations, and arrived at "conservatism" by making gross unsupported conclusions. Finally, the State disputes PFS's bold and unsupported claims that if all 4,000 casks tip over there will be no violation of NRC dose requirements.

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[10] PFS has not conducted a site specific performance of significant SSCs at the PFS site. For one thing, PFS has omitted to include in its discussion of Category I SSCs the performance of the foundation of the Canister Transfer Building ("CTB") and the foundation of the storage pads. This is a glaring omission. For example, an evaluation of whether the crane in the CTB will perform under seismic loads is pointless if the CTB foundation fails under those seismic loads

[11] PFS appears to have abandoned the Staff's analysis in the SER. To the extent that the Staff's justification in the SER is still relevant, it does not substantiate granting the exemption. *See* Arabasz Dec.

I. PFS's Witnesses Give Unsupported Opinions.

PFS's witnesses have testified in areas for which that are not qualified or without analysis, facts and reasons to support their expert opinion. *See* nn 5,6. In particular, Dr. Cornell gives unsupported technical opinions and improper opinions on questions of law.

The Board should disregard certain statements in Dr. Cornell's Declaration. First, in paragraphs 28 and 29 of his Declaration, Dr. Cornell makes several statements about the storage casks that PFS proposes to use. It is unclear from his statements whether he is purporting to opine as an expert on storage casks or whether he is relying on the opinions of others, which he is only paraphrasing. To the extent that he is purporting to give expert opinion on storage casks, those opinions must be disregarded by the Board because Dr. Cornell has no expertise on the subject. To the extent that Dr. Cornell is relying on the opinions of others, the Board should note that Dr. Cornell's opinions are only as good as the opinions on which he is relying, and should not rely on Dr. Cornell's paraphrasing.

Second, in paragraph 28, for example, Dr. Cornell states that transfer casks, "if damaged in an earthquake, can be repaired or replaced without adverse safety consequences." Dr. Cornell does not explain how he reached that conclusion, which is a subject on which he has no expertise. Moreover, he does not indicate whether he is relying for that conclusion on the Declarations of Bruce Ebbeson and Holtec that are mentioned earlier in the paragraph.

Third, in paragraph 29, Dr. Cornell states that "even if [the casks] should tip over, the conservatisms in the design of casks and canisters will prevent the release of radioactivity." Again, Dr. Cornell does not explain how he reached that conclusion, which is

a subject on which he has no expertise. Moreover, he does not indicate whether he is relying for that conclusion on "analyses performed by the cask manufacturer" mentioned earlier in the paragraph, nor does he cite the Board to those analyses.

Dr. Cornell also gives improper opinions on questions of law. Dr. Cornell is not a lawyer. In paragraph 32, however, he gives his opinion that certain legal grounds asserted by the State in support of its contention "are no longer valid."[12] Dr. Cornell's opinion on the legal merits of the State's contention must be disregarded by the Board. Similarly, in paragraph 45, Dr. Cornell opines on the "intent" of the Commission and the Staff in granting a 1998 exemption "to DOE for the Idaho National Engineering and Environmental Laboratory ("INEEL") ISFSI for the Three Mile Island, Unit 2 . . . facility." Dr. Cornell states that "the Staff was advising the Commission that the granting of the exemption for INEEL would be relied upon as precedent for other exemption applications" and that "the decision by the Staff and the Commission was not intended to be" interpreted in the manner put forth by the State. Dr. Cornell's opinions on the Staff's and the Commission's intent and how their decisions are to be interpreted must be disregarded.[13]

Many of the conclusions drawn by PFS's witnesses in their declarations are unsupported[14] and should carry no weight with the Board.

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[12]In paragraph 32, Dr. Cornell states that "the State's challenge" in Basis 1 to Part B of Utah L "appears to be legal rather than technical." He then explains why, in his view, "the grounds asserted by the State in this objection to the granting of the exemption to PFS are no longer valid."

[13] In accordance with the Board's November 27, 2001 (Memorandum and Order granting the State's Motion to Compel) the State reserves the right, following the completion of its discovery, to supplement its discussion of Dr. Cornell's opinions flowing from his involvement in NRC's rulemaking efforts to amend Part 72 to allow use of PSHA for ISFSI seismic design.

[14] Declarations by other PFS witnesses also suffer similar impediments as those of Dr Cornell. For example,

II. The 1998 Rulemaking Plan, SECY-98-128, Still Provides a Valid Consideration of the Appropriately Conservative Design Earthquake for the PFS Site.

First it must be emphasized that the current regulations require license applicants for ISFSI sites located west of the Rocky Mountain Front to conduct a deterministic seismic hazard analysis in accordance with 10 CFR part 100 App. A. 10 CFR § 72.102(f)(1). To obtain an exemption from the Commission's considered judgment of licensing requirements, PFS has the burden of showing that its exemption request to use probabilistic seismic hazard analysis and a design earthquake with a 2,000-year return period is "authorized by law and will not endanger life or property or the common defense and security and [is] otherwise in the public interest." 10 CFR § 72.7. Further, current use of PSHA for assessing seismic hazards does not mean that a deterministic analysis is outmoded or ineffective.[15]

The Commission gave negative approval to SECY-98-126,[16] on April 24, 1998. The 1998 Rulemaking Plan proposed the use of a PSHA methodology and, for Category 1 SSCs, a mean annual exceedance probability of 10^{-4} (*i.e.*, a 10,000-year DBE). Since that time the Staff has modified the rulemaking plan to allow for a 2,000-year DBE. The Commission in granting negative approval to the Staff on the modified plan directed the Staff when proposing the rule to specifically solicit comments on a range of probability of exceedance levels equivalent to a 2,000 year DBE through a 10,000 year DBE. *See* Exh. 4. Moreover,

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PFS relies on the Ebbeson Declaration, ¶¶ 8-11 for the time the crane will be in operational mode in the CBT. The operational time of the crane in the CTB is dependent on how long it takes for transfer operations to be completed in the CTB. Mr. Ebbeson relies completely on SAR, Rev. 6, Table 5-.1-1 He has not personal knowledge whether those times are reasonable. See Exhibit 6, Ebbeson Tr. at 30-34.

[15]For example, the retrofit of the San Francisco Bay Rapid Transit System will be built to a deterministic design basis earthquake. *See* Utah Joint Dec. ¶ 28. *See also* Arabasz Dec. 9.

[16]Rulemaking Plan: Geological and Seismological Characteristics for Siting and Design of Dry Cask Independent Spent Fuel Storage Installation, 10 CFR Part 72.

the Commission's statement that "Staff should undertake further analysis to support a specific proposal" suggests that more work must be done to support the rulemaking. Therefore, as an agency, NRC's position on the question central to generic rulemaking (*i.e.*, determination of the mean annual exceedance probability of an earthquake at a proposed ISFSI) is still an open issue. Given the Staff's Requirements Memo relating to the new plan, the Board should reject PFS's argument that the modified rulemaking plan "moots" Basis 1 in Utah L, Part B. Motion at 10. Also, NRC's continuing development of generic ISFSI seismic siting criteria should not occasion the State to be saddled with a moving target in this site-specific licensing proceeding.

The choice of a design basis earthquake at the PFS site is not a generic issue. PFS either cannot or is unwilling to meet existing regulations by which seismicity at proposed ISFSI sites West of the Rock Mountain Front must be evaluated – an issue that affects the licensability of the ISFSI. An exemption should not defeat the underlying purpose of the rule. When promulgating the existing rule, the Commission found that developing a site specific design earthquake for western sites as required by 10 CFR Part 100, App. A "is considered necessary and appropriate for the protection of an ISFSI which could contain a large inventory of spent fuel."[17] The fuel inventory at PFS is unprecedented in quantity (*i.e.*, 40,000 MTU). Moreover, the design basis earthquake is the building block for other analyses. The peak ground acceleration derived from the PSHA is an input to such calculations as the site specific analyses of the stability of casks, storage pads, CTB and other

[17] Final Rule, *Licensing Requirements for the Storage of Spent Fuel in an Independent Spent Fuel Storage Installation*, 45 Fed. Reg. 74,693, 74,697 (1980) (*codified at* 10 C.F.R. Part 72).

SSCs. At issue is an exemption from existing regulations, and therefore the determination of any allowable deviation from the existing regulatory standard should err on the use of a conservative standard. In an attempt to justify use of a 2,000-year DBE, PFS has slashed all conservatism built into existing NRC standards. *See* Section V below. Such an approach defeats the purpose of the rule which is to provide protections for an ISFSI containing a large inventory of fuel. When PFS applied for an exemption from the seismic regulations in April 1999, the 1998 Rulemaking Plan evinced the policy considerations of the Commission. Now the current agency position on this issue is still open. In sum, the 1998 Rulemaking Plan allowing the use of a 10,000 year return period is a valid consideration of the appropriately conservative design earthquake for the PFS site.

III. The INEEL Exemption Does Not Form a Precedent for Establishing a 2,000-Year Return Period Earthquake at PFS Because Facts and Conditions Underlying the INEEL Exemption Differ from Those at PFS.

PFS makes a claim that is contrary to the fundamental precepts of administrative law: "the NRC Staff and the Commission expected (and intended) that [the INEEL exemption] would serve as a precedent towards the granting of similar exemptions in the future." Motion at 14. PFS continues to cling to the argument that the Staff can dispense with the Administrative Procedure Act, in particular 5 U.S.C. § 551(5), when changing a promulgated rule. *See* Utah Reply Brief to the Commission, dated March 12, 2001 at 7-8. PFS's attempt to transform the grant of an exemption to INEEL into retrospective rulemaking should be soundly rejected.[18]

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[18]*See eg.*, Columbia Falls Aluminum Co. v. E.P.A., 139 F.3d 914, 919 (D.C. Cir. 1998) (once rule is final, agency can amend it only through new rulemaking); Alaska Professional Hunters Ass'n, Inc. v. F.A.A., 177 F.3d 1030, 1033-1034 (D.C. Cir. 1999).

Another reason why INEEL does not serve as a precedent for the PFS exemption is that there are significant differences between the INEEL and PFS sites. *See* Exhibit 5, INEEL/PFS comparison chart. INEEL is a federal reservation covering 892 square miles in southeastern Idaho. Id. The ISFSI at INEEL, located about 15-20 kilometers from the INEEL facility boundary, will store Three Mile Island Unit-2 rubblized reactor core debris, totaling roughly 307,000 pounds of material and stored in a total of 29 NUHOMS horizontal storage modules. Id. By contrast, PFS is located on approximately 820 acres on the northwestern edge of the Skull Valley Reservation next to privately owned ranching property where it will store up to 4,000 HI-STORM casks containing 40,000 MTU of fuel with an average burnup of 40,000 MWD/MTU. SER at 7-4, 7-6.

The site-specific seismicity at INEEL is also significantly less than at PFS, as illustrated by the peak ground acceleration (pga) at the two sites. The pga for a 2,000-year earthquake at INEEL is 0.3g whereas at PFS is it about 0.7g *See* Exh. 5. Furthermore, the DBE at INEEL did not use the pga from a 2,000-year earthquake as PFS will do for its facility. Rather, the DBE at INEEL uses a pga of 0.36g which translates into a return period earthquake of about 3,000 to 4,000 years. Arabasz Tr. at 42-43; Arabasz Dec ¶ 24. Therefore, under the exemption, INEEL designed its ISFSI to a 3,000 to 4,000-year DBE and not to a 2,000-year DBE as PFS intends to do. *See e.g.*, Motion at 3.

Another significant difference between the two sites is the type of storage system each will use. INEEL uses the NUHOMS system – a horizontal storage module (HSM) concrete bunker whereas PFS intends to use the unanchored vertical HI-STORM storage system. Exh. 5. The unanchored HI-STORM cask will likely experience excessive

dislocation and tip over at ground accelerations of 0.7g (Utah Joint Dec. ¶ 55); there is little or no possibility that the horizontal NUHOMS storage vaults at INEEL will tip over.

The Board should reject PFS's claim that it is unnecessary to litigate over the precedential value of the INEEL exemption. Motion at 15. PFS's argument is hardly persuasive when you consider that the Staff did not rely on the "precedential value" of the April 1998 INEEL exemption when it proposed a 10,000-year DBE in the June 1998 rulemaking plan, SECY-98-126. Arabasz Dec. ¶ 26. The INEEL exemption cannot provide a justification for a site specific 2,000-year DBE at the PFS site.

IV. PFS and the Staff Have Misused DOE Standard 1020 by Decoupling the Annual Hazard Probability from the Target Performance Goal.

PFS claims that the Staff's reliance on DOE Standard 1020 in supporting approval of PFS's exemption is appropriate. Motion at 13. Moreover, PFS claims that it has used an approach analogous to DOE Standard 1020 by relying on conservatism built into NRC standard review plans that would meet or better the DOE-1020 performance goal for PC3 facilities. Id. Lacking from the Staff's approach and PFS's justification in its Motion is the fundamental coupling between the annual probability of exceeding the design ground motions and the performance for PC3 SSCs "set at an annual probability of exceedance of about 10^{-4} of damage beyond which hazardous material confinement and safety-related functions are impaired." DOE -STD-1020-94, at B-8. *See also* Utah's Response to PFS's 7th Set of Discovery at 15-16 (Sept. 28, 2001). Significantly, DOE Standard 1020 has recently undergone revision that would require the use of a 2,500-year DBE – not a 2,000-year DBE. *See* Utah Joint Dec. ¶ 31.

In attempting to cobble together a justification, PFS has picked only those parts of DOE Standard 1020 that are to its liking (*i.e.*, a 2,000-year DBE) and ignored the rest such as quantifying whether SSCs at PFS can meet DOE's performance goals. By omitting this quantification, which would required engineering analyses taking into account the site-specific hazard curves, seismic fragility curves for the important SSCs, and the uncertainties from these fragility curves, PFS has divorced the DBE from the performance goals. *See* Utah Joint Dec. ¶¶ 22, 35-49. PFS neither follows DOE-STD-1020 nor does it develop a supportable "analogous" approach. Id. As discussed in Section V, there are serious disconnects with PFS's "analogous" approach.

V. There is Lack of Conservatism in PFS's Design.

In its attempt to justify use of a 2,000-DBE, PFS has opened the door to whether or not the site specific design at PFS will be conservative notwithstanding PFS's contradictory claim that Utah L, Part B is not about the adequacy of PFS's design to satisfy the DBE.[19]

Under 10 CFR § 2.749(b) the State is required to refute PFS's claims with specificity. PFS's boast that it can meet or better DOE's performance goals can be refuted by showing that many conservatisms generally inherent in SSCs do not apply at the PFS site. Moreover, if PFS is going to claim credit for certain aspects of design criteria, it cannot ignore other design elements, such as the foundations of the storage pads and the CTB. In sum, the assumptions underlying the design of the PFS facility and quantitative analyses thereof go to

[19]On the one hand PFS states that Utah L Part B "does not concern whether the design of the PFS [facility] to satisfy design basis earthquake requirements is adequate." Motion at 12. On the other hand PFS spends the remainder of its Motion discussing the design performance of SSCs at PFS being able to satisfy a 2,000-year DBE and even a 10,000-year DBE. PFS cannot have it both ways.

the heart of whether there is conservatism in PFS's design. Contention Utah QQ is directly on point in refuting any conservatism in the calculations and analyses PFS has used in its SAR to support its seismic design. In attempting to justify the 2,000-year DBE, PFS has opened the door to the issues raised in Utah QQ and it cannot now complain that the State is trying to "back door" those issues. PFS Motion at 11.

Central to PFS's justification for the use of a 2,000-year DBE is PFS's analogy to the performance goals of SSCs and risk reduction factors in DOE Standard 1020 and NUREG/CR-6728[20] and reliance on conservatism built into NRC standard review plans. Motion at 6-9. The PFS SSCs cannot be assumed to have the risk reduction factors specified in DOE Standard 1020 or NUREG/CR 6728. First, PFS's design for a site with intense strong ground motions is unprecedented and defies general engineering principles: casks are unanchored and allowed to slide in a "controlled" manner. Utah Joint Dec. ¶ 34. Second, PFS has not conducted the full panoply of analyses required by DOE-STD-1020 or NUREG 6738. Utah Joint Dec. ¶¶ 35-49. For example, as required by DOE-STD-1020, PFS has not developed a fragility curve for each SSC at the PFS site, *i.e.*, the unacceptable performance of an SSC as a function of the amplitude of strong ground motion. Thus, PFS has no way of knowing the probability of unacceptable performance of each SSC. Third, DOE-STD-1020 discusses the use of risk reduction ratios but recognizes that specific acceptance criteria for foundations have not been developed. Id. ¶ 41. Nevertheless, for cases where acceptance criteria have not been developed, the intent of DOE-STD-1020 can

[20] NUREG/CR-6728, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines* (October 2001).

be met by showing there should be less than 10 percent probability of unacceptable performance at input ground motion defined by 1.5 times a seismic scale factor times the DBE ground motion. DOE-STD-1020 at 2-23 to -24; Utah Joint Dec. ¶ 41. This PFS has failed to do.[21] Likewise, in PFS's effort to resort to NUREG/CR-6728 to find conservatism of design at its site, PFS has failed to assess the slope of the seismic hazard curve; the design seismic capacity of the SSC and its uncertainty; and uncertainty in the seismic response as NUREG requires. Id. ¶¶ 45-49. Finally, with respect to reliance in SRPs, NRC standard review plans for nuclear power plants do not address the seismic performance requirements that apply at the PFS.[22] PFS has not shown an equivalence of conservatism in ISFSI SRPs. Utah Joint Dec. ¶ 53.

PFS's attempts to shore up its "analogous" approach by claiming that Chapter 7 of NUREG/CR-6738 provides quantitative findings that components designed to NRC SRP standards have risk reduction ratios of "5 to 20 or greater." Motion at 8; Cornell Dec. ¶ 25. The fragility curves used in Chapter 7 are obtained from Kennedy & Short (1994). Utah Joint Dec. ¶ 56. When Kennedy and Short published the fragility curves they could not have included unanchored dry storage casks and, furthermore, those fragility curves do not apply to SSC foundations. Id. ¶ 56-57.

There is a significant probability of failure of SSCs at PFS. NUREG 1536, at 3-6, states: "the applicant must analyze the cask to show that it will not tip over or drop in its

[21]Whether the foundations of the pads and CTB at PFS meet a factor of safety against sliding of 1.1 as asserted in Utah QQ, signifies that PFS could not meet the test described in DOE-STD-1020. Utah Joint Dec. ¶¶ 78-91

[22]NUREG 1536, *Standard Review Plan for Dry cask Storage Systems* applies to HI-STORM 100; NUREG 1567, *Standard Review Plan for Spent Fuel Dry Storage Facilities* applies to CTB design. Utah Joint Dec. ¶ 53.

storage condition as a result of a credible natural phenomenon event." The State has conducted an independent analysis of Holtec's cask stability reports ("Altran Report"). *See* Att. F of Utah Joint Dec. [23] The Altran Report concludes for ground motion at the PFS site from a 2,000-year earthquake, the HI-STORM 100 cask horizontal displacement could be as high as 372.76 inches (x direction), 229.65 inches (y direction), and 27.24 inches in the vertical direction. Further, the results of the Altran study show that the casks will tip over if subjected to ground accelerations from a 2,000-year earthquake. Utah Joint Dec. ¶ 74. Importantly, the results from Holtec non-linear cask stability reports are unconservative and non-unique and have not been validated with any experimental data, such as shake table test data. Utah Joint Dec. ¶¶ 71-74. In addition, actual earthquake experience has shown that objects with masses similar to a loaded HI-STORM 100 cask have moved under ground motions similar to those predicated at the PFS site. *See* Utah Joint Dec. ¶ 75 and Att. G. It is not surprising, therefore, that casks at the PFS site will uplift, slide and tip over.

VI. PFS's Inflated Claims of Lack of Dose Exposure from a Severe Seismic Event Are Speculative, Unsupported and Erroneous.

PFS makes the bold claim that if one cask or all of them tipped over in a 10,000-year return period earthquake, "doses at the boundary remain essentially unchanged." Motion at 17. PFS even asserts that if a cask "remains in a horizontal position, the doses at the site boundary are less than if the cask had remained upright."[24] Id. These bold claims by PFS are not supported by quantified analyses and are outside the bounds of the Holtec Certificate

[23]The methodology and conclusions drawn from the Report are described in the Joint Dec. ¶¶ 64-74.

[24]This begs the question of why the casks are not stored horizontally rather than in an upright position.

of Compliance (CoC) for the HI-STORM cask system.[25] Moreover, Holtec's hypothetical site specific cask tip over report for the PFS site is flawed, especially as it relates to the rate of impact of the fuel assemblies on the pad, the degree of flattening and concrete cracking of the casks, loss of shielding through cask heat-up and failure to compute resultant dose effects from these conditions.

The facts and conditions supporting the HI-STORM CoC differ significantly from those at the PFS site. The analysis in the HI-STORM CoC to determine the maximum zero point acceleration that will not cause incipient tipping is bounded by an acceleration of 0.45g horizontal and 0.16g vertical; at the PFS site 2,000-year motions are 0.711g horizontal and 0.695g vertical. Resnikoff Dec. ¶¶ 12, 13. The Holtec CoC used a dose exposure time of 8,760 hour, per year to compute dose exposure at the fence post whereas PFS used only 2,000 hours per year; had PFS used 8,760 hours per year it would not comply with10 CFR § 72.104(a) dose limits.[26] PFS's site specific analyses to support use of the Holtec casks come up short. In particular, PFS's failure to quantify the consequences from ground motions for a 2,000-year, 10,000-year or deterministic earthquake is fatal to its analysis.[27] These ground motions are so far outside the bounds of the Holtec CoC that PFS cannot ignore this fact while as the same time claiming that conservatism built into the fabrication of the HI-

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[25]Because PFS is outside the bounds of the HI-STORM CoC, it must conduct a site specific analysis to determine whether the performance of the HI-STORM cask at the PFS site adequately protects health and safety. Resnikoff Dec. ¶¶ 9, 10.

[26] PFS's computed 5.85 rem per year rises to 25.6 rem per year when a duration time of 8,750 hours per year is used. Resnikoff Dec. ¶ 15. This dose rate, even in the absence of an earthquake, is in excess of the allowable 25 rem/year specified in 10 CFR § 72.104(a). Resnikoff Dec. ¶ 15.

[27] A further shortcoming is that Holtec calculated the dose consequence in a non-mechanistic single cask tip over for the PFS site whereas at PFS the entire field of casks could tip over. Resnikoff Dec.¶ 12.

STORM cask system justifies the use of a 2,000-year DBE.

PFS admits that cask tip over could cause localized damage. PFS Joint Dec. ¶ 25. Contrary to the Holtec HI-STORM TSAR and without any quantified analyses, PFS's claim that there will be no noticeable dose increase at the boundary is mere speculation. Resnikoff Dec. ¶ 18. For support of its claim, PFS rests on the general proposition that the HI-STORM Multi Purpose Canister (MPC) "has a very substantial margin built into it." PFS Joint Dec ¶ 20; Motion at 17. Whether or not the confinement boundary in the MPC would be breached in a 2,000- or 10,000-year DBE at the PFS site cannot rest on such a generalized assertion, especially when the "substantial margin" assertion does not hold true at the PFS site.

There are a number of defects in PFS's claims relating to dose exposure. First, PFS's claim that the tip over analysis in the SAR showed that the maximum fuel deceleration is below the 45g limit that bounds the design basis limits for fuel is premised on an incorrect assumption. PFS Joint Dec. ¶ 18. In determining whether fuel assemblies would be damaged in a tip over event at the PFS site, Holtec incorrectly assumed the initial angular velocity would be zero. Resnikoff Dec. ¶ 19. At an initial angular velocity greater than zero, the fuel assemblies will be damaged when they hit the pad at greater than 45g, the cask will flatten more than contemplated by PFS, and concrete in the casks will crack. Id. ¶¶ 19, 20. Unresolved safety concerns remain because PFS has not quantified the degree of concrete flattening or cracking and the concomitant reduction of gamma and neutron shielding from the consequences of cask tip over. Id. ¶ 21. Even if the casks do not tip over they will certainly collide and PFS has not evaluated the damage nor calculated the dose increase from

cask collisions. Id. ¶ 27.

Second, the gamma dose at the fence and gamma and neutron dose to workers will increase from reduced shielding when HI-STORM casks are tipped over and the bottom of the casks face the fence post. For example, reduced shielding from the bottom of the cask[28] will cause the dose rate due to gamma rays at the site boundary to increase between 1.8 to 18 (2,000 hour year) or between 7.7 and 77 (8,760 hour year) than that calculated by PFS. Resnikoff Dec. ¶ 24.

Third, PFS's ability to upright overturned casks to ensure that the cask continues to comply with the CoC is suspect. The Holtec calculations supporting its CoC show that after 33 hours of 100% air inlet blockage, the concrete temperature will exceed the short-term limit of 350°F specified in the CoC for the HI-STORM 100 cask. The CoC temperature limit is established to ensure the continued effectiveness of the neutron shielding by ensuring the water does not evaporate from the concrete, reducing the amount of hydrogen available for neutron scattering and capture. For cask tip over at the PFS site, PFS has not considered or quantified the effects of reduced shielding from failure to upright and re-position the 175 ton casks that are about 20 feet high and 11 feet in diameter. Without hydrogen, the neutron dose rate to workers could be up to 57.3 times greater than Holtec's value of 1.88 mrem/hour and would result in a worker dose of approximately 108 m/rem/hour. This would exceed the worker dose limit in 10 CFR § 20.1201, Subpart C. Resnikoff Dec ¶ 26.

Finally, there is the potential the canister contents in HI-TRAC will be exposed to

[28]Compared with the sides of the cask, the bottom of the HI-STORM casks provides reduced shielding and indirect gamma rays and neutrons will stream though the cask bottom. Resnikoff Dec. ¶ 24.

the environment. The Holtec's cask drop calculations[29] for the HI-TRAC cask assume that the cask will drop vertically from a height of 25 feet onto a concrete base, but the HI-TRAC is more likely to drop on its edge. Such an unanalyzed "corner drop" could impair the welds and expose the canister contents to the external environment. Resnikoff Dec ¶ 29.

At PFS the HI-STORM cask will be operated under conditions that are outside the parameters analyzed in the CoC and would lead to doses at the boundary that exceed regulatory limits as well as jeopardize the health and safety of on-site workers. PFS's attempt to grasp at generic conservatism built into the HI-STORM cask system and make inflated unquantified claims of lack of dose exposure from a severe seismic event do not meet the burden imposed by 10 CFR § 72.7 in granting an exemption from duly enacted regulations.

CONCLUSION

The State urges the Board not to let PFS's claimed "conservative" design to go unchallenged. Casks that uplift 27 inches and slide sideways up to 30 feet when subjected to strong ground motions cannot constitute a conservative design. For the reasons stated above, PFS has failed to show that its requested design basis ground motion will not endanger life or property or is otherwise in the public interest as required by 10 CFR § 72.7 and the State requests the Board to set this matter for hearing.

DATED this 7th day of December, 2001.

Respectfully submitted,

Denise Chancellor, Fred G Nelson, Laura Lockhart
Assistant Attorneys General

[29] *Evaluation of the Confinement Integrity of a Loaded Holtec MPC Under a Postulated Drop Event*, dated November 30, 2000.

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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)	)	December 7, 2001

STATE OF UTAH'S STATEMENT OF DISPUTED AND RELEVANT MATERIAL FACTS

In support of its Response and Opposition to Applicant's Motion for Summary Disposition of Utah Contention L, Part B, the State submits this Statement of Disputed and Relevant Material Facts.

1. On April 2, 1999, PFS submitted to NRC an exemption request from 10 CFR § 72.102, which requires a deterministic seismic hazard analysis ("DSHA") for seismic hazard assessments, to allow the use of probabalistic seismic hazard analysis ("PSHA") methodology and a 1,000-year design basis earthquake ("DBE").

2. In response to NRC comments, on August 6, 1999 PFS revised its exemption request from a 1,000-year DBE to a 2,000-year DBE. PFS Commitment Resolution Letter # 14.

3. The Staff's accepted PFS's use of a PSHA with a 2,000-year return period and in the SER justified its decision based on the following:

   a. The Staff concluded that the mean annual probability of exceedance for the PFS facility would be less[1] than 10^{-4} based on its position that a design ground motion for a Safe Shutdown Earthquake at the PFS site, which had a median reference probability of exceedance of 10^{-5}, would be the same as a design ground motion with a mean annual probability of exceedance of 10^{-4}. SER at 2-41 to 42; Arabasz Dec. ¶ 13.

[1]The SER actually states the the mean annual probability of exceedance would the "greater than" but it is assumed the Staff mean "less than."

b. DOE-STD-1020-94 for performance category 3 facilities designed for ground motion that has a mean recurrence interval of 2,000 years. SER at 2-42

c. The Staff's grant of an exemption to the Idaho NEEL for the TMI-2 ISFSI. Id.

4. The State disputes PFS Material Facts ¶ 19 and also ¶¶ 13-18 to the extent that they support ¶ 19. It is not the State's arguments that the median estimate should be used ""in lieu of the mean estimate for the design of nuclear power plants, and similarly for ISFSIs . . ."; rather, it is the flawed logic of the Staff. *See* Arabasz Dec. ¶¶ 13-16.

5. The State disputes PFS Material Facts ¶¶ 13, 14, and 15. The average mean SSE for nuclear power plants in the Eastern and Central U.S. has not "been calculated to be 1×10^{-4}"; rather, "[t]he mean annual probability of exceeding the SSE . . . is on the average about 10^{-4}." Cornell Dec. ¶ 38

6. Current seismic hazard maps such as those published by U.S. Geological Survey and the National Earthquake Hazard Reduction Program (NEHRP) have also adopted a 2,500-year motion for design use. Utah Joint Dec. ¶ 31.

7. Design codes such as those adopted or considered by the International Building Code and American Association of State Highway and Transportation Officials require a 2,500-year motion for design Id.

8. The State disputes the following statement in Cornell Dec. ¶ 49:

> [V]irtually all areas of public safety hazards are measured as annual probabilities (or frequencies) of occurrence, regardless of the length of the activity in question, the exposure time, the estimated facility life, or the licensing duration [Ref. 12 (Paté-Cornell paper)].

See Arabasz Dec. ¶¶ 30-37.

9. The Utah Department of Transportation currently requires all interstate highway bridges to be designed to the equivalent to an average return period of 2,500 years. Id. ¶ 32.

10. DOE Standard 1020-01 requires a 2,500-year return period earthquake for PC 3 facilities. Utah Joint Dec. ¶ 31.

11. The Rulemaking Plan, SECY-98-128, in effect at the time PFS submitted its

original and revised exemption requests, proposed a PSHA with a 10,000-year earthquake.

12. NRC has modified its plan to amend 10 CFR § 72.102 (SECY-98-128) with SECY-01-0178. The modified plan proposes to use a 2,000-year DBE but the Commission will require comments on the propsed rule on a range of DBE from between 2,000 years to 10,000 years. *See* Exhibit 4 to State's Response. There is still an open question of what allowable generic DBE will be for ISFSIs when 10 CFR § 72.102 is formally amended. Id. *See also* Arabasz Dec. ¶¶ 11-12; 29.

13. In 1998 NRC granted INEEL (TMI 2 ISFSI) an exemption from § 72.102. The effective DBE at INEEL ISFSI site is about 3,000 to 4,000 years. Arabasz Dec. ¶ 22.

14. There are significant differences between the facts and conditions at the INEEL ISFSI site from those at the proposed PFS Skull Valley site. *See* Exhibit 5 to State's Response; Arabasz Dec. ¶¶ 22-25.

15. To the extent that PFS Material Facts ¶¶ 5, 6 and 9 suggest that DSHA is no longer used or a valid methodology, the State disputes ¶¶ 5, 6 and 9.

a. DSHA is used for critical facilities such as the retrofit of the Bay Area Rapid Transit System and in Utah for seismic evaluation of State regulated dams. Joint Dec. ¶¶ 28 & 29.

b. DSHA establishes a benchmark to which results of any PSHA can correctly be compared to evaluate the conservatism of the PSHA results. Arabasz Dec. ¶ 9.

16. The State disputes PFS Material Fact ¶ 8; the use of a PSHA is violative of the current licensing requirements in 10 CFR § 72.102, which mandate the use DSHA methodology.

17. The State disputes PFS Material Fact ¶ 10; whether or not an ISFSI or a nuclear power plant is vulnerable to earthquake-initiated accidents depends upon the propability of an earthquake occurring and the probability of failure of the SSCs; the probability of an earthquake-initiated accident at a specific site, such as the PFS site, would be the same for an ISFSI or nuclear power plant ("NPP"). *See e.g.*, Utah Joint Dec ¶ 23.

18. The State disputes PFS Material Fact ¶ 11 to the extent it suggests that there is a de-coupling in the choice of the appropriate design basis earthquake from the performance of SSCs. Utah Joint Dec. ¶ 22.

19. The State disputes PFS Material Fact ¶ 12. The State also disputes Material Fact ¶ 20 as it relates to the factors referenced in PFS Material Fact ¶ 12 to determine

seismic safety. There are factors other than those listed in ¶ 12 relevant to determining the likelihood of seismic failure of a facility or structure due to an earthquake event. Other factors include establishment of fragility curves and determination of unacceptable performance of SSCs for all levels of strong ground motion. Utah Joint Dec. ¶¶ 37, 39, 45, 46, 47, 48; *see also* Arabasz Dec. ¶ 19.

20. Fragility curves are a probabilistic method used to determine the probability of failure of an SSC. A fragility curve expresses the expected damage or unacceptable performance of an SSC as a function of the amplitude of strong ground motion. Once a fragility curve has been established for a particular SSC, the probability of unacceptable performance can be calculated for all levels of strong ground motion, even for levels beyond those incurred by the DBE. Id.

21. PFS has not produced any fragility curves for the HI-STORM 100 casks, CTB and its foundation system. Utah Joint Dec. ¶¶ 43, 44.

22. Current applicable codes and standards do not address many of the specific design issues, such as those raised by the State regarding the seismic stability of the casks and pads. The are no provisions in codes or standards for the design of unanchored casks resting atop pads subject to high levels of strong ground motion that are supposed to slide in a controlled manner without excessive sliding or tipping. Id. ¶¶ 34, 52, 56, 57.

23. For cases where no deterministic codes or standards have been developed, such as SSC foundations that may fail in overturning and sliding, DOE-STD-1020 gives an acceptance criterion for seismic stability of SSCs. This acceptance criterion is that there should be a less than 10 percent probability of unacceptable performance for a scaled input DBE. Id. ¶ 41.

24. The PFS facility, including SSCs, and the calculations and analyses PFS has used in its SAR to support an ISFSI license, cannot be analogized to the risk reduction factors in DOE-STD-1020 or NUREG/CR 6728 because PFS has only chosen those parts of DOE-STD-1020 and NUREG/CR-6728 that may support its position; it has not conducted the full panoply of analyses required by those documents. Id. ¶ 23.

25. To the extent that PFS Material Facts ¶¶ 20 through 28 differ from or are inconsistent with Utah Joint Dec. ¶¶ 34 through 44, the State disputes PFS Material Facts ¶¶ 20 through 28.

26. The State disputes PFS Material Fact ¶ 23 to the extent that it implies that PFS has used DOE-STD-1021 to classify the performance goals of SSCs at the PFS facility. Joint Dec. ¶ 36.

27. The State disputes PFS Material Fact ¶ 24; PFS has not completed an analysis

for the performance goals of SSCs at the PFS site in accordance with DOE-STD-1020. Id.

28. The State disputes PFS Material Fact ¶ 25. PFS has neither followed DOE-STD-1020 nor NUREG/CR 6728 and SRPs for nuclear power plants are not applicable to the PFS ISFSI. *See, e.g.* Utah Joint Dec. ¶¶ 22, 23.

29. The State disputes PFS Material Fact ¶ 26. DOE is likely to adopt DOE-STD-1020-01. Utah Joint Dec ¶ 31. The risk reduction factors for DOE-STD-1020-01 are 4 and the mean probability of exceedance of the DBE is 4 x 10^{-4} for PC3 SSCs. Id. ¶ 36.

30. The State disputes PFS Material Fact ¶¶ 29, 31 and 35.

a. NRC standard review plans (SRPs) for nuclear power plants are not the SRPs applicable to ISFSI; any conservatism in SRPs for NPPs are not comparable to SRPs for ISFSIs. Utah Joint Dec. ¶¶ 53-55.

b. The canister transfer building ("CTB") is not designed to SRPs governing NPPs; it must be designed according to NUREG-1567, Standard Review Plan for Spent Fuel Dry Storage Facilities. Id. ¶ 53.

c. The HI-STORM 100 cask system that PFS will use is not designed to SRPs governing NPPs; it must be designed to NUREG-1536 Standard Review Plan for Dry Cask Storage Systems. Id.

d. NUREG/CR-6728 uses fragility curves obtained from *Basis for Seismic Provisions of DOE-STD-1020*, R.P Kennedy and S.A. Short (1994). NUREG/CR-6728 at 7-5. The Kennedy and Short fragility curves relied upon in NUREG/CR-6728 could not have included unanchored dry storage casks. NUREG/CR-6728 cannot be used to find conservatism in the HI-STORM 100 cask design for the PFS site. Id. ¶ 56

31. The State disputes PFS Material Fact ¶ 31.

a. PFS design requires unanchored casks, founded on a shallow pad foundation and buttressed by cement-treated soil, to slide in a controlled manner without tipping when subject to high levels of strong ground motion. NRC SRPs do not address the seismic performance requirements that apply to this situation at the PFS site. Utah Joint Dec. ¶¶ 34, 52, 56, 57.

b. The PFS design and analysis are not conservative. Utah Joint Dec. ¶¶ 34-49, 55, 60, 76, 78-89.

c. The cask will slide excessively and tipover under the 2,000-year Design Basis

Earthquake. Utah Joint Dec. ¶¶ 74-75.

d. PFS has not calculated the fragility of the safety related SSCs and the uncertainty in the fragility and the slope of the hazard curve due to variations in material properties and nonlinearity. Utah Joint Dec. ¶¶ 35, 39, 42, 43, 44, 48, 49.

e. PFS has not demonstrated that the target performance goal has been meet for overturning and sliding of foundations. The basic principle based on DOE-STD-1020 is to ensure that if deterministic seismic evaluations are used, then DOE-STD-1020 outlines that the target performance goals are met when the minimum 10% probability of failure corresponds or is equal to 1.5 times the seismic scale factor times the DBE. Utah Joint Dec. ¶ 41.

32. The State disputes PFS Material Fact ¶ 32.

a. The risk reduction ratios claimed by PFS do not apply to foundations and interface conditions between the casks and the pad. Utah Joint Dec. ¶¶ 52, 59, 79.

b. HI-STORM casks are designed to NUREG-1536 and the CTB to NUREG-1567. PFS has not shown that these regulatory guides provide conservatisms equal to NRC standard review plans for nuclear power plants. Utah Joint Dec. ¶¶ 53, 54.

c. The Kennedy and Short fragility curves cited in NUREG/CR-6728 do not apply to HI-STORM 100 casks. Utah Joint Dec. ¶¶ 56 and 57.

d. The risk reduction ratios claimed by PFS are not supported by any calculations by the cask vendor. Utah Joint Dec. ¶ 58.

33. The State disputes PFS Material Fact ¶ 33 and Ebbeson Dec. ¶¶ 5-11 and 27. The percentage of time the crane will be in operational mode is dependent upon the time it takes to conduct transfer operations in the CTB and the rate of receipt of casks. *See* Ebbeson Dec. ¶11:

a. PFS's claimed 20% of the time or less operational mode of the crane in the CTB is premised on SAR, Rev. 6, Table 5.1-1 (anticipated time and personnel requiremenrts for HI-STORM canister transfer operations). Id. ¶¶ 8-11.

b. SAR Table 5.1-1 states that transfer operations in the CTB take approximately 20 hours.

c. Mr. Ebbeson does not know whether 20 hours is a reasonable period for tranfer operations. Ebbeson Tr. at 30-33.

34. Mr. Ebbeson has no direct knowledge of the cask receipt rate at the PFS faciltiy. Ebbeson Tr. at 33-34.

35. The State disputes PFS Material Facts ¶¶ 34 and 35. *See* Utah Material Facts ¶¶ 30-33; *see also* Resnikoff Dec.

36. The State disputes PFS Material Facts ¶¶ 37 and 38. To determine whether or not SSCs at PFS would have a mean annual probability of failure lower than essential structures designed to IBC-200 standard or bridges in Utah designed to a 2,000-year earthquake, PFS must first determine the failure of each SSC at the PFS site. Neither PFS nor Holtec have computed the failure of SSCs at the PFS site.

37. The State disputes PFS Material Facts ¶¶ 37 and 38. Whether the design of the PFS facility will provide a higher level of safety than certain essential structures in Utah is a matter of the conservatism or non-conservatism in the design of SSCs at the PFS facility. Arabasz Dec. ¶ 28.

38. The State disputes PFS Material Fact ¶ 39 and Cornell Dec. ¶ 49. Again it is the Staff's logic that the State is challenging, this time for the contradictory use of operational life of a facility. Arabasz Dec. ¶ 29. Under a correct application of the Staff's logic, the 40 year operational life of the PFS facility would have a DBE with a mean return period of 3,980 years. Id.

39. The State disputes PFS Material Facts ¶¶ 40 and 41.

a. The State conducted an independent study of the cask stability of the HI-STORM casks proposed for the PFS site. *See* Altran Report, Att. F to Utah Joint Dec.

b. The HI-STORM 100 cask vendor modeled the cask stability for a 2,000-year and 10,000-year earthquake at the PFS facility using beam finite elements with lumped masses. Utah Joint Dec. ¶ 63. On behalf of the State, Altran modeled the HI-STORM 100 cask for a 2,000-year earthquake at the PFS site using beam finite elements with lumped masses. Id. ¶ 62. Altran validated its HI-STORM 100 stability results generated by a structural analysis code, SAP2000, with ANSYS. Id. The structural analysis code used by the HI-STORM 100 vendor was not validated or compared with respect to a dry cask stability analysis. Id. ¶¶ 72, 73, Exhibit F.

c. The conclusions of the PFS HI-STORM 100 cask stability analyses are invalid and underestimate the potential for cask tip over and sliding because:

i. the cask dynamic analysis is highly sensitive to the local stiffness values used in the model which shows the model generates non-unique solutions (Utah Joint Dec. ¶¶ 66, 67);

ii. the contact vertical stiffness used in the PFS HI-STORM 100 cask stability analysis for a 2,000-year and 10,000-year earthquake at the PFS facility is unrealistic, thus resulting in an underestimation of vertical and horizontal displacement (id. ¶¶ 67, 68);

iii. the high BETA damping coefficients used in the PFS cask stability analyses underestimate the sliding and rocking (id. at ¶ 69); and

iv. the PFS cask stability analyses underestimates the vertical uplift because it did not account for amplification of the time history response due to soil structure interactions (id. at ¶ 70).

d. When subject to a 2,000-year earthquake at the PFS site and considering a realistic vertical contact stiffness of 1 x 10^6 pounds per inch, 0.01% damping, and a coefficient of friction of 0.8, the HI-STORM 100 cask displacement could be as high as 372.76 inches horizontally in the x direction, 229.65 inches horizontally in the y direction, and 27.24 inches vertically. Utah Joint Dec. at ¶ 74.

e. The HI-STORM 100 cask system will likely tipover when subject to the ground motions from a 2,000-year earthquake at the PFS site. Id.

f. Sliding, uplift, and tipover of objects similar or larger than a HI-STORM 100 cask have been documented during actual earthquakes. Id. ¶ 75, Att. G.

40. The State disputes PFS Material Fact ¶ 40. PFS is operating outside the bounds of the HI-STORM Certificate of Compliance (CoC). Resnikoff Dec. ¶¶ 10-13.

41. The State disputes PFS Material Facts ¶¶ 42-49. PFS and cask designer, Holtec have not quantified the consequences of a potential 2,000-year earthquake or a 10,000-year earthquake. Resnikoff Dec. ¶¶ 11, 13, 18, 19, 20, 21, 27.

42. The State disputes PFS Material Fact ¶ 45. In addition to not quantifying radiation dose increase at the site boundary from a tipped over cask with localized damage to the radial shield, PFS has not computed dose increase once those casks with localized damage are uprighted. Resnikoff Dec. ¶¶ 21, 22.

43. The State disputes PFS Material Fact ¶ 46.

a. Holtec has only performed an analysis on one cask tipping over; there is no

quantified analysis for 4,000 casks tipping over. Resnikoff Dec. ¶ 23.

b. The bottom of the HI-STORM cask provides reduced shielding. Id. ¶ 24 and Att. A.

c. The bottom of a row of tipped over HI-STORM casks facing the fence post will increase the dose rate at the site boundary between1.8 and 18 times (assuming 2,000 hours) or 7.7 and 77 times (assuming 8,760 hours) than that calculated by PFS. Id. ¶ 24 and Att. B.

d. The HI-STORM CoC set short term temperature limits to ensure the continued effectiveness of neutron shielding. Id. ¶ 26.

e. It is likely that PFS would not comply with the CoC short term temperature limits because PFS could not timely upright all HI-STORM casks. Id.

f. If the CoC short term temperature limits are exceeded, there would be an increase of neutron doses to workers of approximately 108 mrem/hour, which is up to 57.3 times greater than the value calculated by Holtec. Id.

44. The State disputes PFS Material Facts ¶¶ 47 and 48.

a. PFS computed the the occupational dose at the site boundary for 2,000 hours; 8,760 hours were used to compute the dose at the site boundary in the HI-STORM CoC. Resnikoff Dec. ¶ 14.

b. The the occupational dose at the site boundary for 8,760 hours is 25.6 mrem per year. Id. ¶ 15.

c. PFS cannot control whether there will be residential housing at the site boundary. Id. ¶ 16.

d. Holtec in its hypothetical tipover analysis assumed that the intial angular velocity of the cask was zero. Id. ¶ 19.
   i. During an earthquake, initial angular velocity would be greater than zero and the cask would decelerate at greater than 45g. Id.
   ii. At greater than zero initial angular velocity as the cask center passes the pivot point, the casks will flatten more than if the initial angular velocity is zero. Id. ¶ 20.

45. The State disputes PFS Material Facts ¶¶ 50 through 52, 54 and 55 because PFS has not considered the seismic stability of the foundation of the CTB. Utah Joint Dec. ¶ 76.

46. The State disputes PFS Material Fact ¶ 53.

a. The calculation for cask drop assumes a vertical drop from a height of 25 feet onto a concrete base. Resnkoff Dec. ¶ 29.

b. HI-STORM casks could be lifted vertically up to 27 inches. Id ¶ 28.

c. It is more likely that the HI-TRAC will drop on its edge (*i.e.*, corner drop). Id. ¶ 29.

d. Holtec has not evaluated a corner drop or a drop greater than 25 feet. Id.

e. Sheer stresses in a corner drop would be more severe than in a vertical drop. Id.

47. The issues raised inContention Utah QQ are inextricable linked to Utah L, Part B and to the extent that PFS Material Fact ¶ 57 imples that they are not relevant to Utah L, Part B, the State disputes PFS Material Fact ¶ 57. Utah Joint Dec.

CERTIFICATE OF SERVICE

I hereby certify that a copy of STATE OF UTAH'S RESPONSE AND OPPOSITION TO APPLICANT'S MOTION FOR SUMMARY DISPOSITION OF PART B OF UTAH CONTENTION L was served on the persons listed below by electronic mail (unless otherwise noted) with conforming copies by United States mail first class, this 7th day of December, 2001:

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EXHIBIT 1

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)	)	December 6, 2001

DECLARATION OF DR. WALTER J. ARABASZ

I, Dr. Walter J. Arabasz, declare under penalty of perjury and pursuant to 28 U.S.C. § 1746, that:

1. I am a Research Professor of Geology and Geophysics at the University of Utah in Salt Lake City, Utah, and also Director of the University of Utah Seismograph Stations. I have more than 30 years professional experience in scientific research, consulting, occasional teaching, and publishing articles in observational seismology, seismotectonics, and earthquake hazard analysis with a primary focus on Utah and the Intermountain West. Since 1977 I have routinely provided professional consulting services on earthquake hazard evaluations for dams, nuclear facilities, and other critical structures and facilities. Since the mid-1980s I have been directly involved in methodology development and applications of probabilistic seismic hazard analysis. During the past decade I have had major involvement in assessing vibratory and fault-displacement hazards for the high-level nuclear waste repository at Yucca Mountain, including serving on a Peer Review Group for Early Site Suitability Evaluation, reviewing technical reports, and serving on expert teams for seismic source characterization for probabilistic hazard analyses. My service on numerous national and state advisory boards and panels has included — relevant to this filing — serving on the National Research Council's Panel on Seismic Hazard Evaluation (1992-96), the Utah Seismic Safety Commission (1994 to present; chair, 1997-2001), and numerous panels and work groups under the National Earthquake Hazards Reduction Program since the early 1980s. An updated version of my *curriculum vitae* is attached hereto as Attachment A.

2. I was designated one of the State's testifying experts with respect to Contention Utah L, Basis 2, on June 28, 1999. I have reviewed the Applicant's SAR sections,

and updates thereof, relating to its earthquake hazards investigation of the proposed site, and relevant reports and other documents prepared by the Applicant or its contractors and submitted to the NRC or produced to the State in discovery. I have participated in answering the Applicant's discovery to the State as well as assisted in the preparation of discovery for the State directed to the Applicant. I am also familiar with NRC regulations, Rulemaking Plan to amend Part 72, guidance documents, the methodologies for earthquake hazard evaluation and new developments pertaining to the latter.

3. I have reviewed the NRC Staff's preliminary and final Safety Evaluation Report ("SER") for the PFS facility, dated December 15, 1999 and September 29, 2000 respectively, as well as the Staff's Position on Utah L (April 28, 2000).

4. I assisted in the preparation of the State of Utah's Request for Admission of Late-Filed Modification to Basis 2 of Utah Contention L, filed on January 26, 2000 and the State's November 9, 2000 Request for Admission of Late-filed Modification to Basis 2 of Utah Contention L.

5. I was deposed by Private Fuel Storage ("PFS") on October 31, 2001. I was present at the State's deposition of PFS's witness on the appropriateness of using probabilistic seismic hazard methodology, Dr. C. Allin Cornell, held on October 31 and November 1, 2001.

6. I have reviewed relevant portions of PFS's Motion for Summary Disposition of Part B of Utah Contention L (November 9, 2001), with primary attention to PFS's Motion, its Statement of Material Facts on Which No Genuine Dispute Exists, and the attached declaration of Dr. C. Allin Cornell. I provide this declaration in support of the State's Response and Opposition to Applicant's Motion for Summary Disposition of Part B of Utah Contention L (December 7, 2001).

7. I first became involved in providing technical expertise to the State of Utah regarding seismic hazards at the PFS facility in August 1998. Arabasz Tr. at 124-125. Since then, considerations by both the Applicant and the NRC Staff regarding the seismic design basis ground motions – or, for simplified reference, the design basis earthquake ("DBE") – for the PFS facility have continually evolved, providing a "moving target" for critical evaluation. Some of the noteworthy stages in this process include: (1) PFS's submission of its Safety Analysis Report in 1997 in which a "deterministic" approach was used for establishing the DBE aimed at meeting requirements of 10 CFR 72.102(f)(1); (2) PFS's Request for Exemption to CFR 72.102(f)(1) (April 2, 1999) in which PFS requested to calculate the DBE using a probabilistic seismic hazard analysis

("PSHA") and a 1,000-year recurrence interval; (3) the Staff's review of the Applicant's request and finding that use of a 1,000-year return-period value was not acceptable – but that use of a PSHA with a 2,000-year return-period value could be acceptable for reasons provided by the Staff (Staff's Preliminary Safety Evaluation Report ("PSER") (December 15, 1999) at 2-44 to 2-45; (4) Staff's finding the PSHA with a 2,000-year return period acceptable (Staff's Final Safety Evaluation Report ("FSER) (September 29, 2000)) at 2-41 to 2- 42); and (5) PFS's Motion for Summary Disposition of Part B of Utah Contention L (November 9, 2001) in which PFS has presented, for the first time in a documented way, its own case for justifying a DBE with a 2,000-year mean return period ("MRP").

8. I will proceed to frame the remainder of my declaration as follows. First, I will briefly revisit the original issue of a deterministic seismic hazard analysis ("DSHA"). Then I will address those issues, within my scope of expertise and testimony, associated with Contention Utah L, Part B, as set forth in the Board's Order dated June 15, 2001. *See also* PFS's Summary Disposition Motion at 1-3. In my remarks I will address issues that arose directly from arguments put forward by the Staff to justify a seismic exemption for the PFS facility (allowing a probabilistic DBE with a 2,000-year MRP) as well as new issues, relevant to my area of expertise, raised in PFS's Summary Disposition Motion.

I. Deterministic Seismic Hazard Analysis

9. In previous submissions to the NRC, I stated that PFS had not conducted a fully deterministic seismic hazard analysis ("DSHA") as required by 10 CFR § 72.102(f)(1) and, by reference, 10 CFR 100 Appendix A. *See e.g.*, State of Utah's Objections and Response to Applicant's Second Set of Discovery Requests With Respect to Groups II and III Contentions at 33-38 (June 28, 1999). The NRC Staff has acknowledged that the DSHA performed by Geomatrix Consultants, Inc. for the PFS facility and reported in the 1997 SAR and the updated DSHA reported in April 1999 "did not meet the deterministic requirements in 10 CFR 100 Appendix A." NRC Staff's Objections and Responses to the "State of Utah's Sixth Set of Discovery Requests Directed to the NRC Staff (Utah Contention L)" (February 14, 2000), Response to Requests for Admissions 1 and 2 at 7-8. (A later updated DSHA by Geomatrix Consultants, Inc. reported in April 2001 follows the same methodology as earlier and presumably would also not meet the deterministic requirements of 10 CFR 100 Appendix A.) The relevance of a valid DSHA, other than being required by current NRC regulations, is that it establishes a benchmark to which results of any probabilistic seismic hazard analysis can correctly be compared to evaluate the conservatism of the PSHA results, such as

earlier done for the NRC Staff by 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) at 2-46.

II. Bases of Contention Utah L, Part B, as Admitted

10. Basis 1 of Contention Utah L, Part B, states:

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

Board's Order of June 15, 2001 at 2.

My scope of testimony with respect to Basis 1 excludes radiological dose consequences. Arabasz Tr. at 29. Basis 2, which also deals with radiological dose limits, is similarly outside my scope of testimony. Id. The State has challenged the NRC Staff's proposal to grant an exemption request to PFS that would allow use of a DBE with a 2,000-year return period; the State argued, in part, that the NRC Rulemaking Plan set forth in SECY-98-126 (June 4, 1998) provides only two alternatives for design basis ground motions: a 1,000-year return period or a 10,000-year return period. State of Utah's Request for Admission of Late-filed Modification to Basis 2 of Utah Contention L (November 9, 2000) ("Request for Modification of Utah L")at 6-7. The Staff has rejected the use of a 1,000-year return period. FSER at 2-41. The Commission has instructed that the State "may not rely solely on the rulemaking plan [SECY-98-126] to prove its contention." CLI-01-12 (June 14, 2001) at 16. At the same time, the Commission instructed that "PFS is not bound by the rulemaking plan, but it does have the burden to show that the 2000-year design standard is sufficiently protective of public safety and property." Id.

11. PFS argues, in part, in its Motion for Summary Disposition (at 10) that non-compliance of a 2,000-year return period with SECY-98-126 is now mooted because the Staff has recommended a Modified Rulemaking Plan in which use of

a DBE with a 2,000-year MRP is proposed for dry-cask ISFSIs. SECY-01-0178 (September 26, 2001). Whether the latter indeed moots the issue is questionable in light of the Commission's recent issuance of Staff Requirements relating to SECY-01-0178, wherein the Commission writes:

> Central to this rulemaking is the determination of the mean annual exceedance probability of an earthquake at a proposed ISFSI. The proposed rule should solicit comment on a range of probability of exceedance levels from 5.0E-04 through 1.0E-04. Staff should undertake further analysis to support a specific proposal.

Memorandum to William D. Travers dated November 19, 2001.

12. The key contested issue linked to Basis 1 is the validity of PFS's claim that it has met the Commission's requirement to show that "the 2000-year design standard is sufficiently protective of public safety and property." PFS's Motion for Summary Disposition at 10. PFS's claim fundamentally rests on the proposition that sufficient protection "depends on both the probability of occurrence of the seismic event (often expressed as the mean annual probability of exceedence or "MAPE" of a given earthquake level) and the level of conservatism incorporated in the design procedures and criteria." PFS's Motion for Summary Disposition at 6. I agree with the proposition – but the latter critical part of PFS's claim of sufficient protection is challenged by the State's engineering experts, who dispute PFS assertions that it has demonstrated adequate conservatism in design of SSCs at the PFS facility. Here, and ultimately at the end of my declaration, I defer to these experts for more complete discussion of their disputes, which go the heart of "appropriately conservative" and "sufficiently protective" design of the PFS facility. *See* Joint Declaration of Dr. Steven F. Bartlett, Dr. Mohsin R. Khan, and Dr. Farhang Ostadan ("Utah Joint Declaration").

13. Basis 3 of Contention Utah L, Part B, states:

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

SSE for the PFS facility.

Board's Order of June 15, 2001 at 2.

In its Request for Modification of Utah L, the State evaluated the rationale put forward by the Staff in its September 2000 SER to justify a DBE with a 2,000-year return period for the PFS facility and characterized the Staff's reasons as *ad hoc* and either flawed or not compelling. Request for Modification of Utah L (November 9, 2000) at 7. Basis 3 concerns a series of three statements made by the Staff leading to the conclusion: "On the basis of the foregoing, the mean annual probability of exceedance for the PFS Facility may be less than [sic] 10^{-4} per year." FSER at 2-42. The Staff's flawed reasoning, as presented, was to posit that a design ground motion (for an SSE) at the PFS site which had a median reference probability of exceedance of 10^{-5} as defined in Regulatory Guide 1.165 would be the same as a design ground motion with a mean annual probability of exceedance of 10^{-4}. *See* Request for Modification of Utah L (November 9, 2000) at 8-10.

14. PFS's witness, Dr. Cornell, challenges Basis 3 on various grounds and concludes that "the argument raised by the State in Basis 3 is inconsequential and irrelevant to the issue whether a 2,000-year earthquake should be used at the PFSF." Declaration of C. Allin Cornell ("Cornell Dec.") at ¶40. What remains relevant is the benchmark for an SSE at the PFS site if the DBE for an ISFSI is to be compared to that benchmark, as was done by the Staff in its September 2000 SER. Absent a determination by the Staff along the lines of Dr. Cornell's beliefs of what the Staff "today would both select and prefer" (Cornell Dec. ¶35), or "could reasonably be expected to revert to" (id. ¶37), or "would likely conclude" (id. ¶38), or "would today not only accept but prefer" (id. ¶39), the State relied on guidance in Regulatory Guide 1.165 and on corresponding commentary by the Staff. Murphy et al., *Revision of Seismic and Geologic Siting Criteria*, Transactions of the 14th International Conference on Structural Mechanics in Reactor Technology (August 17-22, 1997), 1-12.

15. Dr. Cornell states that "The provision in Regulatory Guide 1.165 that a median value of 10^{-5} could be used is only the result of historical circumstances . . . [involving] a significant discrepancy in the assessment of the mean estimates between the two major CEUS seismic hazard studies then available . . . [which has] since been resolved . . ." (Id. ¶36). This assertion is at odds with the following commentary by the Staff in 1997:

It should be noted that this RP [Reference Probability of 1E-5/yr] is

> calibrated with the past design bases, it is not derived directly from any quantitative risk or safety goals. In fact, one of the reasons for using the median hazard curve in the regulatory guide approach is that the controlling earthquakes resulting from the de-aggregation of the median hazard curve are very similar to those used in the past licensing from the deterministic procedures.

Murphy et al. (1997) op. cit. at 7.

A similar commentary by the Department of Energy notes the following:

> In developing Regulatory Guide 1.165, NRC staff considered whether to define the reference probability as a mean or median value. The mean value has the advantage of better reflecting the uncertainty in the seismic hazard evaluation (i.e., it is sensitive to the range of interpretations of seismic source zone configurations, earthquake magnitude recurrence relationships, and ground motion attenuation relationships). However, precisely because the median is less sensitive to uncertainties, it provides a more stable regulatory benchmark than does the mean. Another consideration leading to the staff's preference for the median was the finding that, when median hazard curves were disaggregated, the magnitudes and distances of the controlling earthquakes tended to be more sharply defined and to agree better with the safe shutdown earthquakes of the selected plants than when mean hazard curves were disaggregated (Bernreuter et al. 1996).

DOE Topical Report YMP/TR-003-NP, 1997) at §3.1.2.1; *see* Exhibit 3 in PFS's Motion for Summary Disposition at pages 2-3 of 7.

16. From the above, it is not the State's argument that a median estimate should be used "in lieu of the mean estimate for the design of nuclear power plants, and similarly for ISFSIs . . ." PFS's Statement of Material Facts on Which No Genuine Dispute Exists at ¶19. Rather, the argument rests with the Staff's guidance in Regulatory Guide 1.165. Therein the procedure is specified for determining the reference probability, the annual probability of exceeding the SSE, at future nuclear power plants: "The reference probability [median annual exceedance probability of 1.0E-05] is also to be used in conjunction with sites not in the Central and Eastern United States (CEUS) . . . However, the final SSE at a higher reference probability may be more appropriate and acceptable . . . for some sites . . . Reference B.4 includes a procedure to determine an alternative reference

probability on the risk-based considerations; its application will also be reviewed on a case-by-case basis." Regulatory Guide 1.165 at 12.

17. Basis 4 of Contention Utah L, Part B, states:

> In supporting the grant of the exemption based on 2000-year return period, the staff relies upon United States Department of Energy (DOE) standard, DOE-STD-1020-94, and specifically the category-3 facility SSC performance standard that has such a return period, notwithstanding the fact the staff categorically did not adopt the four-tiered DOE category scheme as part of the Part 72 rulemaking plan.

Board's Order of June 15, 2001 at 3.

The Staff's reliance on DOE-STD-1020-94 in its December 1999 PSER and its September 2000 FSER to justify a DBE with a 2,000-year return period for the PFS facility suffers from two circumstances. First, DOE-STD-1020-94 was fully available to, and was referenced by, the Staff when it drafted its 1998 Rulemaking Plan (SECY-98-126). Yet the Staff chose in its 1998 Rulemaking Plan not to propose the use of a 2,000-year return period for ISFSIs. Second, the Staff cited the 2,000-year return period (mean annual probability of exceedance of 5×10^{-4}) for Performance Category-3 ("PC3") SSCs without acknowledging that in the design approach of DOE-STD-1020-94, the MAPE for PC3 is fundamentally coupled to a target seismic performance goal of 1×10^{-4} (the annual probability of exceedance of acceptable behavior limits). DOE-STD-1020-94 at B-7 to B-8.

18. PFS's Motion for Summary Disposition is replete with acknowledgments that, just as in the overall design approach of DOE-STD-1020-94, there should be a coupling of the hazard exceedance probability and a level of conservatism in design procedures that together ensure a desired performance goal. For example:

> [T]he risk of failure of a facility or structure depends on both the probability of occurrence of the seismic event (often expressed as the mean annual probability of exceedence or "MAPE" of a given earthquake level) and the level of conservatism incorporated in the design procedures and criteria. Cornell Dec. ¶13.

PFS's Motion for Summary Disposition at 6.

> As discussed above, the level of safety achieved depends on both the earthquake threat definition and the design procedures and criteria utilized to protect against that threat; thus, looking only at the earthquake return period is incorrect.

Id. at 15.

> Two factors are relevant to determining the likelihood of seismic failure of a facility or structure due to an earthquake event. These are (1) the seismic design basis earthquake ("DBE") for the facility or structure and (2) the conservatism embodied in the codes and standards applicable to its seismic design. Cornell Dec. ¶¶18-19; see also Arabasz Dep. At 41-42, 81-84, 115-117.

PFS's Statement of Material Facts on Which No Genuine Dispute Exists, ¶12.

> While the risk-graded approach is implemented in somewhat different ways in the various fields of seismic design, the standards of practice almost invariably utilize a DBE defined at some mean annual probability of exceedance and a set of design procedures and acceptance criteria.

Cornell Dec. ¶18.

> Both the MAPE of the DBE and the level of conservatism incorporated in the design procedures and criteria affect the failure probability of seismically-designed faciltities and structures. . . . [I]t is important to understand that both the MAPE and the level of conservatism in the design procedures and criteria must be considered when assessing and comparing the safety implications of various seismic design standards.

Cornell Dec. ¶19.

19. The discovery and deposition process for Contention Utah L, Part B, has led me to the opinion that determination of the mean annual exceedance probability (or equivalent return period) of a DBE for the proposed PFS facility, and whether it

ensures sufficient protection, cannot be made independent of an evaluation of conservatism (or non-conservatism) in design procedures.

20. A final point of particular relevance to Basis 4 is the recent release of Revised DOE Standard 1020-2001 for review and comment. Memorandum from Richard L. Black to Technical Standards Program Managers dated August 22, 2001. For PC3 the revised standard changes the MAPE from 5 x 10^{-4} (2,000-year return period) to 4 x 10^{-4} (2,500-year return period) while retaining the same target seismic performance goal of 1 x 10^{-4} per year for sites not near tectonic plate boundaries. Revised DOE-STD-1020-2001, Table C-3 at C-6, attached to Utah Joint Declaration as Att. D.

21. Basis 5 of Contention Utah L, Part B, states:

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

Board's Order of June 15, 2001 at 3.

22. In my opinion, circumstances specific to the seismic exemption awarded to DOE for the TMI-2 ISFSI at INEEL (SECY-98-071, April 8, 1998) do not justify using the exemption as a compelling precedent for the PFS exemption request. *See* State of Utah's Objections and Responses to Applicant's Seventh Set of Formal Discovery Requests to Intervenor State of Utah (September 28, 2001) at 16-1. *See also* Request for Modification of Utah L at Exhibit 1 (November 9, 2000); Deposition of Walter J. Arabasz (October 31, 2001) at 14-18, 42-43, 84-89; and State of Utah's Objections and Response to Staff's First Set of Formal Discovery Requests to State of Utah (November 5, 2001), Answer to Interrogatory No. 1 at 10-11.

23. The design basis of an existing higher risk facility, namely the Idaho Chemical Processing Plant ("ICPP"), at the host site for the TMI-2 ISFSI was a definite

consideration in DOE's proposal of a DBE for the ISFSI. Chen and Chowdhury, *Seismic Ground Motion at Three Mile Island Unit 2 Independent Spent Fuel Storage Installation Site in Idaho National Engineering and Environmental Laboratory – Final Report* (June 1998) at 4-1. Under existing DOE design standards at INEEL, based on DSHA results from the 1970s, the peak design basis horizontal acceleration for the ICPP was set at 0.36 g, including effects of soil amplification. Id. DOE proposed to use the same acceleration for the DBE for the TMI-2 ISFSI. In an analysis for the NRC, the regulatory problem was stated this way:

> [T]he DOE-proposed design PHA of 0.36 g does not bound the most recent 84th-percentile deterministic value of 0.56 g and 10,000-yr return period probabilistic value of 0.47 g. Therefore, a judgment of whether the DOE-design approach is acceptable depends on whether there are regulatory and technical bases to accept an ISFSI-design value that bounds the 50th-percentile deterministic value and the 2,000-yr return period probabilistic value.

Id. at 4-2.

24. Ultimately, DOE was allowed to use a design earthquake with 0.36 g peak horizontal acceleration (together with an appropriate response spectrum) for the TMI-2 ISFSI. SECY-98-071 at 3. What the NRC approved in terms of a design-basis ground motion was a design value higher than the 2,000-year return period mean ground motion from the PSHA. In their analysis for the NRC, Chen and Chowdhury provide information showing that the 0.36 g horizontal design value for the ISFSI soil site lies between the 2,000-year probabilistic value of 0.30 g and the 10,000-year probabilistic value of 0.47 g. Id. at 3-5. Although the report by Chen and Chowdhury does not contain sufficient information to identify precisely the return period corresponding to 0.36 g on soil, the bounding probabilistic values for 2,000 years (0.30 g) and 10,000 years (0.47 g) suggest that 0.36 g corresponds to a return-period value on the order of three to four thousand years (the precise return period would have to be determined from the original PSHA data). Thus, a 2,000-year return period for the PFS facility would be significantly lower than what was approved for the INEEL ISFSI.

25. Another factor that significantly influenced the Staff's approval of the TMI-2 ISFSI exemption was a site-specific radiological risk analysis coupled with "the lack of a credible mechanism to cause a failure." SECY-98-071 at 3.

26. On April 8, 1998, the NRC informed the DOE, "Since the rulemaking to revise the Part 72 seismic requirement for ISFSIs is unlikely to be completed before issuance of the TMI-2 ISFSI license, the staff intends to grant the exemption as requested if the Environmental Assessment (EA) is favorable. SECY-98-071 at 3. Two months later in June 1998, the Part 72 Rulemaking Plan (SECY-98-126 was released with allowance only for design basis ground motions with mean annual probabilities of exceedance corresponding to return periods of 1,000 years or 10,000 years, depending on risk. This sequence of events, in my opinion, does not support PFS's assertion that "there is no doubt that at the time the INEEL exemption was approved, the NRC Staff and the Commission expected (and intended) that it would serve as a precedent towards the granting of similar exemptions in the future." PFS's Motion for Summary Disposition at 14.

27. Basis 6 of Contention Utah L, Part B, states:

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

Board's Order of June 15, 2001 at 3.

28. PFS's witness, Dr. Cornell, addresses the relative comparison of a DBE with a 2,000-year mean return period proposed for the PFS facility with the higher return period value of approximately 2,500 years to be required by the International Building Code 2000. Cornell Dec. ¶46. He states:

> One should not draw the erroneous conclusion, however, that this difference in the definition of the DBE implies a lower probability of failure for SSCs designed to IBC-2000 versus those, such as the PFSF, designed to the 2,000 MRP and the NRC's SRP design procedures and criteria.

Id. Granting that "the safety achieved depends on *both* the DBE MRP and the design procedures and criterion utilized" (id.), the contested issue once again becomes the conservatism (or non-conservatism) in design of SSCs at the PFS facility. As in ¶12 above, I defer the latter issue to the State's engineering experts (including implications for the analogous situation of comparing a 2,000-year MRP DBE for the PFS facility with a 2,500-year MRP DBE for new highway

bridges in Utah). *See* Utah Joint Declaration.

29. Part (b) of Basis 6 (the significance of a 20-year initial licensing period versus a 30- to 40-year total operational period) concerns a metric the Staff put forward for justifying the adequacy of a 2,000-year return period for seismic design of the PFS facility, namely, a 99-percent probability that the DBE not be exceeded in the 20-year licensing period of the facility. The Staff wrote:

> Considering the radiological safety aspects of a dry spent fuel storage facility, conservative peak ground motion values that have a 99 percent likelihood of not being exceeded in the 20-year licensing period of the facility are considered adequate for its seismic design. This exceedance probability corresponds to a return period of 2,000 years.

PSER at 2-45. The Staff again relies on this same metric in its recent Modified Rulemaking Plan as one basis to justify the proposed mean annual probability of 5×10^{-4} (return period of 2,000 years) for a DBE for dry-cask ISFSIs. Attachment to SECY-01-0178 at 7. Therein, the Staff argues:

> The total probability of exceedance for a design earthquake at an ISFSI facility with an operational period of 20 years (20 years x 5.0E-04 = 1.0E-02) is the same as the total probability of exceedance for an earthquake event at the proposed pre-closure facility at Yucca Mountain with an operational period of 100 years (100 years x 1.0E-04 = 1.0E-02).

Id. Using this metric, a facility with an operational life of 40 years would have to have a DBE with a mean return period of 3,980 years. State of Utah's Objections and Responses to Staff's First Set of Formal Discovery Requests to State of Utah (November 5, 2001), Answer to Interrogatory No. 1 at 8-10.

30. PFS's witness, Dr. Cornell, attacks Basis 6(b) stating:

> This contention is unfounded because in virtually all areas of public safety hazards are measured as annual probabilities (or frequencies) of occurrence, regardless of the length of the activity in question, the exposure time, the estimated facility life, or the licensing duration [Ref. 12 (Paté-Cornell paper)].

Cornell Dec. ¶49. In my deposition, I deferred to probability experts, including Dr. Cornell, when asked, "Do you have an opinion as to whether risks should be expressed on an annual basis or the total life of a facility?" Arabasz Dep. at 51-52. However, I beg to differ with Dr. Cornell's statement above and will elaborate.

31. One of the well-established standards for portraying ground-shaking hazard in the United States is the suite of national seismic hazard maps published by the U.S. Geological Survey. "The hazard maps depict probabilistic ground motions and spectral response with 10%, 5%, and 2% probabilities of exceedance (PE) in 50 years." *National Seismic-Hazard Maps: Documentation June 1996, USGS Open-File Report 96-532* at 1. These maps provide reference ground motions for the International Building Code 2000. Dr. Cornell and I were co-members of a Review Panel for the USGS national maps in 1996.

32. Another well-established standard linked to building codes are the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 1997 Edition (FEMA 303)* ("Provisions"). The Commentary to the Provisions states:

> In past editions of the *Provisions*, seismic hazards around the nation were defined at a uniform 10 percent probability of exceedance in 50 years . . . . While this approach provided for a uniform likelihood throughout the nation that the design ground motion would not be exceeded, it did not provide for a uniform margin of failure for structures designed to that ground motion. . . . The approach adopted in these *Provisions* is intended to provide for a uniform margin against collapse at the design ground motion. . . . For most regions of the nation, the maximum considered earthquake ground motion is defined with a uniform likelihood of exceedance of 2 percent in 50 years (return period of about 2500 years.

Provisions, Part 2—Commentary at 37.

33. The National Research Council's Panel on Seismic Hazard Analysis noted the following:

> [A]TC-3 (Applied Technology Council, 1978) has suggested the design seismic hazard level should have a 10 percent probability of exceedance in 50 years, which

corresponds to an annual exceedance probability of about 2 x 10^{-3}. . . . The proposed Department of Defense tri-services seismic design provisions (Joint Departments of Army and Air Force, USA, 1985) suggests for category II facilities a dual level for the design seismic hazard. Such facilities should remain essentially elastic for seismic hazard with about a 50 percent probability of exceedance in 50 years or about a 1 x 10^{-2} annual exceedance probability and should not fail for a seismic hazard that has about a 10 percent probability of exceedance in 100 years . . ."

Panel on Seismic Hazard Analysis, *Probabilistic Seismic Hazard Analysis*, National Academy Press, Washington, D.C. (1988).

34. Procedures for estimating the probability of exceeding some level of ground motion during an exposure period of interest are commonly given for design guidance. For example, DOE-STD-1020-94 includes such a procedure at A-1, and Leon Reiter in his text, *Earthquake Hazard Analysis*, similarly includes such a procedure, including a graph from NUREG/CR-1582, 2 (1980), for relating return period, period of interest and desired probabilities of exceedance during the period of interest. L. Reiter, *Earthquake Hazard Analysis*, Columbia University Press (1990) at 185.

35. The cited paper by Paté-Cornell does not convincingly establish as a norm for public safety that "hazards are measured as annual probabilities (or frequencies) of occurrence, regardless of the length of the activity in question, the exposure time, the estimated facility life, or the licensing duration." Cornell Dec. ¶49. First, in the context of noting that "current PRA [probabilistic risk analysis] methodology tends to focus on the technical causes of system failure" (while ignoring human and organizational factors), Paté-Cornell writes: "Classical technical PRA's tend to focus on the probability that an extreme value of the loads to which a system may be exposed (during a given year <u>or lifetime</u>) exceeds its capacity." Paté-Cornell paper at 148, footnote 4, underlining added. Second, while hardly a commentary on "virtually all areas of public safety," the paper reviews five precedents as examples of safety targets: (a) nuclear power plants in the U.S., (b) cancer risks in the U.S., (c) offshore oil and gas industry in Norway, (d) fatality accident rate in the U.K., and (e) the Dutch government standards. Significantly, cases (b) and (d) involve risk measured per individual or worker <u>lifetime</u>. In case (c) the Norwegian Petroleum Directorate temporarily adopted a severe-accident criterion in terms of an annual probability of major initiators of platform failure but "recently backed away from their severe-accident criterion . . .

because this criterion was leading to a 'numbers game' that seemed to be distracting both the industry and the regulators from fundamental safety issues. . ." Id. at 150. Third, after discussing issues that have emerged in recent years in safety debate, Paté-Cornell proposes an approach to a global safety strategy, of which one element (of six) is that "it should be ensured that the *annual probability of catastrophic failure* (the severe accident criterion) is less than a specified threshold, e.g., 10^{-4} per year." Id. At 151. Fourth, the cited paper includes discussion of "time horizon" as a relevant risk factor, albeit in the context of shorter lifetime of aging facilities versus new ones.

36. Dr. Cornell attempts to bolster his argument by noting that "risk acceptance guidelines promulgated by the NRC" (for nuclear power plants) are in terms of annual risk for Core Damage Frequency and Large Early Release Frequency. Nevertheless, within a context of evolving regulatory guidance for ISFSIs, the Staff itself uses the metric of total probability of exceedance during a 20-year operational period to justify a DBE with a 2,000-year mean return period for dry-cask ISFSIs. Attachment to SECY–01-0178 at 7.

37. Finally, Dr. Cornell explains the reasons for focusing on annual risks in making safety decisions, in part, because "any facility providing a needed service will, at the end of its operating life, most likely be replaced by some other facility used for the same purposes with its own, similar risks." Cornell Dec. ¶49. While consideration of risk involving where spent fuel is now stored or may eventually be stored in the future at Yucca Mountain may be relevant for a societal global safety strategy (such as described in the Paté-Cornell paper), the issue at hand is a risk-acceptance decision specific to the the PFS site.

38. In this declaration I have attempted to systematically address each of the bases, within my scope of expertise and testimony, associated with Contention Utah L, Part B. In my opinion, the key contested issue is the validity of PFS's claim that it has met the Commission's requirement to show that "the 2000-year design standard is sufficiently protective of public safety and property" as called for by the Commission in CLI-01-12. PFS's claim fundamentally rests on the proposition that sufficient protection "depends on both the probability of occurrence of the seismic event (often expressed as the mean annual probability of exceedence or "MAPE" of a given earthquake level) and the level of conservatism incorporated in the design procedures and criteria." PFS's Motion for Summary Disposition at 6. I agree with the proposition – but the latter critical part of PFS's claim of sufficient protection is challenged by the State's engineering experts, who dispute PFS assertions that it has demonstrated adequate conservatism in design of SSCs at the PFS facility. I defer to these experts for more complete

discussion of their disputes, which go the heart of "appropriately conservative" and "sufficiently protective" design of the PFS facility. *See* Utah Joint Declaration.

Walter J. Arabasz

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discussion of their disputes, which go the heart of "appropriately conservative" and "sufficiently protective" design of the PFS facility. *See* Utah Joint Declaration.

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A

WALTER J. ARABASZ

Birthplace and Date: Acushnet, Massachusetts, September 30, 1942.

Current Position: Research Professor of Geology and Geophysics and Director, University of Utah Seismograph Stations; University of Utah, Salt Lake City, Utah.

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Education: B.S., Geology, summa cum laude, Boston College, 1964; M.S., Geology, California Institute of Technology, 1966; Ph.D., Geology and Geophysics, California Institute of Technology, 1971. Dissertation (supervised by Professor Clarence R. Allen): Geological and Geophysical Studies of the Atacama Fault Zone in Northern Chile.

Professional Positions: Post-Doctoral Research Fellow, Dept. of Scientific and Industrial Research, Geophysics Division, Wellington, New Zealand, 1970-73; Research Scientist, Lamont-Doherty Geological Observatory, 1973-74; University of Utah (1974-present): Research Professor of Geology and Geophysics (since 1983); Director, University of Utah Seismograph Stations (since 1985).

Society Affiliations: Seismological Society of America; American Geophysical Union; Geological Society of America; Earthquake Engineering Research Institute; Consortium of Organizations for Strong-Motion Observations Systems (COSMOS); Utah Geological Association.

Current Professional Activities: Utah Seismic Safety Commission—Member (since 1994), past Chair (1994-2001), and Member, Geoscience Standing Committee (since 1994); Regional Coordinator, Advanced National Seismic System, Intermountain West Region (since 2000); Member, Infrastructure Protection Subcommittee, Utah Olympic Public Safety Command (since 1998).

General Statement of Experience: Dr. Walter J. Arabasz has more than 30 years of professional experience in research, project management, consulting, and occasional teaching in observational seismology, tectonics, and earthquake hazard evaluation. He is the author or co-author of 37 published papers, 77 published abstracts and numerous technical reports. His present responsibilities at the University of Utah include seismological research and extensive project management—chiefly relating to the operation and modernization of a 160-station regional/urban seismic network covering Utah and neighboring parts of the Intermountain area.

He has been affiliated, since its inception, with the U.S. National Earthquake Hazards Reduction Program—variously as a Principal Investigator of funded research, as a participant in scores of workshops and conferences, and as a member of peer review panels. He has served on numerous national and state advisory and policy-making committees, including the Committee on Seismology of the National Research Council (1989-1994), the Board of Directors of the Seismological Society of America (1994-1997), the Council of the National Seismic System (1993-1999), a Senior Advisory Group to the U.S. Geological Survey for a 1999 report to Congress on an Assessment of Seismic Monitoring in the United States (1998-1999), and the Utah Seismic Safety Commission (1994-present). Since 1977 he has routinely provided professional consulting services on earthquake hazard evaluations for dams, nuclear facilities, and other critical structures and facilities for engineering firms, the International Atomic Energy Agency, the Department of Energy, the Soil Conservation Service, the Bureau of Reclamation, the Electric Power Research Institute, and the State of Utah.

PUBLICATIONS AND SELECTED REPORTS SINCE 1990

Arabasz, W. J., editor, 1990, A guide to reducing losses from future earthquakes in Utah—"Consensus document": Utah Geological and Mineral Survey Miscellaneous Publication 91-1, 30 pp.

Arabasz, W. J., editor, 1990, Earthquake instrumentation for Utah, Report and Recommendations of the Utah Policy Panel on Earthquake Instrumentation, Utah Geological and Mineral Survey Open-File Report 168, 167 pp.

Arabasz, W. J., and Atwood, G., 1990, Seismic instrumentation—A five-element, $3M state initiative for Utah's earthquake program [abs.]: Seismological Research Letters, v. 61, p. 37.

Johnston, A. C., Arabasz, W. J., Bollinger, G. A., Filson, J. R., Herrmann, R. B., Jones, L., and Kanamori, H., 1990, Assessing the Nation's earthquakes—The health and future of regional seismograph networks: Washington, D. C., National Academy Press, 67 pp.

Pechmann, J. C., Nava, S. J., and Arabasz, W. J., 1990, Left-lateral shear beneath the NW Colorado Plateau: The 1988 San Rafael Swell and 1989 So. Wasatch Plateau earthquakes [abs.]: Seismological Research Letters, v. 61, p. 44.

McGuire, R. K., and Arabasz, W. J. 1990, An introduction to probabilistic seismic hazard analysis, *in* Ward, S. H., ed., Geotechnical and environmental geophysics, Vol. 1: Review and tutorial: Society of Exploration Geophysicists, Investigations in Geophysics No. 5, pp. 333-353.

Arabasz, W. J., Pechmann, J. C., Nava, S. J., and Brown, E. D., 1991, Information from observational seismology relevant to earthquake engineering in the Wasatch Front area, Utah [abs.]: Earthquake Engineering Research Institute, Abstracts With Program 1991 Annual Meeting, Salt Lake City.

Pechmann, J. C., Nava, S. J., and Arabasz, W. J., 1991, Seismological analyses of four recent moderate (M_L 4.8 to 5.4) earthquakes in Utah, Final Report to the Utah Geological Survey, Salt Lake City, Utah, Contract No. 89-3659, 107 pp.

Smith, R. B., and Arabasz, W. J., 1991, Seismicity of the Intermountain seismic belt, *in* Slemmons, D. B., Engdahl, E. R., Zoback, M. D., Zoback, M. L., and Blackwell, D., eds., Neotectonics of North America: Geological Society of America, Decade Map Volume 1, pp. 185-228.

Arabasz, W. J., 1992, Earthquake basics for dam safety [extended abs.]: Abstracts and Proceedings, Utah Department of Natural Resources, The State of Utah Dam Safety Workshop, March 19, 1992, Salt Lake City, 9 pp.

Arabasz, W. J., 1992, New agenda for Committee on Seismology: Eos (Transactions American Geophysical Union), v. 73, no. 50, p. 541.

Arabasz, W. J., Pechmann, J. C., and Brown, E. D., 1992, Observational seismology and the evaluation of earthquake hazards and risk in the Wasatch front area, Utah, *in* Gori, P. L., and Hays, W. W., eds., Assessment of regional earthquake hazards and risk along the

Wasatch Front, Utah: U. S. Geological Survey Professional Paper 1500-A-J, pp. D1-D36.

Pechmann, J.C., Arabasz, W. J., and Nava, S. J., 1992, The St. George, Utah, earthquake of September 2, 1992: A normal-faulting earthquake with very weak aftershock activity [abs.]: Eos (Transactions American Geophysical Union), v. 73, no. 43, p. 399.

Pechmann, J.C., Nava, S. J., and Arabasz, W. J., 1992, Four moderate (M_L 4.8 to 5.4) earthquakes in Utah, 1987 to 1989 [abs.]: Seismological Research Letters, v. 63, p. 38.

Reasenberg, P. A., and others, including W. J. Arabasz, 1992, Remote seismicity triggered by the M 7.5 Landers, California, earthquake of June 28, 1992 [abs.]: Eos (Transactions American Geophysical Union), v. 73, v. 43 (Supplement), p. 392.

Arabasz, W. J., 1993, Earthquakes in Utah—Quantifying and dealing with the threat [abs.]: Proceedings, Utah Water Users Association, 1993 Annual Workshop, March 9-10, 1993, St. George, Utah.

Hill, D. P., and others, including W. J. Arabasz, 1993, Remote seismicity triggered by the M 7.5 Landers, California, earthquake of June 28, 1992: Science, v. 260, pp. 1617-1623.

Pechmann, J. C., Arabasz, W. J., and Brown, E. D., 1993, Seismotectonics of the 1987-88 Lakeside, Utah, earthquakes: Seismological Research Letters, v. 64, pp. 225-238.

Arabasz, W. J., 1994, Fundamentals of the Wasatch Front's earthquake threat: Proceedings, Seminar 1: Economic Impacts of a Large Earthquake, Earthquake Engineering Research Institute Wasatch Front Seismic Risk Regional Seminar; Salt Lake City, Utah, November 1994, pp. 1-1 to 1-25.

Arabasz, W. J., 1994, Wasatch Front seismicity and expectable strong ground motion: Proceedings, Seminar 2: Earthquake Research and Mitigation, Earthquake Engineering Research Institute Wasatch Front Seismic Risk Regional Seminar; Salt Lake City, Utah, November 1994, pp. 2-1 to 2-40.

Arabasz, W. J., and Hill, S. J., 1994, Aftershock temporal behavior and earthquake clustering in the Utah region [abs.]: Seismological Research Letters, v. 65, p. 32.

Nava, S. J., Arabasz, W. J., and Pechmann, J. C., 1994, The M 5.9 Draney Peak, Idaho (Idaho-Wyoming border) earthquake of February 3, 1994—A preliminary report [abs.]: Seismological Research Letters, v. 65, nos. 3/4, p. 239.

Nava, S. J., Arabasz, W. J., Pechmann, J. C., and Smith, R. B., 1994, Status and developments in data distribution and exchange—University of Utah Regional Seismic Network [abs.]: Eos (Transactions American Geophysical Union), v. 75, no. 44, p. 430.

Pechmann, J. C., Arabasz, W. J., and Nava, S. J., 1994, Refined analysis of the 1992 M_L 5.8 St. George, Utah, earthquake and its aftershocks [abs]: Seismological Research Letters, v. 65, p. 32.

Arabasz, W., and Malone, S., 1995, Council of the National Seismic System, background and update: Seismological Research Letters, v. 66, no. 4, pp. 29-30.

Arabasz, W. J, and Nava, S. J., 1995, Jordanelle Reservoir: Status of test monitoring for reservoir-induced seismicity [abs.]: Seismological Research Letters, v. 66, no. 2, p. 24.

O'Brien, E. H., Nava, S. J., and Arabasz, W. J., 1995, Earthquake education services for Utah schools [abs.]: Seismological Research Letters, v. 66, no. 2, p. 41.

Pechmann, J. C., and Arabasz, W. J., 1995, The problem of the random earthquake in seismic hazard analysis: Wasatch Front region, Utah, *in* Lund, W. R., ed., Environmental and engineering geology of the Wasatch Front region: Utah Geological Association Publication 24, pp. 77-93.

Pechmann, J. C., Walter, W. R., Nava, S. J., and Arabasz, W. J., 1995, The February 3, 1995, M_L 5.1 seismic event in the trona mining district of southwestern Wyoming: Seismological Research Letters, v. 66, no. 3, pp. 25-34 [minor correction, including revision of magnitude to M_L 5.2, added in v. 66, no. 4, p. 28].

Arabasz, W. J., and Hill, S. J., 1996, Applying Reasenberg's cluster-analysis algorithm to regional earthquake catalogs outside California [abs.]: Seismological Research Letters, v. 67, no. 2, p. 30.

Arabasz, W. J., and Wyss, M., 1996, Significant precursory seismic quiescence in the extensional Wasatch Front region, Utah [abs.]: Eos (Transactions American Geophysical Union), v. 77, no. 46, p. F455.

Wyss, W., Wiemer, S., and Arabasz, W. J., 1996, Seismic quiescence occurs as a precursor after all [abs.]: Eos (Transactions American Geophysical Union), v. 77, no. 22 (Supplement), p. W92.

Arabasz, W. J., Nava, S. J., and Phelps, W. T., 1997, Mining seismicity in the Wasatch Plateau and Book Cliffs coal mining districts, Utah, USA, *in* Gibowicz, S. J., and Lasocki, S., eds., Rockbursts and Seismicity in mines: Rotterdam, A.A. Balkema, pp. 111-116.

Pechmann, J. C., Brumbaugh, D. A., Nava, S. J., Skelton T. G., Fivas, G. P., and Arabasz, W. J., 1997, The 1994 Draney Peak, ID, earthquake and its aftershocks [abs.]: Eos (Transactions American Geophysical Union), v. 78, no. 46 (Supplement), p. F480.

Arabasz, W. J., 1998, Do regional seismic networks in the U.S. have a future?: Seismological Research Letters, v. 69 no. 6, pp. 513-515.

Arabasz. W. J., 1998, Utah's earthquake information needs—New technologies and solutions: Fault Line Forum (published by Utah Geological Survey), v. 14, no. 12, pp. 1-4.

Arabasz, W. J., and Pechmann, J. C., 1998, Earthquake data-base issues for seismic hazard analysis in the Utah region [abs.]: Proceedings Volume, Basin and Range Province Seismic-Hazards Summit, Utah Geological Survey Misc. Publication 98-2, p. 31.

Arabasz, W. J., and Wyss, M., 1999, Spatially-varying b-values in coal-mining seismicity in Utah: Inferred influence of heterogeneity and stress differences on local mean magnitude [abs.]: Eos (Transactions American Geophysical Union), v. 80, no. 46, p. F651.

Benz, H., Filson, J., Arabasz, W., and Gee, L.,1999, An assessment of seismic monitoring in the United States: Requirement for an Advanced National Seismic System: U.S. Geological Survey Circular 1188, 55 pp.

Pechmann, J. C., Nava, S. J., and Arabasz, W. J., 1999, Installation and calibration of five new broadband digital telemetry stations in Utah [abs.]: Seismological Research Letters, v. 70, no. 2, p. 244.

Arabasz, W. J., and McCarter, M. K., 2000. Mine seismicity and the interface between mining engineers and seismologists [abs.]: Seismological Research Letters, v. 71, no. 2, p. 220.

Arabasz, W. J., and Nava, S. J., 2000, New urban earthquake monitoring in Utah: Fault Line Forum (published by Utah Division of Comprehensive Emergency Management), v. 16, no. 1, p. 1,3.

Arabasz, W. J., Nava, S. J., Cluff, D. C., Drobeck, D. L., Moeinvaziri, A., and Pechmann, J. C., 2000, New real-time urban strong-motion network in Utah (Phase I) [abs.]: Eos (Transactions Ameican Geophysical Union), v. 81, no. 48, Fall Meet. Suppl., S71A-08, p. F864.

Pechmann, J. C., Nava, S. J., Bernier, J. C., and Arabasz, W. J., 2000, A critical analysis of systematic time-dependent coda-magnitude errors in the University of Utah earthquake catalog, 1981-1999 [abs.]: Eos (Transactions American Geophysical Union), v. 81, no. 48, Fall Meet. Suppl., S71C-10, p. F869.

Arabasz, W. J., and Pechmann, J. C., 2001, Seismic characterization of coal-mining seismicity in Utah for CTBT monitoring, Technical Report UCRL-CR-143772, prepared for Lawrence Livermore National Laboratory, Livermore, California, LLNL Research Agreement No. B344836, 120 pp.

Arabasz, W. J., Pechmann, J. C., and McCarter, M. K., 2001, Mechanisms and ground-truth observations for the largest (M_L = 3.0-4.2) coal-mining-induced seismic events in Utah, 1978-2000 [abs.]: Seismological Research Letters, v. 72, no. 2, p. 286.

Pankow, K. L., Pechmann, J. C., Nava, S. J., and Arabasz, W. J., 2001, Implementing ShakeMap in Utah before the 2000 Winter Olympics [abs.]: Seismological Research Letters, v. 72, no. 2, p. 232.

Pechmann, J. C., Bernier, J. C., Nava, S. J., Terra, F. M., and Arabasz, W. J., 2001, Correction of systematic time-dependent coda-magnitude errors in the Utah and Yellowstone National Park Region earthquake catalogs, 1981-2001 [abs.]: Eos (Transactions American Geophyical Union), v. 81, no. 49, Fall Meet. Suppl. (in press).

Youngs, R. R., and others, including W. J. Arabasz, 2001, Probabilistic fault displacement hazard analysis (PFDHA): Earthquake Spectra (submitted).

PROFESSIONAL CONSULTING

1. Rollins, Brown and Gunnell, Inc., Provo, Utah (1977). Site-response analysis for proposed hospital in Logan, Utah.

2. International Atomic Energy Agency, Vienna, Austria (Short-Term Expert, Chile, 1979). Seismotectonic considerations in northern Chile, 21°-27° S, with respect to the siting of a nuclear power plant.

3. Weidlinger Associates, Menlo Park, California (1980). Geological and geophysical information relevant to site-dependent ground motions at Wing V (Wyoming-Nebraska-Colorado).

4. EG&G Idaho, Inc. (Department of Energy), Idaho Falls, Idaho (1980-81). Preparation and presentation of proposal for seismic risk zone revision in southeast Idaho to International Conference of Building Officials.

5. EG&G Idaho Inc. (Department of Energy), Idaho Falls, Idaho (1982). Document review: *Site Investigation at Idaho National Engineering Laboratory, LMFBR Large Developmental Plant (LDP), Conceptual Design Study-Phase III.*

6. Lindvall, Richter and Associates, Los Angeles, California (1981-82). Seismic safety investigation of eight Soil Conservation Service dams in southwestern Utah.

7. U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado (1982-83). Review and analysis of geologic, seismotectonic, and design data for the proposed Jordanelle Dam, Bonneville Unit, Central Utah Project, Utah. (Consultant review by W. J. Arabasz, R. H. Jahns, and R. B. Peck.)

8. EG&G Idaho, Inc. (Department of Energy), Idaho Falls, Idaho (1983). Member, Geotechnical Advisory Panel to assist EG&G Idaho, Inc. and DOE regarding programmatic efforts toward site characterization of the Idaho National Engineering Laboratory for the proposed siting of a New Production Reactor Facility.

9. Electric Power Research Institute, Palo Alto, California (1984). Participant, "Data Needs Workshop; regarding data management plan and tectonic evaluation for earthquake hazards in the eastern U.S.; participant and editor of *Proceedings of a Seminar on Defining Tectonic Mechanisms Causing Earthquakes in the Eastern United States*.

10. Dames & Moore, Golden, Colorado/Electric Power Research Institute (EPRI), Palo Alto, California (1984-85). Member of "Seismic Hazard Methodology Team," EPRI Seismic Hazards Research Program, for evaluation of earthquake hazards in the eastern United States for the siting of nuclear generating facilities. (Participation in seven formal workshops, two academic seminars, and three series of interactive meetings with six teams of tectonic evaluation contractors in the central and eastern U.S.).

11. Electric Power Research Institute, Palo Alto, California (1985-87). Participation in scientific review, technical description, and comparative evaluation of EPRI seismic hazard methodology for the central and eastern United States.

12. U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado (1986-87). Review and analysis of geologic, seismotectonic, and design data for the proposed Jordanelle Dam, Bonneville Unit, Central Utah Project. (Consultant review by R. B. Peck, W. J. Arabasz, G. S. Tarbox, and D. D. Campbell.)

13. U.S. Bureau of Reclamation, Engineering and Research Center,Denver, Colorado (1988). Review and evaluation of seismotectonic conclusions and details of final embankment dam design for Jordanelle Dam, Bonneville Unit, Central Utah Project. (Consultant review by R. B. Peck, W. J. Arabasz, and T. G. McCusker.)

14. Dames & Moore, Los Angeles, California (1989). Member of advisory panel for project on seismic code decisions under risk, sponsored by the National Science Foundation.

15. Lawrence Livermore National Laboratory, Livermore, California (1990-91). Member of Seismicity and Tectonic Expert Group, New Production Reactors Project, Idaho National Engineering Laboratory Site.

16. U.S. Bureau of Reclamation-Engineering and Research Center, Denver, Colorado, and Regional Office, Salt Lake City, Utah (1990-92). Review and evaluation of foundation conditions, ongoing geologic mapping procedures, and seismic-safety aspects of the Jordanelle Dam, Bonneville Unit, Central Utah Project. (Consultant review by W. J. Arabasz, R. B. Peck, and D. D. Campbell.)

17. Science Applications International Corporation, Las Vegas, Nevada (1991). Member of Peer Review Group for Early Site Suitability Evaluation of the Potential Repository Site at Yucca Mountain, Nevada.

18. Geomatrix Consultants, San Francisco, California (1991-92). Member of expert panel, Electric Power Research Institute High Level Waste (EPRI-HLW) project to assess earthquake and tectonic issues for the proposed high-level nuclear waste repository at Yucca Mountain, Nevada.

19. Risk Engineering, Inc., Golden, Colorado (1992-94). Investigator for Seismology as part of a Seismic Hazard Study for Systematic Evaluation Program, Rocky Flats Plant, conducted for EG&G Rocky Flats, Inc. and sponsored by the U.S. Department of Energy.

20. Woodward-Clyde Federal Consultants, Las Vegas, Nevada (1993-94). Technical reviewer for (1) *Topical Report: Methodology to Assess Seismic Hazards at Yucca Mountain* and (2) *Seismic Design Inputs for the Exploratory Studies Facility at Yucca Mountain.*

21. U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado (1994). Review of design, construction, and operation of Jordanelle Dam and Reservoir, Bonneville Unit, Central Utah Project, Utah. (Consultant review by W. J. Arabasz, D. D. Campbell, and R. B. Peck.)

22. Jack R. Benjamin & Associates, Inc., Mountain View, California (1994). Technical reviewer for *Probabilistic Seismic Hazard Assessment for the U.S. Army Chemical Demilitarization Facility, Tooele, Utah.*

23. TRW Environmental Safety Systems, Inc., Vienna, Virginia (1995). Member of expert team for seismic source characterization for a probabilistic seismic hazard assessment of a high-level nuclear waste repository at Yucca Mountain, Nevada.

24. Rutherford & Chekene, San Francisco, California (1995). Technical review and consulting advice on seismicity and ground-motion considerations for design of a manufacturing plant at Lehi, Utah, for Micron Technology, Inc.

25. TRW Environmental Safety Systems, Inc., Vienna, Virginia (1995). Organizer and chair of plenary session of FOCUS 95—Methods of Seismic Hazard Evaluation (a topical meeting co-sponsored by the American Nuclear Society and the Geological Society of America, September 18-20, 1995, Las Vegas, Nevada).

26. William Lettis & Associates, Inc., Walnut Creek, California (1995). Technical review and consulting advice on seismic source characterization for the stability evaluation of Lake Almanor and ButtValley Dams, California.

27. Parsons Brinckerhoff, Salt Lake City, Utah (1995-96). Member of Seismic Advisory Committee to Utah Department of Transportation for seismic hazard analysis of the I-15 interstate highway corridor (consulting undertaken under a University of Utah contract).

28. TRW Environmental Safety Systems, Inc., Vienna, Virginia (1996-98). Member of expert team for seismic source characterization for a probabilistic seismic hazard assessment of a high-level nuclear waste repository at Yucca Mountain, Nevada.

29. Utah Department of Environmental Quality (1998-2002). Seismicity and earthquake expert for evaluation of a proposed high-level radioactive waste storage facility in Skull Valley, Tooele County, Utah (consulting undertaken under a University of Utah contract).

30. Los Alamos National Laboratory, Los Alamos, New Mexico (2000-2001). Member of Laboratory Seismic Review Committee, Nuclear Materials and Stockpile Management Program, an advisory group on the Laboratory's seismic risks and hazards and related technical and operational activities.

31. U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado (2001). Review of seismic hazard and design of Deer Creek Dam and Reservoir, Utah. (Consultant review by W. J. Arabasz, J. Mitchell, and R. B. Peck).

EXHIBIT 2

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)	)	December 7, 2001

JOINT DECLARATION OF DR. STEVEN F. BARTLETT, DR. MOHSIN R. KHAN, AND DR. FARHANG OSTADAN

Dr. Steven F. Bartlett ("SFB"), Dr. Mohsin Khan ("MK"), and Dr. Farhang Ostadan ("FO") hereby declare under penalty of perjury and pursuant to 28 U.S.C. § 1746, that:

1. (SFB) I, Steven F. Bartlett, am an Assistant Professor in the Civil and Environmental Engineering Department of the University of Utah, where I teach undergraduate and graduate courses in geotechnical engineering and conduct research. I hold a B.S. degree in Geology from Brigham Young University and a Ph.D. in Civil Engineering from Brigham Young University. I am a licensed professional engineer in the State of Utah.

2. (SFB) Prior to this University of Utah faculty position, I worked for the Utah Department of Transportation ("UDOT") as a research project manager and have held a number of other positions with UDOT and other employers where I have applied my expertise in geotechnical engineering, earthquake engineering, geoenvironmental engineering, applied statistics, and project management. My curriculum vitae is attached hereto as Attachment A.

3. (SFB) From 1991-1995 I worked for Department of Energy's ("DOE") contractor, Westinghouse, at the DOE Savannah River Site ("SRS"), near Aiken, South Carolina. I was Westinghouse's principal geotechnical investigator on a multi-disciplinary team overseeing the seismic qualification of the ITP/H-Area high-level radioactive waste storage tank farm for the SRS; the principal geotechnical investigator reviewing the Safety Analysis Report ("SAR") for the seismic qualification of Defense Waste Processing Facility, which is a high-level radioactive waste vitrification and storage facility at the SRS, and the project manager for the design of a hazardous waste landfill closure at the SRS. I used

DOE, NRC, and EPA regulatory guidance documents for design and review of these projects. I also have experience in the application of DOE Standard 1020 ("DOE 1020") to the design and seismic qualification of DOE nuclear facilities and the development of fragility curves for probabilistic-based assessment. I have also worked as a consulting engineer for Woodward-Clyde Consultants in Salt Lake City, mainly as a geotechnical designer for the Interstate-15 Reconstruction Project.

4. (FO) I, Dr. Farhang Ostadan, hold a Ph.D. in civil engineering from the University of California at Berkeley. I am a consultant in the field of soil dynamics and geotechnical earthquake engineering. I am also a visiting lecturer at the University of California at Berkeley and teach a graduate course on soil dynamics and soil-structure interaction. My curriculum vitae listing my qualifications, experience, training, and publications is attached hereto as Attachment B.

5. (FO) I have more than 20 years experience in dynamic analysis and seismic safety evaluation of above and underground structures and subsurface materials. I co-developed and implemented SASSI, a computer program for seismic soil-structure interaction analysis currently in use by the industry worldwide. I am also the technical sponsor of this program in collaboration with the University of California at Berkeley. Additionally, I am a member of the National Earthquake Hazard Reduction Program NEHRP foundation committee.

6. (FO) I have participated in seismic studies and review of numerous nuclear structures, among them Diablo Canyon Nuclear Station; the NRC/EPRI large scale seismic experiment in Lotung, Taiwan; the large underground circular tunnel for Super Magnetic Energy Storage; General Electric ABWR and SBWR standard nuclear plants; Westinghouse AP600 standard nuclear plant; Tennessee Valley Authority nuclear structures (Browns Ferry, Sequoyah, Watts Bar); and the ITP, RTF, and K-facilities in the Savannah River Site for the Department of Energy. I have published numerous papers in the area of soil structure interaction and seismic design for nuclear and other structures.

7. (MK) I, Dr. Mohsin Khan, hold a Ph.D. in solid mechanics (structures) from Clarkson College of Technology, Potsdam, N.Y., am currently employed by Altran Corporation, and am a consultant and registered professional engineer in the field of structural mechanics and mechanical engineering. My curriculum vitae listing my qualifications, experience, training, and publications is attached hereto as Attachment C.

8. (MK) I have more than 22 years experience evaluating various structures, systems,

and components subject to seismic ground motion. I have performed seismic studies and review of numerous nuclear structures, including Diablo Canyon Nuclear Power Plant ("Diablo Canyon"), Hanford, Indian Point, Humbolt Bay, Susquehanna Nuclear Power Plant (Mark II), Limerick, Hope Creek-Mark I, and Shoreham.

9. (MK) I am the Engineering Manager of Altran's Structural Mechanics group in San Francisco, which is involved in structural analysis, including conducting structural probabilistic risk assessments and developing seismic specifications for testing and design review support. I also developed a design basis criteria document for the Humbolt Bay Power Plant ("Humbolt Bay") Dry Cask/ISFSI project.

10. (MK) I have extensive experience designing and interpreting non-linear finite element models to show the structure, systems, or component performance under seismic forces. I have extensively used STRUDL, ANSYS, BSAP, SAP90, and SAP2000 computer programs and several in-house computer programs. Occasionally I have used the NASTRAN and STARDYNE computer programs. I have also extensively used the AISC Code and ASME Code, Section III and VIII, and IEEE-344 Standard for dynamic qualification of equipment. I am currently a member the of IEEE-344 working group. As an example of my experience, I was involved in the development of finite element models of equipment and the performance of computer analyses to qualify the equipment for the Diablo Canyon design basis seismic events. This involved reviewing the results of the analyses for completeness, accuracy, validity and documenting the results in design calculations.

11. (MK) I have substantial experience designing shake table tests and interpreting and utilizing shake table data to calibrate and refine engineering calculations. I have incorporated shake table data into probabilistic risk assessments and fragility evaluations.

12. (MK) When I was employed by Pacific Gas and Electric, I was a team member of the Diablo Canyon and Humbolt Bay Power Plants' Dry Cask Projects. I have unique technical and business experience at the sites and in the vendor selection process for the Dry Cask projects. I am familiar with Holtec International Inc.'s HI-STAR 100 and HI-STORM 100 design features and limitations. Further, I managed the equipment and structural seismic design and dynamic testing of many other capital projects at Diablo Canyon.

13. (MK) While employed at Clarkson College of Technology as an Assistant

Professor, I taught Finite Element Methods, Theory of Elasticity and Structural Dynamics as graduate courses, as well as developed several special finite element computer programs, structural optimization programs, and kinematic analysis programs.

14. (SFB, FO) We were designated as the State's testifying experts for this proceeding on June 28, 1999, and later for Contention Utah L part B.[1] We have reviewed the Applicant's Safety Analysis Report ("SAR") sections and updates thereof, relating to its geotechnical investigation of the proposed site and the seismic designs of various PFS structures, systems, and components ("SSCs"), and relevant calculations, reports, and other documents prepared by the Applicant its contractors and submitted to the NRC or produced to the State in discovery. We have participated in answering the Applicant's discovery to the State as well as assisted in the preparation of discovery for the State directed to the Applicant. We are familiar with and have applied NRC regulations and guidance documents as they relate to geotechnical review and seismic design of SSCs.

15. (SFB, FO) We have reviewed a number of calculations and analyses submitted by PFS to the NRC, including the following:
    a. Calc. No. 05996.02-SC-5, Rev. 2, *Seismic Analysis of Canister Transfer Building*, 4/4/01 (SWEC);
    b. Calc. No. 05996.02-G(B)-04, Rev. 7, *Stability Analysis of Storage Pads*, 3/30/01 (SWEC);
    c. Calc. No. 05996.02–G(B)-13, Rev. 4, *Stability Analysis of the Canister Transfer Building Supported on a Mat Foundation*, 3/30/01 (SWEC); and
    d. Errata to Correct SAR Chapter 2 of the PFSF License Application #22;
    e. Calc. No. 05996.02F(PO18)-3, Rev. 1, *Development of Time Histories for 2000-year return period design spectra*, 3/21/01 (Geomatrix);
    f. *Fault Evaluation Study and Seismic Hazard Assessment*, Rev. 1, March 2001 (Geomatrix);
    g. *Development of Design Basis Ground Motions for the Private Fuel Storage Facility*, Rev. 1, March 2001 (Geomatrix); and
    h. *Update of Deterministic Ground Motion Assessments*, Rev. 1, April 2001 (Geomatrix). Stone & Webster Calculation No. 05996.02-G(B)-04, Rev. 8, *Stability Analyses of Cask Storage Pads* (May 31, 2001);
    i. Stone & Webster Calculation No. 05996.02-G(B)-13, Rev. 5, *Stability Analyses of Canister Transfer Building* (May 31, 2001);

[1]Dr. Bartlett was named on September 28, 2001. Dr. Ostadan was named on October 23, 2001.

j. Stability Analyses of Cask Storage Pads, Cal. No. 05996.01-G(B)-04, Revision 9 (Stone & Webster).

k. Stability Analyses of Canister Transfer Building, Cal. No. 05996.01-G(B)-13, Revision 6 (Stone & Webster).

l. Holtec letter dated August 8, 2001, related to Holtec's site-specific ISFSI pad evaluation and submitted to NRC by PFS on August 7 as "Commitment Resolution Letter #37";

m. HI-2012780, *Dynamic Response of Free-Standing HI-STORM 100 Excited by 10,000 Year Return Earthquake at PFS*, Rev. 0 (November 9, 2001); and

n. PFSF Site Specific HI-STORM Drop/Tipover Analyses, Rev. 2, HI-2012653.

16. (MK) I have reviewed relevant calculations and reports relating to the seismic design of the Holtec HI-STORM 100 and its ability to withstand ground motions at the PFS site. I became involved with assisting the State on this contention at the end of October 2001.

17. (SFB, FO) We have reviewed Applicant's Motion for Summary Disposition of Part B of Utah Contention L (November 9, 2001), its Statement of Material Facts on Which No Genuine Dispute Exists, and all attachments thereto.

18. (MK) I have reviewed relevant portions of Applicant's Motion for Summary Disposition of Part B of Utah Contention L (November 9, 2001), including the Joint Declaration of Krishna P. Singh, Alan I. Soler, and Everett L. Redmond II.

19. (SFB, MK, FO) We provide this declaration in support of the State of Utah's Response Opposing PFS's Motion for Summary Disposition of Contention Utah L Part B. The following statements in this declaration are based on our experience, training, and best professional judgment.

20. (SFB, MK, FO) Dr. Bartlett was present for the telephone depositions of Bruce Ebbeson, Dr. Krishna Singh, and Dr. Alan Soler. Specific to Utah Contention L, Part B, we have reviewed the deposition transcripts of Dr. C. Allin Cornell, Bruce Ebbeson, Dr. Krishna Singh, and Dr. Alan Soler.

21. (SFB, FO) We were deposed by PFS with respect to Utah Contention L, Part B.[2]

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[2]PFS deposed Dr. Ostadan on November 1, 2001 and Dr. Bartlett on November 2, 2001.

We have reviewed the State of Utah's Statement of Disputed and Relevant Material Facts (November 29, 2001), which includes citations to our deposition testimony. The opinions expressed in our individual depositions remain the same. In addition to this declaration, the transcripts of our individual depositions describe many of the concerns we have with PFS's seismic analysis.

Overview

22. (SFB, FO) Utah L, Part B centers on the choice of the appropriate design basis earthquake. Such a choice, however, is inextricably linked to the performance of structures, systems and components important to safety at the PFS facility. Another way of saying this is: given the target performance of the SSC, what is the appropriate earthquake hazard level to be used in the design. The key for this selection is the performance of the SSC subjected to the design motion and the collective experience gained from previous design and performance of the SSC subjected to similar seismic loading. This would amount to a fragility curve for the SSC. The fragility curve in combination with the seismic hazard curve yields the probability of failure of the SSC and this probability is compared with the probabilistic target performance goal for the SSC to determine if the performance is adequate. At this time, PFS has not produced any fragility curve for the casks and its foundation system nor does any precedence exist for such a system subjected to the high level of seismic loading postulated for the PFS site.

23. (SFB, FO) As can be seen from the above paragraph, there is a co-joining of the probability of an earthquake and the probability of failure of SSCs. PFS has adopted a design motion on the basis of mean annual probability of exceedance of 5×10^{-4} (2,000-year return period). Central to PFS's justification for the use of 2,000-year motion is PFS's analogy to the performance goals of SSCs and risk reduction factors in DOE Standard 1020, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* and NUREG/CR-6728, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines* (October 2001), and reliance on conservatism built into NRC review plans. The PFS facility, including SSCs, and the calculations and analyses PFS has used in its SAR to support an ISFSI license, cannot be analogized to the risk reduction factors in DOE Standard 1020 or NUREG/CR 6728 because PFS has only chosen those parts of DOE Standard 1020 and NUREG/CR-6728 that may support its position; it has not conducted the full panoply of analyses required by those documents. In addition, NRC review plans do not address the seismic performance requirements that apply at the PFS site, *i.e.*, unanchored casks resting on a shallow pad foundation, buttressed by soil treated cement and subject to high

levels of strong ground motion. PFS has failed to use a cohesive or comprehensive approach to justify using a 2,000-year earthquake at the PFS site.

24. (SFB, FO) The SSCs of concern for seismic analysis are Category I SSCs. At the proposed PFS facility, Category I SSCs are the Canister Transfer Building ("CTB") and certain components therein, the storage pads, and the HI-STORM cask system. SAR at 3.4-3 to -4, Revs. 17, 9 respectively. PFS has omitted to include in its discussion of Category I SSCs the performance of the foundation of the CTB and the foundation of the storage pads. This is a glaring omission. For example, an evaluation of whether the crane in the CTB will perform under seismic loads is pointless if the CTB foundation fails under those seismic loads.

25. (SFB, FO) What is clear to us is that PFS's choice of a DBE cannot be divorced from the critical issues the State has raised in Contentions Utah L, Part A, Basis 3 and Utah QQ. The assumptions underlying the design of the PFS facility and quantitative analyses thereof are central to whether there is conservatism in PFS's design. In our opinion, PFS's attempt to justify a 2,000-year DBE by claiming conservatism in its design cannot be judged without evaluating these claims against the unconservatism of PFS's design, such as the use of unrealistic assumptions, omissions and gross generalizations to show that certain SSCs at PFS will adequately perform given a 2,000-year DBE.

26. (SFB, FO) In this joint declaration, we first give examples of the design basis ground motions based on deterministic and 2,500-year DBEs for certain facilities in Utah and the San Francisco area and as well give a brief background of the development of ground motions for the PFS site. Next we discuss the factors affecting selection of the DBE, the application of DOE Standard 1020 and NUREG/CR-6728 and show how PFS fails to conform with those documents. Finally, we address, specifically, how PFS has failed to show conservatism in the design of SSCs, including the foundations of the storage pad and CTB and the stability of the casks. As part of our showing, we rely on an independent analysis performed by Dr. Khan to evaluate sliding and tip-over of the HI-STORM 100 casks under seismic motions for a 2,000-year DBE. Dr. Khan concludes that Holtec's analysis significantly underestimates cask sliding, uplift and tip over and that, without actual test data, Holtec's model generates meaningless results. This evaluation clearly indicates that the target performance set for the controlled and stable movement of the cask cannot be met under the 2,000-year DBE.

27. (FO) The regulation from which PFS has requested an exemption, 10 CFR § 72.102(f)(1), essentially requires that the DBE be equivalent to the deterministic or maximum credible earthquake. *See* PFS Request for Exemption to 10 CFR

72.102(f)(1) (April 2, 1999). In its exemption request, PFS asked to use probabilistic seismic hazard analysis methodology. Id. At the time PFS requested its exemption, PFS estimated the 84th percentile peak ground accelerations at the site were 0.72g in the horizontal direction and 0.80g in the vertical direction. In 2001, PFS's revised 84th percentile peak ground acceleration shows 1.15g in the horizontal direction and 1.17 g in the vertical direction. *See* Geomatrix, *Update of Deterministic Ground Motion Assessment*, Rev. 1, April 2001 at 3. The 2001 revised peak ground accelerations for a 2,000-year return period are now 0.711 g in the horizontal direction and 0.695 g in the vertical direction. SAR at 2.6-107, Rev. 22. The increase in the ground motion increases the demand on the design of the storage pads and Canister Transfer Building as well as on the unanchored casks placed on the pads.

Deterministic Design Basis Earthquake

28. (FO) Either probabilistic or deterministic methodologies are acceptable in establishing design basis ground motions. I am currently working on or aware of major projects that will be built to a deterministic DBE, including the retrofit of the San Francisco Bay Area Rapid Transit System and seismic upgrade of the facilities of the Bay Area East Bay Municipal Utility District based on deterministic scenario earthquakes. These projects rely on a deterministic DBE to meet the performance expected of the facility during the design earthquake.

29. (SFB) Seismic evaluation of State regulated dams in Utah requires the use of a deterministic maximum credible earthquake ("MCE"). Utah Administrative Code R655-11-5A.

Use of Anchored Casks

30. (MK) During the course of my work associated with dry cask storage projects for Pacific Gas and Electric ("PGE"), NRC staff has not granted a license for unanchored vertical casks at any sites with peak ground accelerations greater than 0.4 g due to the greater potential for sliding and tipping of these casks containing irradiated fuel assemblies.[3] In fact, based on my personal knowledge, because of the high ground motion levels at Diablo Canyon and Humbolt Bay sites, PGE required Holtec to provide a design and analysis for an anchored vertical HI-

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[3]PFS witness, Dr. Soler, stated that unanchored HI-STORM 100 casks have never been considered at any site with ground motions equivalent or greater than those for the PFS 2,000 year earthquake. Singh/Soler Tr. at 70.

STORM 100S/HI-STAR 100 cask system.

Design Basis Earthquake Return Period

31. (FO) PFS relies, in part, on DOE-STD-1020-94 to support its claim that design basis ground motion for a 2,000-year return period is conservative for the PFS facility. However, the recently released draft DOE-STD-1020-2001 now requires a 2,500-year return period ground motion for PC3 SSCs. *See* DOE-STD-1020-01, at C-6, attached hereto as Attachment D. Current seismic hazard maps such as those published by U.S. Geological Survey and the National Earthquake Hazard Reduction Program (NEHRP) have also adopted a 2,500-year motion for design use. Also, the most recent design codes such as those adopted or considered by the International Building Code and American Association of State Highway and Transportation Officials require a 2,500-year motion for design. Based on the direction of other prominent agencies and organizations in the field of seismic design, it is my opinion that DOE will in fact require a 2,500-year motion in the final DOE-STD-1020-01. Hence, PFS's reliance on a 2,000-year design basis earthquake is no longer adequate.

32. (SFB) The Utah Department of Transportation currently requires all interstate highway bridges to be designed to levels of strong ground motion that exceed the proposed design basis ground motion at the PFS site. UDOT design basis ground motions are based on a uniform hazard spectrum with spectral values that have a 2 percent probability of exceedance in 50 years. This is equivalent to an average return period of 2,500 years.

33. (SFB) For sites with nearby faults, such as the PFS site, strong ground motions derived from a deterministic maximum credible earthquake event are likely to exceed the 2,000-year design basis proposed by PFS. Thus, the design of interstate bridges and critical facilities in the State of Utah are designed to more stringent earthquake requirements than those proposed by PFS. *See* above ¶ 29.

Factors Affecting Selection of Design Basis Earthquake

34. (SFB, FO) The PFS design and analysis are not conservative. For example, and as discussed above, it is unprecedented to design unanchored dry storage casks for a seismically active area with such intense strong ground motions similar to those at the PFS facility. Also, PFS's claim that the casks will only slide in a "controlled" manner atop the pads contradicts general engineering principles. The lack of conservatism in its analysis is further compounded when PFS uses its claim of "controlled" cask sliding to reduce the seismic loadings to the pad foundations.

These and other non-conservative aspects of PFS's design and analysis will be discussed below.

35. (SFB, FO) To support its motion for summary disposition of Part B, Utah L, PFS relies, in part, on DOE-STD-1020 to justify the adequacy of setting the design basis ground motion on a 2,000-year earthquake. PFS Motion at 6-9. PFS relies on DOE-STD-1020, but it has failed to recognize the basic principles in the DOE-STD-1020 for selection of the DBE. These principles include recognizing the behavior of specific SSCs, the fragility of SSCs, and the sensitivity of the fragility to normal variation of the material properties and uncertainties. DOE-STD-1020 principles provide further assurances that there is adequate conservatism when using a design basis ground motion at a specific site. The following will discuss the necessity to consider DOE-STD-1020 or similar critical design principles prior to setting the design basis earthquake for the PFS facility.

36. (SFB, FO) In accordance with DOE-STD-1020, the design and evaluation criteria for a critical facility, such as an ISFSI, must consider the level of conservatism or lack of conservatism introduced in the design/evaluation process by the DBE. Such an evaluation must be based on the performance of the facility under the proposed earthquake loading. PFS relied substantially on the philosophy of DOE-STD-1020 to justify its selection of the DBE at the PFS site. Cornell Dec. at ¶¶ 20-25. However, DOE-STD-1020 first requires that the SSC be categorized according to DOE-STD-1021, and performance goals are established based on the hazard classification. DOE-STD-1020 gives the design and evaluation criteria that control the level of conservatism introduced in the design/evaluation process. These criteria ensure that the level of conservatism and rigor in the design/ evaluation process is appropriate for the category of the facility. DOE-STD-1020 requires the selection of a target performance goal for the SSC and sufficient evaluations that document the SSC will indeed meet the performance goal for the DBE. The performance goals used in DOE-STD-1020 are probabilistic thresholds, where the probability of unacceptable performance or failure of an SSC is expressed in terms of a mean annual probability of exceedance. Unacceptable performance is considered to be damage to the SCC beyond which hazardous material confinement and <u>safety-related functions are impaired</u>. Thus, the selection of the DBE ground motion is explicitly coupled with a thorough evaluation of the fragility of or damage to the SSC.

37. (SFB, FO) A probabilistic method to determine if a performance goal has been met for a particular SSC is to develop a fragility curve for each SSC. A fragility curve expresses the expected damage or unacceptable performance of an SSC as a function of the amplitude of strong ground motion. Once a fragility curve has

been established for a particular SSC, the probability of unacceptable performance can be calculated for all levels of strong ground motion, even for levels beyond those incurred by the DBE.

38. (SFB, FO) In selecting the performance goals for an SSC, DOE-STD-1020 adopted a graded approach for SSCs. Based on this approach, the performance goal of the SSC is selected. For PC3 SSCs, the performance goal is set to be 10^{-4}. The key for this selection is the knowledge of the fragility curve for the SSCs. By evaluating the fragility curve for the SSCs and recognizing the detail design and ductility of the SSC under earthquake loading, a risk reduction factor of 4 has been adopted for PC3 SSCs. Therefore, to meet the performance goal of 10^{-4}, the DOE-STD-1020-2001 recommends a 2,500-year return earthquake for PC3 SSCs.

39. (SFB, FO) The determination of fragility as expressed as a fragility curve allows the assessment of the conservatism of the design for multiple levels of ground motion. The calculation and application of a fragility curve are necessary to determine if an SSC has met a desired performance goal for all levels of strong ground motion.

40. (SFB, FO) DOE-STD-1020 also discusses the use of a risk reduction ratio to determine if the SSC performance goal has been met. Sometimes SSCs are evaluated according to deterministic methods, which are found in applicable codes and standards. The basic principle based on DOE-1020 is to ensure that the target performance goals are met when the minimum 10% probability of failure corresponds to 1.5 times the seismic scale factor times the DBE.

41. (SFB, FO) DOE-STD-1020 also recognizes that specific acceptance criteria for foundations have not been developed. It states that the intent of DOE-STD-1020 must still be met for some system components or overturning or sliding of foundations (p. 2-24). This intent is that "there should be less than 10 percent probability of unacceptable performance at input ground motion defined by a scale factor [SF] of 1.5SF times the DBE." DOE-STD-1020 at 2-24. PFS has not made this calculation nor demonstrated that the intent of DOE-STD-1020 has been met for the foundation systems of the storage pads and CTB.

42. (SFB, FO) For soil sites, like the PFS site, because the slope of the hazard curve can be impacted by the soil nonlinear behavior, NUREG/CR-6728 recommends to establish the slope of the hazard curve by including the nonlinear soil effects for determination of the seismic scale factor. This concept is applicable to any nonlinear behavior such as cask sliding on the pads since the response is nonlinear and is effectively based on performance design and cannot be extrapolated from

the response at lower level ground motions. PFS has not considered these nonlinear effects, nor has it calculated the seismic scale factor SF based on considerations of the slope of the hazard curve.

43. (SFB, FO) In adopting the 2000-year design basis ground motion, PFS failed to meet the principles of the DOE graded approach. Specifically, the critical role of defining the fragility curve for the SSC and slope of the hazard curve, the effect of soil nonlinear behavior on the slope of the hazard curve, and the nonlinear response of the casks on the pads have been omitted.

44. (SFB, FO) PFS has not developed fragility curves for the HI-STORM 100 cask system relating to excessive movement and collision of the casks, tipover of the casks, excessive uplift and separation of the casks from the pad, or the consequence of such unstable cask and pad conditions.[4] Nor has PFS demonstrated that the storage pad and CTB foundation meet the acceptance criteria required in DOE-STD-1020. Thus, PFS has failed to show that the SSCs can meet a target performance goal of 1 x 10^{-4} for the associated 2,000-year annual return period under DOE-STD-1020-94.

45. (SFB, FO) PFS states the recently released NUREG/CR-6728, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard and Risk-consistent Ground Motion Spectra Guidelines* (October 2001), Chapter 7 supports, in general, that NRC standard review plans ("SRPs") provide equal or greater levels of conservatism than DOE-STD-1020. Cornell Dec. at 25. NUREG/CR-6728 defines a probabilistic approach similar to DOE-STD-1020 for determining the seismic performance of an SSC. NUREG/CR-6728, page 7-4 states:

> The goal in developing risk-consistent spectra is to understand the relevant design codes and their implicit seismic margin, choose an appropriate DRS [design response spectrum] (perhaps scaled by some factor), and ensure that the resulting probability of component failure (represented by its mean or median) is acceptable.

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[4]PFS's DBE witness had no knowledge of any fragility curves for the HI-STORM 100 cask system, the storage pad, or the CTB at the PFS facility. Cornell Tr. at 49. In fact, PFS's witness responsible for the seismic stability evaluations of the storage casks was unfamiliar with a fragility curve or its purpose. *See* Singh/Soler Tr. at 63.

46. (SFB, FO) NUREG/CR-6728, page 7-4, also lists the items that affect the calculated probability of failure of an SSC. These are:

- The annual probability of exceedance that is chosen for the Design Response Spectrum, and whether the choice is a median or mean value.
- The slope of the seismic hazard curve.
- The design seismic capacity and its uncertainty.
- The uncertainty in the seismic response

47. (SFB, FO) NUREG/CR-6728, page 7-5, defines how the probability of failure of an SSC can be calculated from the hazard curve and the fragility of the SSC. The fragility is the conditional probability of failure for a particular SSC for given strong ground motion with an amplitude a. The failure probability, P_F of an SSC is calculated from:

$$P_F = \int H(a)\,(dP_{F/a} / da)\,da$$

where H(a) is the annual probability of exceeding a from the probabilistic seismic hazard curve and $P_{F/a}$ is the probability of failure (i.e., fragility) given amplitude a. Values of $P_{F/a}$ define a fragility curve and the determination of this curve must take into account uncertainties in the response and capacity of the SSC as listed above.

48. (SFB, FO) PFS, in trying to justify the selection of the DBE, has failed to consider the guidance given in NUREG/CR-6728. PFS has failed to assess: 1) the slope of the seismic hazard curve, 2) the design seismic capacity of the SSC and its uncertainty, and 3) uncertainty in the seismic response.

49. (SFB, FO) In summary, PFS has not met the intent and requirements of DOE-STD-1020 and NUREG/CR-6728. It is impossible to assess the fragility for the storage pads, storage casks, and the CTB and their foundations because of many errors, omissions and unconservative assumptions in PFS's evaluations. PFS has not demonstrated that the performance goal for the PFS facility has been met. Without this demonstration, the selection of the proposed DBE is not founded on a proper technical basis and is basically arbitrary.

Performance Goals Are Not Clearly Inherent in ISFSI and Cask SRPs

50. (SFB, FO) PFS's witness asserts that "using design basis earthquake (or "DBE") ground motions associated with" a 2,000 year earthquake in conjunction with

"applying the design procedures and criteria of the NRC's SRPs . . . will achieve a level of seismic safety that is appropriate" for the PFS facility. Cornell Dec. ¶ 13.

51. (SFB, FO) In an attempt to demonstrate that performance goals are unnecessary, PFS claims that NRC SRPs have equivalent or greater risk reduction ratios as those stated in DOE-STD-1020-94 for performance category 3 and 4 facilities.[5] Cornell Dec. ¶ 25. Thus, surmises PFS, risk reduction factors of approximately 5 to 20 can then be claimed for the PFS SSCs. Id.

52. (SFB, FO) PFS's asserted risk reduction ratios of 5 to 20 for PFS SSCs are unsubstantiated. NRC SRP requirements do not address the seismic performance requirements of unanchored casks supported by shallowly embedded pad foundations which are buttressed by cement-treated soil and subject to high levels of strong ground motion. The proposed PFS design has unique seismic interface issues and must be analyzed accordingly.

53. (SFB, FO) PFS itself only claims that the SRPs for nuclear power plants ("NPP") are equivalent or greater than DOE-STD-1020 design criteria. Cornell Dec. ¶ 25. The HI-STORM 100 cask system is not designed to SRPs governing NPPs but to NUREG-1536, Standard Review Plan for Dry Cask Storage Systems. The canister transfer building must be designed according to NUREG-1567, Standard Review Plan for Spent Fuel Dry Storage Facilities. PFS has not shown that the SRPs for dry cask storage systems and ISFSIs provide an equivalent or greater level of conservatism as that claimed for the NPP SRPs.

54. (SFB, FO) NRC staff and PFS claim that the potential consequences of seismic failure of ISFSIs are much less severe than those of NPPs. *See, e.g.* Cornell Dec. ¶ 16. PFS and the Staff further claim that ISFSI facilities are less vulnerable to earthquake-initiated accidents than nuclear power plants. *See* Cornell Dec. ¶ 17. Thus, the SRPs in NUREG 1536 and 1567 may already incorporate less conservatism than NPP SRPs. Additionally, the dry cask storage system SRP design standards are based on the assumption that the design basis earthquake is equivalent to the safe shutdown or deterministic earthquake used for nuclear facilities, under 10 CFR Part 50. NUREG 1536 at 2-10, NUREG-1567 at 7-20, 7-

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[5] The risk reduction ratio is a measure of the conservatisms incorporated into the design of an SSC. The risk reduction ratio must be sufficiently large to show that the target performance goals are achieved. DOE requires a minimum risk reduction ratio of 5 and 10 for PC3 and PC4 SSCs, respectively. DOE-1020-94, Table C-3 at C-5.

54. In sum, SRPs for dry storage cask system and ISFSI may already incorporate less design conservatism than NPP SRPs.

55. (SFB, FO) NUREG 1536 requires the applicant to demonstrate that the dry cask system will not tipover or drop as a result of a credible natural phenomenon event, such as an earthquake. NUREG 1536 at 3-6. As discussed below, the HI-STORM 100 cask system will likely tipover if subject to the peak ground accelerations for a 2,000-year earthquake. Thus, even if the SRPs for NUREG-1536 result in design criteria that are equal or more conservative than posed in DOE-STD-1020, PFS has not shown that at its site it meets the NUREG-1536 SRPs.

56. (SFB, FO) PFS witness, Dr. Cornell, claims that NUREG/CR-6728 provides a "quantitative finding that the [risk reduction ratio] levels for typical systems, structures, and components designed to NRC SRPs are in the range 5 to 20 or greater" (or in the range of the DOE-STD-1020-94 risk reduction ratios). *See* Cornell Dec. at ¶ 25. To support his claim, Dr. Cornell compares the risk reduction factors for nuclear power plant SSCs using both the NRC SRPs and DOE-STD-1020-94. *See* in general Cornell Dec., Attachment A. However, in Att. A, Dr. Cornell relies upon "numerous engineering evaluations of safety margins and 'fragility curves' of SSCs." Id. at 3. NUREG/CR-6728, Chapter 7 contains fragility curves for a variety of NPP sites. *See* 7-10 to 7-15. The fragility curves used in NUREG/CR-6728 are obtained from *Basis for Seismic Provisions of DOE-STD-1020*, R.P Kennedy and S.A. Short (1994). NUREG/CR-6728 at 7-5. It is important to note that the only site with similar peak ground accelerations to the PFS site is the California site located near Santa Maria, *i.e.*, Diablo Canyon. Id. at 7-11, 7-22. In 1994, when Kennedy and Short published the fragility curves, Diablo Canyon did not have dry storage casks, let alone freestanding dry storage casks. *See* Pacific Gas and Electric web page describing the spent nuclear fuel is currently stored in pools (12/05/01), attached hereto as Attachment E. Thus, the Kennedy and Short fragility curves relied upon in NUREG/CR-6728 could not have included unanchored dry storage casks, and Dr. Cornell's attempt to correlate NUREG/CR-6728 to DOE-STD-1020-94 risk reduction ratios in his Declaration, Att. A, again fails with respect to HI-STORM 100 casks at the PFS facility.

57. (FO) Furthermore, and in general, the Kennedy and Short fragility curves do not apply to SSCs such as storage casks sliding on the pads to maintain stability and control for excessive movement and tipping. The fragility curve pertains to inherent strength and ductility of the member and the design code upon which the component was designed. The fragility curve as it pertains to controlled and

stable movement of the casks on the pads has not been developed by PFS, nor any appropriate design code.

58. (SFB, FO) PFS claims:

> risk reduction ratios achieved by the conservatisms in the designs of the IFSF [sic] SSCs are confirmed by analyses recently conduced by PFS contractors and vendors, which show that the facility's SSCs can survive earthquakes with return periods significantly greater than the 2,000 years of the PFSF DBE; specifically, the storage casks and canisters and the CTB have been shown to be able to survive, without loss of safety function, the ground motions of a 10,000 year return period earthquake. Thus, they [SSCs] easily meet or surpass the 10^{-4} performance goal set in DOE 1020.

PFS Motion at 8-9. However, PFS's cask stability witness could not support PFS's claim that the PFS SSCs, including the HI-STORM 100 cask, had risk reduction ratios of at least 5 to 20.[6] Singh/Solar Tr. at 53. The witness further stated that he was not familiar with the term "fragility curves" and therefore, could not have generated fragility curves. Id. at 63. This shows a general lack of understanding and support by the cask designer of a key claim made by PFS. It also shows that the risk reduction ratios or mean component failure return period claimed by PFS are not based on any calculations performed by the cask vendor.

59. (SFB, FO) In our opinion, it is inappropriate to apply generalized risk reduction ratios deemed appropriate for NPPs to the proposed storage pad, unanchored HI-STORM 100 cask, and the CTB. The basis for selecting appropriate risk reduction factors can only adequately be conducted by evaluating a thorough uncertainty analysis of the fragility of each SSC at the PFS site, as outlined in DOE-STD-1020 and NUREG/CR-6728.

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[6] The basis for Dr. Cornell's premise of "mean component failure return period 5-20 times or more greater" (quote from PFS's response to the State's 11th Set of Discovery (Oct. 2, 2001), response to Interrogatory No. 15, item 9, supported by Dr. Cornell) comes from the risk reduction ratios described in DOE-STD-1020. *See* Cornell Dec. ¶¶ 22, 23, 25, and 29.

Probability of Failure of PFS SSCs

60. (SFB, FO) PFS failed to demonstrate adequate conservatism has been applied in the seismic design of foundations for the storage pads and CTB and to the seismic stability of the pads and HI-STORM 100 storage casks for the proposed DBE. There are numerous unconservative assumptions, errors and oversights in the calculations.[7]

61. (SFB, MK, FO) We agree with PFS's definition of a failure "as exceeding a behavior limit state that may preclude the SSC from fulfilling its intended function." Cornell Dec. at 14. Based on this definition, a reduction of its ability to shield radiation, thereby causing an increase in dosage, would be a failure of the HI-STORM 100 cask.

Independent HI-STORM 100 Storage Cask System Stability Analysis

62. (MK) I was hired to evaluate the results of seismic analyses of the Holtec's cask stability (report numbers: HI-971631, HI-2012653, HI-2012640) by independently modeling the sliding and tipover phenomenon of the HI-STORM 100 cask under seismic motion for a 2,000 year earthquake at the PFS site. *See* Analytical Study of HI-STORM 100 Cask System Under High Seismic Condition, Technical Report No. 01141-TR-000, Revision 0 ("Altran Report") attached hereto as Attachment F. To mathematically simulate the cask behavior, I modeled the cask as beam elements. The cask base is connected to the ISFSI pad using non-linear elements. I used the SAP2000 structural analysis code to perform a non-linear time history analysis. My modeling was based on cask design parameters provided in the previously referenced Holtec reports. The SAP2000 results were compared with another general purpose structural analysis code, ANSYS, for a two dimensional model. The results for SAP2000 and ANSYS were identical, which validates the SAP2000 results.

63. (MK) In its cask stability analysis, Holtec models the cask using beam finite elements with lumped masses. Holtec uses an in-house non-linear analysis program to perform dynamic analyses with a lump mass mathematical model. Singh, Soler Tr. at 100. The SAP2000 and ANSYS programs also model a cask

[7]We have identified many of the shortcomings in the calculations in our declarations in support of the State of Utah's Request for Admission of Contention QQ and subsequent modifications thereto. *See* State's Requests dated May 16, 2001, June 19, 2001, and August 23, 2001.

with beam finite elements with lumped masses for nonlinear time history analyses. Both SAP2000 and ANSYS have been benchmarked with known analytical solutions to provide adequate results for dynamic analyses.

64. (MK) I conducted studies using three mathematical cask models with varying degrees of complexity. The purpose of the first model was to obtain horizontal sliding displacements for the HI-STORM 100 cask without any vertical excitation. For this case, two industry standard structural analysis codes, SAP2000 and ANSYS, were used to compare the maximum sliding displacement values and the nodal displacement time history traces for benchmarking purposes.

65. (MK) The second model included the effect of vertical excitation without any rocking effects due to cask height. The third model represented a three-dimensional cask with vertical and horizontal rigid beam finite elements. The mass density of the vertical beam elements are adjusted to obtain approximately the weight of a fully loaded HI-STORM100 cask. For the last two models, several parametric analyses were performed to show the effect of change of contact stiffnesses and coefficient of friction values on the HI-STORM100 cask motion. The effect of the use of various structural damping values (ALFA, BETA, or Modal) was also studied in the third model.

66. (MK) As a result, I determined that the HI-STORM 100 cask dynamic analysis is highly sensitive to the local stiffness values used as input in the mathematical model. Thus, the estimates of potential sliding and rocking displacements are dependent upon the local stiffness values used. However, in reality, the sliding displacement of a cask under seismic ground motion should not be very sensitive to the local contact stiffness values because only frictional forces would dissipate the maximum amount of energy during sliding and rocking of an unanchored cask acting as a rigid body.

67. (MK) Holtec used an initial high local contact vertical stiffness of 454×10^6 lbs/in. (*see* HI-971631, Appendix C). Although high contact stiffness values are generally used in mathematical simulations, the high stiffness values artificially treat the solution as linear without amplifying it in the upward direction and give non-unique or invalid results. A contact stiffness of 454×10^6 lbs/inch is too high for an unanchored cask because the contact stiffness makes the vertical frequency of the cask too rigid, thus artificially reducing the vertical displacement. Based on my experience, it is my opinion that a more appropriate contact stiffness value for unanchored casks is 1×10^6 lbs/inch.

68. (MK) High contact stiffness values also absorb significant amounts of energy before sliding actually occurs. Holtec's use of an initial high local contact vertical stiffness significantly minimizes vertical excitation. In addition, the high values of vertical stiffness used by Holtec also artificially and significantly reduce the estimated horizontal sliding displacements.

69. (MK) Holtec assumed BETA damping coefficients corresponding to 5% structural damping. Singh/Soler Tr. at 100. This is a high damping value. In reality, the structural damping would be small or insignificant for a rigid cask, and only friction should be the primary energy dissipation mechanism. Holtec's use of high BETA damping coefficients also underestimates potential sliding and rocking displacements that may lead to cask collision or tipover during a seismic event.

70. (MK, FO) The Altran analysis did not account for the amplification due to soil structure interaction in the 2,000-year earthquake input time histories. The soil structure interaction of the pad amplifies the time history response on top of the storage pad. Therefore, the vertical input motions at the base of the cask would be much higher than those applied in this study. Because the vertical response alters the cask dynamic reactions significantly, the computed cask reactions during the cask uplift and drop and subsequent impact due to vertical amplifications would be much higher than those used in PFS's pad design.

71. (MK) The sensitivity of a non-linear finite element model, such as Holtec's, to the local vertical stiffness values demonstrates that Holtec's model does not produce a unique or conservative solution. Hence, the cask stability results from Holtec's non-linear model stated in HI-951312, HI-2012653, and HI-951312 are not conservative and would lead to an unconservative pad design.

72. (MK) In my opinion, the only way to validate Holtec's analyses is for Holtec to benchmark its sliding displacements calculated by Holtec's non-linear mathematical model with actual shake table test data.[8] This is common practice in the seismic performance field. I frequently perform shake table tests to benchmark mathematical models. In fact, NRC requires shake table tests in IEEE

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[8]The results of a cask stability analysis using Holtec's mathematical model have not been compared to other computer codes. Singh/Soler Tr. at 93-94. PFS's witness stated that Holtec's mathematical model was compared with other programs to support wet storage applications. Id. There is available data for HI-STORM 100 to calibrate Holtec's model. Id. at 95.

Std 344-1987 (Institute of Electrical and Electronics Engineers, Inc. *Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations*) for many components and equipment, *e.g.*, electrical pumps.[9] Shake table testing is required to demonstrate the structural integrity and functionality of equipment. It is also used for initial correlation with mathematical models to predict accurate dynamic responses.

73. (MK) In the past, I have always relied upon test data using shake tables to calibrate my seismic analysis models. In my opinion, Holtec must conduct shake tests to calibrate its model. Several facilities are available that may accommodate a HI-STORM 100 cask.

74. (MK) In summary, Holtec's analysis significantly underestimate the potential for the HI-STORM 100 cask system to slide, uplift, and tipover. Additionally, without actual test data, Holtec's model generates unconservative and non-unique results. I acknowledge that my results have also not been calibrated with actual test data. However, I believe the range of contact stiffnesses used in my analyses includes realistic assumptions for vertical contact stiffness and damping. So, under the peak ground accelerations for a 2,000-year annual return period earthquake at the PFS site, my analyses using a realistic vertical contact stiffness value of 1×10^6, coefficient of friction of 0.8, and 0.01% damping estimate the HI-STORM 100 cask displacement could be as high as 372.76 inches horizontally in the x direction, 229.65 inches horizontally in the y direction, and 27.24 inches

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[9]NRC Reg. Guide 1.100, Rev. 2, *Seismic Qualification of Electric and Mechanical Equipment for Nuclear Power Plants* endorses use of the IEEE standard 344-1987 (Revision of ANSI/IEEE Std 344-75) for equipment qualification. The introductory note of Section 6, Analysis, of IEEE-344-87 states that "[t]he analysis method is not recommended for complex equipment that cannot be modeled to adequately predict its response. Analysis without testing may be acceptable only if structural integrity alone can ensure the design-intended function." Section 5.3.1 states that "[i]n analysis, a mathematical model is made of the equipment so as to predict the response to the seismic motion. The damping used in this model should correspond to the actual energy dissipation in the equipment to enable the response to be accurately predicted. An alternate approach is to use a conservative value of linear damping to obtain a conservative estimate of response. In any case, there is a need to know the ranges of damping for the specific equipment and the nature of nonlinearities and their effect on the response. Appropriate values of damping may be obtained from tests or other justifiable sources."

vertically.[10], [11] Thus, in my judgment, from the results of the Altran study, the HI-STORM 100 cask could tipover or collide with other casks if subject to the peak ground accelerations for a 2,000-year earthquake at the PFS site.

75. (SFB, MK, FO) The uplift, sliding, and tipover of the HI-STORM 100 cask are not surprising given the ground motions at the PFS site. Actual earthquake experience has shown objects with masses similar to a loaded HI-STORM 100 cask have been moved under ground motions that are similarly predicted for the PFS site. *See* copy of slides, as annotated by Dr. Bartlett, attached hereto as Attachment G.

PFS Calculations Not Conservative

76. (SFB, FO) In arguing that a 2,000-year DBE is appropriate, PFS claims that its SSCs are capable of withstanding a "more severe earthquake" than a 2,000-year DBE. PFS Motion at 16-18. PFS failed to demonstrate adequate conservatism in its seismic design of foundations for the storage pads, CTB, and in its seismic stability analysis of the pads, and HI-STORM 100 storage cask system for the proposed 2,000-year DBE. For example, PFS's stability analyses rely on seismic loads generated by simple assumptions that are inadequate and unconservative. Also, PFS made major errors and oversights in its application of the seismic loads in the stability analysis which essentially invalidate the conclusions. Many of these concerns have been previously raised.[12] The inadequacies of PFS's supporting seismic analyses are discussed below because they are intrinsically tied to ensuring adequate conservatism in the SSC design and the ultimate selection of an appropriate DBE at the PFS site.

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[10]Note that the time histories used in this analyses were obtained from a PFS document, *Development of Design Basis Ground Motions for the Private Fuel Storage Facility, Revision 1,* Geomatrix Consultants, Inc. (March 2001). Thus, the lack of conservative assumptions discussed in this joint declaration have not been remedied in Altran's analysis.

[11]The HI-STORM 100 cask can only withstand a drop of less than one foot and stay within the CoC limits. Singh/Soler Tr. at 102-103.

[12] The State filed Utah Contention QQ to address these numerous unconservative assumptions, errors and oversights in PFS's revised foundation design calculations. *See* footnote 7 above.

77. (SFB) In the spring of 2001, PFS again revised its design basis response spectra (hereinafter referred to as the "2001 revision"). The 2001 revision was needed to correct errors and inconsistencies used in the development of the previous design basis ground motion. The 2001 revision significantly increased the design basis ground motion. *See* State's Request for Admission of Late-filed Utah QQ (seismic stability) at 3-4 (May 16, 2001) and references cited therein. Previous estimates of the peak horizontal and vertical ground acceleration for a 2,000-year earthquake were about 0.5 g. Id. The 2001 revised peak ground accelerations for a 2,000-year earthquake increased to 0.711 and 0.695 g in the horizontal and vertical direction, respectively. SAR at 2.6-107, Rev. 22. These increases in ground motion place a significantly larger demand on the foundation systems.

78. (SFB, FO) NUREG-75/087 requires a 1.1 minimum factor of safety against sliding, bearing capacity failure, and overturning for foundations systems at the proposed DBE. *See* NUREG-75/087, Section 3.5.5, "Foundation," and Section II.5, "Structural Acceptance Criteria." Failure of the foundation systems of an SSC can lead to unacceptable SSC performance. In this case, failure of the pad foundation system may result in excessive sliding and overturning of the casks. Thus, the seismic performance of the foundation system is inextricably linked to the performance of the SSC and cannot be decoupled from the evaluation of the SSC. Adequate assessment of the seismic loading (*i.e.*, demand) and the capacity of the SSCs and their foundations to resist seismic loadings is necessary to calculate the fragility of a particular SSC. Thus, the fragility of the SSC must also consider the fragility of the SSC's foundation system.[13] *See* ¶¶ 36-49 above for discussion of importance of SSC fragility determination in establishing DBE. Hence, PFS's seismic calculations for the CTB and pad foundations must be considered in determining the probability of failure of the PFS SSCs.

79. (FO) The stability of the cask is dependent, in part, upon the response of the pad foundation to resist seismic loads and assumptions in calculating the seismic loads. PFS has not considered the range of applicable phasing of the foundation pad motion and the casks motion, the actual interface conditions between the casks and the pad on cement-treated soil, and the applicable wide range of phasing relationship in input time histories and types of earthquake waves striking the pads.

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[13] DOE-STD-1020 and NUREG/CR-6728 both require the fragility calculations to consider potential variations in the demand placed on the SSC and the capacity of the SSC to resist the seismic demand. DOE-STD-1020, App. C; NUREG/CR 6728, Ch. 7.

a. Since the analysis of the cask-pad-cement-treated soil is a nonlinear analysis, it is very important to consider all potential variation in the motion of the pad and the casks. If the pads and the casks move out of phase, significant instability conditions arise. The soil spring and damping used in the Holtec Report No. HI-2012640 titled *Multi Cask Response at PFS ISFSI from 2000-Yr Seismic Event*, Rev. 2 (Aug. 20, 2001) do not properly consider the frequency dependency of these parameters.[14] Holtec provided no check to compare the parameters used by other available rigorous solutions to ensure the foundation parameters are reasonably accurate.[15] The contrast in the dynamic properties of the underlying stratum has significantly increased by inclusion of cement-treated soil in the foundation, thus increasing the concern on frequency dependency of soil spring and damping.

b. Because the PFS site is located close to a set of major faults dipping under the site (*see Development of Design Basis Ground Motions for the Private Fuel Storage Facility*, Rev. 1, March 2001, Geomatrix), seismic waves arriving at foundation structures are not necessarily vertically propagating waves. Based on my experience and the literature, there is a distinct possibility that the waves may come at an angle, and waves at an angle tend to cause larger rocking and torsional vibration above and beyond what is captured by the assumption that the waves will be vertically propagating. This is contrary to assumptions in HI-2012640 for the storage pad and Calculation No. 05996.02-SC-5, Rev. 2, *Seismic Analysis of Canister Transfer Building* (SWEC April 4, 2001), and Calculation No. 05996.02-SC-4, Rev. 2, *Development of Soil Impedance Functions for Canister Transfer Building* (SWEC Mar. 21, 2001) for the CTB. Waves striking at angles will cause additional rocking and torsional motion of the

[14]Note that calculations performed for the foundation spring and damping coefficients for the CTB recognize these coefficients are a function of frequencies and show that they are highly frequency-dependent.

[15] In the calculation of soil spring and damping for the CTB (Calc. No. 05996.02-SC-4, Rev. 2, *Development of Soil Impedance Functions for Canister Transfer Building*, Mar. 21,2001, SWEC), it has been shown that the soil spring and damping are highly dependent on the frequency due to soil layering at the site.

foundation above and beyond the motion caused by vertically propagating waves.[16]

c. The nonlinear analysis in HI-2012640 is sensitive to phasing of the input motion and thus multiple time histories should be used. Only one set of time histories, developed by Geomatrix (Calc. No. 05996.02F(PO18)-3, Rev. 1, *Development of Time Histories for 2000-year return period design spectra*, Mar. 21, 2001), has been used by Holtec. Based on my experience, the common industry practice for nonlinear calculations is to use at least three sets of time histories because the nonlinear analysis is sensitive to phasing. In order to cover the variation of the phasing in the design, a minimum of three (or sometimes four) time histories are used. This is an important safety consideration that PFS has failed to address.

d. In developing the time histories, PFS's efforts failed in part because PFS did not look at the entire design package, in particular the nonlinear analysis of the casks on the pad (prepared by Holtec) and impacts of pulses caused by "fling."[17] *See* Geomatrix Calc. No. 05996.02F(PO18)-3. Such pulses could be symmetric, asymmetric, one-sided or two-sided. PFS's failure to give adequate consideration to the variation in ground motion may have a significant impact on seismic loading of the foundations and the design.

e. Holtec has assumed that the casks will slide on the pad in a controlled manner during a large earthquake. There is no other redundancy built into Holtec's expected design. But such a bold assumption is negated by the potential that cold bonding between the cask and the pad may occur over time. When two bodies (cask and pad) with such a large load (the cask) are in contact, some local deformation and redistribution of stresses may occur at the points of contact which would create a bond, and this would

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[16]Although he performed the HI-STORM 100 cask stability analysis, PFS's witness acknowledged that he had no expertise to determine whether the angle of earthquake waves hitting the cask would cause additional rocking and torsional motion. Singh/Soler Tr. at 98-99.

[17]Geomatrix recognized that "fling" pulses should be included in the time history but PFS has no parametric study in the design package to reveal the impact of the pulses on the design.

not allow the cask to slide on the pad or move smoothly during an earthquake.

80. (SFB) PFS has not provided a realistic evaluation of the foundation pad motion with cement-treated soil under and around the pads in relation to motion of the casks sliding on the pads. The actual load path under seismic loading has not been adequately addressed. In order to evaluate the adequacy of the foundation design and the bearing soil, it is critical to understand the nature of the loads and where they are coming from.

81. (FO) In the stability analysis of the pads, the calculation *Stability Analysis of Storage Pads*, Calculation No. 05996.02, G(B)-04, Rev. 7 (SWEC), PFS has failed to consider the natural frequency of the cask-pad-cement-treated soil system, thus underestimating the seismic loads significantly. In addition, the actual load path under seismic loading has not been considered. While it has been shown that the effect of soil-structure interaction is important in seismic response of the cask-pad-cement-treated soil system, the effect of pad-to-pad interaction only five feet apart in the longitudinal direction has been ignored. In the stability analysis, the passive resistance for one pad will act as a pushing force on the next pad. This interaction has been totally ignored in the evaluation, thus seriously invalidating the conclusion of the stability of the pads. In the continuation of the stability analysis, a row of ten pads has been considered. PFS ignored the fact that cement-treated soil has limited capacity under tensile and bending stresses and cannot behave as a reinforced concrete mat. The cracking caused by out-of-phase motion of the pads and the cement-treated soil, and the other impacts of striking seismic waves prevent the cement-treated soil pad for ten rows of the pads to act as an integrated unit. Furthermore, on Page 2.6-50 of the SAR Rev. 17, the estimated static settlement for the pads is shown to be 3.3 inches. The differential settlement between the pad and the surrounding cement-treated soil cause bending and cracking of the cement-treated soil propagating away from the pad. This condition invalidates assumption of an integrated foundation for ten rows of pads and also negates the validity of the passive pressure used in the stability analysis of the individual pads.

82. (FO) PFS used incorrect rigidity assumptions in the calculation of the dynamic forces acting upon the CTB mat (*see* Calculation Nos. 05996.02-SC-5, 05996.02-SC-4) and storage pad foundations (*see* HI-2012640). Based on the results reported in *Storage Pad Analysis and Design*, Calc. No. 0599602-G(PO17)-2, Rev. 3, 4/5/01, ICEC), this assumption is not valid and results in erroneous calculation of the foundation damping and also violates the assumption that the

coefficient of friction between the casks and the pad is necessarily constant[18]. The assumption of rigidity leads to an overestimation of the foundation damping and an underestimation of the seismic loads.

83. (FO) PFS has failed to analyze the dynamic interaction of the cement-treated soil with the CTB mat foundation and the storage pad foundations. In the case of the CTB foundation, the influence of the large cement-treated soil mass around the building has been ignored. *See* Calculation Nos. 05996.02-SC-5 and 05996.02-SC-4. Also, the presence of a stiff, cement-treated soil perimeter around the CTB of about one building dimension impacts the soil impedance parameters and kinematic motion of the mat foundation.

84. (FO) In the drop/tipover analysis of the casks (*PFSF Site-Specific HI-STORM Drop/Tipover Analyses*, Rev. 0 and Rev. 1, Holtec Report No. HI-2012653, Apr. 3, and May 7, 2001 respectively), Holtec assumed a lower stiffness of the cement-treated soil under the pad to meet the drop/tipover condition. In doing so, it has failed to recognize the difference between the static and dynamic modulus of the cement-treated soil and the effect of significant temporal and spatial change in bearing pressure acting on the cement-treated soil. The expected large difference between the static and dynamic modulus invalidates the assumption made for the design.

85. (SFB) PFS has not substantiated the use of passive earth pressure to resist earthquake loadings. The passive earth pressure is an additional resisting force assumed to be present due to the use of cement-treated soil in the foundation design. PFS has not supported the use of passive earth pressure resulting from the cement-treated soil by the requisite engineering calculations and testing. Without the resisting passive earth pressure resisting force (*i.e.*, buttress effect), PFS cannot demonstrate adequate resistance to dynamic sliding of the CTB and storage pads. However, the Applicant has failed to demonstrate that the proposed cement-treated soil buttress will not simply crack and be rendered ineffective during a seismic event. For the case of the CTB, the Applicant has not considered the deleterious effects of separation and cracking of the cement-treated soil buttress

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[18]PFS's witness had no opinion on whether damping would be overestimated assuming the CTB mat is rigid. Ebbeson Tr. at 48. Additionally, although calculation 05996.02 SC-6, *Finite element analysis of Canister Transfer Building*, Rev. 0 (SWEC), considered the flexibility of the mat and PFS's witness was familiar with the calculation, he did not consider whether the results of SC-6 supported his assumption that the CTB base mat is rigid. Id. at 49.

caused by out-of-phase motion of the CTB mat foundation and the cement-treated soil buttress. For the case of the storage pads, the Applicant has not considered the deleterious effects of separation and cracking of the cement-treated soil caused by out-of-phase motion and pad-to-pad interaction forces resulting from closely spaced pads. For both the CTB and storage pad foundations, the Applicant has not calculated the bending and tensile stresses that will develop in the cement-treated soil and how these stresses will affect the ability of the cement-treated soil buttress to resist these forces without cracking or separation.

86. (SFB) Foundation stability calculations, such as Calculation No. G(B)04, assume that the maximum inertial force transmitted to the foundation system and soils cannot exceed the friction force between the bottom of the cask and the top of the concrete pad. Based on an assumed upper limit of the coefficient of sliding friction of 0.8, the maximum inertial force transmitted to the foundation system has been limited in the design of the foundation to 0.8 times the combination of the static and dynamic normal forces. This assumption may not represent the upper bound for dynamic loading, and inertial forces may larger than those used by PFS.

87. (FO) In its hypothetical non-mechanistic tipover analysis, Holtec incorrectly assumes an initial angular velocity of essentially zero ("infinitesimally small motion"). Singh/Soler Tr. at 74-75. However, because of the potential for excessive sliding, uplift, collision and tipping of HI-STORM 100 cask shown by the Altran Report (*see* ¶ 74 above and Attachment F) when subject to a 2,000-year earthquake, the angular velocity of the cask will be greater than zero.[19]

88. (SFB, FO) In summary, PFS's underestimation of seismic demand, the overestimation of damping, the failure to analyze the interaction of the foundations with the cement-treated soil, and the potential for cracking of the cement-treated soil are serious oversights. These oversights, unconservative assumptions and errors create unacceptable uncertainties in the estimation of the true seismic demand and its potential impacts to foundation and cask stability. Without a proper assessment of these issues, it is impossible to determine the fragility of the PFS SSCs and determine if the target performance goal has been met, as required by DOE-STD-1020.

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[19]PFS's cask stability witness acknowledged that even under Holtec's 2,000-year earthquake cask stability analysis the bottom of the cask would lift up or rock from the pad. Singh/Soler Tr. at 73, 74.

89. (SFB, MK, FO) Further, independent cask stability calculations, the Altran Report, as discussed in this declaration, show that excessive sliding, uplift, collision and tipping are possible for the unanchored HI-STORM 100 storage casks under the earthquake motion of the proposed DBE. *See* ¶¶ 62-75 above. The Altran Report nonlinear analysis shows that if realistic and applicable range of interface parameters are considered, the casks will be subjected to severe dislocation, lift off and tipping. The Altran Report is further supported by case history examples of sliding and tipping of large, heavy objects during strong ground motion. *See* ¶ 75 above. These independent evaluations of the cask stability clearly demonstrate that the performance goal of 1×10^{-4} as it relates to cask stability and controlled behavior has not been achieved at the PFS site under the 2000-year earthquake.

Dated this 7th day of December 2001.

By Steven Bartlett

Steven F. Bartlett, Ph.D., P.E.

By: ______________________

Farhang Ostadan, Ph.D., P.E.

By: ______________________

Mohsin Khan, Ph.D., P.E.

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By: ______________________________
Mohsin Khan, Ph.D., P.E.

89. (SFB, MK, FO) Further, independent cask stability calculations, the Altran Report, as discussed in this declaration, show that excessive sliding, uplift, collision and tipping are possible for the unanchored HI-STORM 100 storage casks under the earthquake motion of the proposed DBE. *See* ¶¶ 62-75 above. The Altran Report nonlinear analysis shows that if realistic and applicable range of interface parameters are considered, the casks will be subjected to severe dislocation, lift off and tipping. The Altran Report is further supported by case history examples of sliding and tipping of large, heavy objects during strong ground motion. *See* ¶ 75 above. These independent evaluations of the cask stability clearly demonstrate that the performance goal of 1×10^{-4} as it relates to cask stability and controlled behavior has not been achieved at the PFS site under the 2000-year earthquake.

Dated this 7th day of December 2001.

By:

Steven F. Bartlett, Ph.D., P.E.

By:

Farhang Ostadan, Ph.D., P.E.

By:

Mohsin Khan, Ph.D., P.E.

A

Steven F. Bartlett, Ph.D., P.E.
Resume

Areas of Research

Geotechnical Earthquake Engineering
Ground Response Modeling
Geotechnical Instrumentation
Site Characterization
Behavior of Soft Soils
Risk Assessment
Hazard Mapping

Areas of Expertise

Geotechnical Engineering
Earthquake Engineering
Transportation Engineering
Geoenvironmental Engineering
Applied Statistics
Project Management

Education

Ph.D., Civil Engineering (geotechnical emphasis), Brigham Young University, 1992.

B.S., Geology, Brigham Young University, 1983.

Professional History

Assistant Professor, Civil and Environmental Engineering Department, University of Utah, 2000-current.

Adjunct Assistant Professor, Brigham Young University, Department of Civil and Environmental Engineering, Brigham Young University, 2001.

Instructor, Civil and Environmental Engineering Department, University of Utah,

Utah Dept. of Transportation, Research Project Manager, Research Division, 1998 - 2000.

Woodward-Clyde Consultants, Project Engineer, 1996-1998.

Westinghouse Savannah River Company, Senior Engineer, 1991-1995.

Professional Experience

- **Assistant Professor, Civil and Environmental Engineering** - Teaching of graduate and undergraduate courses in geotechnical engineering and performing research.
- **Research Project Manager, I-15 Reconstruction Testbed** - UDOT Project manager for I-15 research involving construction and instrumentation of innovative embankment systems, foundation treatments and ground modification; long-term settlement monitoring and performance of embankments, mechanically stabilized earth walls, geofoam fills, etc.; response of pile and geopier foundation systems to lateral and uplift loads; carbon fiber retrofitting and non-destructive testing of bridges.
- **I-15 Design-Build Project Geotechnical Designer** - Design engineer for Woodward-Clyde Consultants responsible for geotechnical design from 800 South to 2100 South of I-15 in Salt Lake City, Utah. Design included foundation treatments, ground modification, slope stability, settlement considerations, geofoam fills, liquefaction assessments, and seismic modeling of embankment and MSE wall systems.
- **Value Engineering and Design Team** - Geotechnical member of the Value Engineering and Concept Design Team for the University Parkway Interchange (1300 South) at I-15, Orem, Utah.
- **Private Fuels Storage Facility** - Geotechnical expert witness for the State of Utah in the proceedings before the Nuclear Regulatory Commission for thePrivate Fuel Storage, LLC proposed interim high-level radioactive waste storage facility. Lead reviewer of the safety analysis report (SAR) and supporting calculations for geotechnical investigations, Skull Valley, Utah.
- **Kennecott Utah Copper Tailing Impoundment Modernization Project** - Performed steady state and transient seepage analyses for dewatering system for the upgrade and expansion of Kennecott's tailings impoundment, Magna, Utah.

Brigham Young University, Research Assistant, 1988-1991.

Utah Department of Transportation, Preconstruction Materials Engineer, 1987-1988.

Utah Department of Transportation, Construction Technician, 1984-1987.

Geokinetics In-Situ Oil Shale Development, Retort Engineer and Technical Writer, 1984.

Awards and Recognitions

BYU Presidential Scholar (University Scholarship).

Alvin Barrett Scholar (Geology Department).

Civil Engineering Departmental Scholar.

BYU Scientific Research Society (Sigma-Chi) Recipient, Outstanding Ph.D. Dissertation, 1992.

Total Quality Achievement Award, Environmental Restoration Department, Westinghouse Savannah River Company, 1992, 1993.

Finalist for outstanding paper, ASCE Journal Geotechnical Engineering, 1995.

Vice President's Award, Westinghouse Engineering and Construction Services Division, 1995.

Excellence in Research Award, Utah Dept. of Transportation, 1999, 2000.

Registrations

Professional Engineer: Utah.

Affiliations

American Society of Civil Engineers.

Transportation Research Board.

National Council of Examiners for Engineering and Surveying.

American Society of Engineering Educators?

Boy Scouts of America.

- **Wasatch County Water Efficient Project** - Performed geologic and geotechnical assessments of canal stability and pump station locations, Heber Valley, Utah.
- **Bear River Pipeline** - Performed geologic and geotechnical assessments of pipeline route alternatives for the Salt Lake Water Conservancy District, Weber, Davis and Salt Lake Counties, Utah.
- **Cainville Dam Investigation** - Project Engineer responsible for preliminary geologic and geotechnical assessments of foundation conditions at this proposed dam site. Performed drilling of abutment areas, pump testing, and seepage assessments, Wayne County, Utah.
- **DMAD and Gunnison Bend Dam Investigations** - Performed geotechnical investigations and assessments to determine the piping potential and seismic stability of these embankment dams for the State of Utah, Dam Safety Program, Delta, Utah.
- **Seismic Retrofit of Salt Lake City Waste Water Treatment Plant** - Lead geotechnical design engineer and field oversight engineer of jet grouting operations to stabilize potentially liquefiable soils under an effluent pump station, North Salt Lake City, Utah.
- **Hurricane Bridge Foundation Investigation** - Performed geologic and bridge foundation investigations and analyses for UDOT, Hurricane Bridge Crossing, Hurricane, Utah.
- **ITP/H-Area Tank Farm Geotechnical Investigation and Seismic Qualification, Department of Energy, Savannah River Site** - Westinghouse's principal geotechnical investigator on a multi-disciplinary team overseeing the seismic qualification of the ITP/H-Area high-level radioactive waste storage tank farm. This project included extensive subsurface investigations, strong ground motion modeling, probabilistic liquefaction hazard evaluations, dynamic settlement and slope stability calculations, and risk assessment.
- **Review Team for the Seismic Design of the Defense Waste Processing Facility, Department of Energy Savannah River Site** -

Committees and Panels

Chairman of Utah Strong Motion Advisory Committee, 2001-current.

Member of Transportation Research Board, Committee on Soils and Rock Instrumentation, 2000-current.

Member of Utah Seismic Safety Commission Lifelines Subcommittee, 1998-current.

Program Committee Chair, EPS Geofoam 2001 3rd International Conference, Salt Lake City, December 10-12, 2001.

Member of Organizing Committee, Geologic Hazards in Utah, Salt Lake City, Utah, April 12-13, 2001.

Member of FEMA Project Impact and Salt Lake City Seismic Hazard Ordinance Committee, 2000.

Member of Organizing Committee, 34th Annual Symposium on Engineering Geology and Geotechnical Engineering, Logan, Utah, April, 1999.

Member of Organizing Committee, Environmental Geotechnology, ASCE, Salt Lake City, Utah, March, 1997.

Member of Municipal Landfill Site Selection Committee, Columbia County, Georgia, 1993.

Training and Certifications

OSHA 1910.120 Health and Safety Training for Hazardous Waste Operations and Emergency Response.

Department of Energy, Radiation Worker Training, Westinghouse Savannah River Company.

U.S. Department of Labor Mine Safety and Health Administration (MSHA) Underground Mining Training.

Peer Reviewed Publications and Reports

Bartlett S. F., and Farnsworth, C. "Performance of Lime Cement Stabilized Soils for the I-15 Reconstruction Project, Salt Lake City, Utah, "Transportation Research Board Annual Meeting, Jan. 2002, Washington, D.C. (in press).

Bartlett S. F., Farnsworth, C., Negussey, D., and Stuedlein, A. W., 2001, "Instrumentation and Long-

Westinghouse's principal geotechnical investigator reviewing the Safety Analysis Report (SAR) for the seismic qualification and start-up of this high-level radioactive waste vitrification and storage facility, Savannah River Site, Aiken, South Carolina.

- **Department of Energy Savannah River Site Hazardous Waste Landfill Closure** -Project manager and lead design engineer for the RCRA Facility Investigation and closure of a 51-acre hazardous waste landfill. Also, oversaw the preparation of CERCLA feasibility study for the same closure, Savannah River Site, Aiken South Carolina.
- **RCRA/CERCLA Investigations** - Project Manager for hazardous waste investigations at the Bingham Pump Outage Pits, Burma Road Rubble Pits, and H-Area Retention Ponds, Savannah River Site, Aiken, South Carolina.
- **UDOT Region 2 Preconstruction Materials Engineer** - Performed material testing and pavement design for highway alignment and urban interchanges in West Valley City and the I-215 interchange at California Avenue. Evaluated compaction and quality of subgrade for east-side I-215 between 2700 South and 4500 South. Conducted geologic investigations on new and existing highway alignments in Salt Lake and Wasatch Counties, located fill and gravel sources for construction. Instrumented and monitored I-215 fill slopes for settlement and slope stability.
- **Construction/Survey Technician** - Survey of highway projects and construction inspection. Development of construction project accounting system for UDOT.
- **Retort Engineer** - Monitored process control of underground retorting of oil shale for Geokinetics under Syn-Fuels research contracts for the Department of Energy, Vernal, Utah.

Research and Educational Experience

- **Development of Design ResponseSpectra for Soft Soil Site from Probabilistic Based Bedrock Specta** - Principal Investigator, Utah

Term Monitoring of Geofoam Embankments, I-15 Reconstruction Project, Salt Lake City, Utah," Proceedings of EPS 2001, (in press).

Youd, T.L., Hansen, C.M., Bartlett S.F., 2001, "Revised MLR Equations for Prediction of Lateral Spread Displacement," Journal of Geotechnical (in press).

Bartlett, S.F., Monley, G., Soderborg, A., Palmer, A., 2001, "Instrumentation and Construction Performance Monitoring for the I-15 Reconstruction Project, Salt Lake City, Utah," Transportation Research Board Annual Meeting, Jan. 2001, Washington, D.C.

Bartlett, S. F., Negussey, D., Kimball, M., 2000, "Design and Use of Geofoam on the I-15 Reconstruction Project," Transportation Research Board Annual Meeting, Jan. 2000, Washington, D.C.

Bartlett, S. F. and Youd, T. L., April 1995, "Empirical Prediction of Liquefaction-Induced Lateral Spread," Journal of Geotechnical Engineering, ASCE.

Bartlett, S. F., 1992, "Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads," Ph.D. dissertation and report published by National Earthquake Engineering Research Center, NCEER Report #92-0021.

Bartlett, S. F. and Youd, T. L., 1992, "Case Histories of Lateral Spreads from the 1964 Alaska Earthquake," NCEER Report #92-0002.

Other Publications and Reports

Saye, S. R., Esrig, M. I., Williams, J. L., Pilz J., Bartlett S.F., "Lime Cement Columns for the Reconstruction of Interstate 15 in Salt Lake City, Utah." ASCE Geo-Odessey, Blacksburg, VA., June 10 - 13th, 2001.

Bartlett, S. F., 1999, "Research Initiatives for Monitoring Long Term Performance of I-15 Embankments, Salt Lake City, Utah," 34th Annual Symposium on Engineering Geology and Geotechnical Engineering, Logan, Utah, April, 1999.

Youd, T. L., Hansen C. M., Bartlett, S. F., 1999, "Improved MLR Model for Predicting Lateral Spread Displacement," 7th US-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, Seattle, Washington, August, 1999.

Department of Transportation (2000-2001).

- **I-15 Long Term Monitoring of Embankments and Innovative Foundation Treatments** - Principal Investigator, Utah Department of Transportation (1998 - 2008).
- **Deformation and Modeling of MSE Wall Behavior** - Co-Principal Investigator, Utah Department of Transportation and Utah State University (1999-2000).
- **Evaluation of Properties and Long-Term Performance of Geofoam Fills** - Co-Principal Investigator, Utah Department of Transportation and Syracuse University (1998-2000).
- **Geostatistical Assessment of In-Situ and Engineering Properties at H-Tank Farm** - Co-Principal Investigator, Westinghouse Savannah River Company and Georgia Institute of Technology (1994 - 1995).
- **Evaluation of Geopiers and Pile Foundation to Lateral and Uplift Loads** - Project Manager, Utah Department of Transportation, University of Utah, and Brigham Young University (1999).
- **Design, Application, and Use of Carbon-Fiber Composites in Bridge Repair and Seismic Retrofitting** - Project Manager, Utah Department of Transportation and University of Utah (1998-2000).
- **Use of Forced Vibration Testing to Assess Bridge Damage** - Project Manager, Utah Department of Transportation and Utah State University (1998).
- **Identification and Ranking of UDOT Lifelines** - Project Manager, Utah Department of Transportation (1998 - 2000).
- **Wick Drain Performance** - Project Manager, Utah Department Transportation (1998 - 1999).
- **Assessment of Dynamic Soil Properties for the Savannah River Site** - Geotechnical Reviewer, Westinghouse Savannah River Company and University of Texas at Austin.
- **Research Assistant, Brigham Young University,** "Empirical Prediction of Liquefaction-Induced Lateral Spread," U.S. Army Corps of Engineers and National Center

Simon, D. B., Shlemon, R .J., and Bartlett, S.F., 1999, "Holocene Ground Failure in Downtown Salt Lake City, Utah," Geological Society of America, Cordilleran Section, Vol. 31, Number 6, Berkeley, California, May 1999.

WSRC, 1995, "In-Tank Precipitation Facility (ITP) and H-Tank Farm (HTF) Geotechnical Report," Report No. WSRC-TR-95-0057, Rev. 0, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., 1995, "Probabilistic Liquefaction Settlement Evaluation for the In-Tank Precipitation Facility (ITP)," Report No. C-CLC-H-00815, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., 1995, "Geotechnical Seismic Assessment Report for the Defense Waste Processing Facility (DWPF)," Report No. SRC-TR-95-0072, Westinghouse Savannah River Company, Aiken, S.C.

Rouhani, S., Lin, Y. P., and Bartlett, S. F., 1995, "H-Area/ITP Geostatistical Assessment of In-Situ and Engineering Properties," Final Technical Report, ERDA Project No. 93044, Site Geotechnical Services, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., 1994, "Determination of Soft Zones and Consolidation Properties for the Santee Formation," Report No. K-CLC-H-00058, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F. and Youd, T. L., 1993, "Prediction of Liquefaction-Induced Ground Displacement Near Bridges," Proceedings from the U.S. National Earthquake Conference, Memphis, Tenn., May, 1993.

Bartlett, S. F., 1993, "RCRA Facility Investigation / CERCLA Remedial Investigation for the Burma Road Rubble Pit," Environmental Restoration Department, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., McMullin, S. R., and Serrato, M., 1993, "State of the Art Design: A Closure System for the Largest Hazardous Waste Landfill at the Savannah River Site," Proceedings of Waste Management '93 Symposium.

Bartlett, S. F. and Youd, T. L., 1992, "Empirical Prediction of Lateral Spread Displacement," Proceedings of 4th Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, May, 1992.

Bartlett, S. F. and Youd, T. L., 1990, "Evaluation of

for Earthquake Engineering Research (1988-1991).

- **Thesis Committee Member**, Kiehl, S.J. "Distribution of Ground Displacements and Strains Induced by Lateral Spread During the 1964 Niigata Earthquake, Brigham Young University (1996).
- **Thesis Committee Member**, Hansen C. M, "Improved MLR Model for Predicting Lateral Spread Displacement, Brigham Young University (1999).

Teaching Experience

- **Assistant Professor**, University of Utah, Fall 2000 to current.
- **Teaching Assistant**, Earthquake Engineering, Brigham Young University, Winter Semester, 1989.
- **Teaching Assistant**, Soil Mechanics, Brigham Young University, Fall Semester, 1989.
- **Teaching Assistant**, Field and Laboratory Testing of Soil, Brigham Young University, Spring Term, 1989.
- **Missionary** - Church of Jesus Christ of Latter-Day Saints, Catania, Italy, 1979 - 1981.

Graduate Courses Taught

- CVEEN 7330 Geotechnical Earthquake Engineering (1 time)
- CVEEN 7340 Advanced Geotechnical Testing

Undergraduate Courses Taught

- CVEEN 3310 Geotechnical Engineering I (2 times)
- CVEEN 3320 Geotechnical Engineering II (2 times)

Papers Reviews

Ground Failure Displacement Associated with Soil Liquefaction: Compilation of Case Histories," Miscellaneous Paper S-73-1, U.S. Army Corps of Engineers.

Invited Lectures

"UDOT Guidance for Developing Design Response Spectra for Soft Soils," Geologic Hazards in Utah, Sponsored by AEG and ASCE, Salt Lake City, Utah, April 12 -13, 2001.

"Instrumentation and Research of Geofoam Embankments for the I-15 Reconstruction," Huntsman Chemical Geofoam Seminar"May 16th, 2000, Salt Lake City, Utah

"Design of Geofoam Embankment for the I-15 Reconstruction," Conference on Application and Design of Expanded Polystrene, Sponsored by Taiwan Area National Expressway Engineering Bureau and China Engineering Consultants, Inc., March 3rd, 2000, Taipei, Taiwan.

"Issues Related to the Seismic Design of I-15 Reconstruction Project - A Geotechnical Perspective," Association of Engineering Geologist 42nd Annual Meetings, Sept. 28, 1999, Salt Lake City, Utah.

"Assessment of the Hazard Potential for the East Side of I-80," Conference on the Sesimic Retrofit of Utah's Highway Bridges, sponsored by the Utah Department of Transportation, January 20-22, 1999. Salt Lake City, Utah.

"Geofoam Design, Construction and Research on the I-15 Corridor Reconstruction Project," Annual Meeting of the Society of the Plastics Industry, Inc., April 23 and 24, 1998, New Orleans, La.

B

FARHANG OSTADAN

2 Agnes St.
Oakland, CA 94618

510-547-6881

fostadan@aol.com

EDUCATION:

Ph.D., Civil Engineering
University of California, Berkeley, California, 1983.

SUMMARY:

<u>15 Years</u>: Extensive experience in dynamic analysis and seismic safety evaluation of above and underground structures and subsurface materials. Co-developed and implemented SASSI, a system for seismic soil-structure interaction analysis currently in use by the industry worldwide. Developed a method for liquefaction hazard analysis currently in use for critical facilities in the United States.

EXPERIENCE:

As Chief Soils Engineer with Bechtel, San Francisco office, Mr. Ostadan was responsible for providing guidance and support to all projects in the areas of earthquake resistant design, dynamic analysis of structures, soil-structure interaction (SSI) analysis, and seismic stability evaluation of subsurface materials. He has participated in seismic studies and reviews of numerous nuclear structures, offshore structures, underground structures and transportation structures; conducted technology transfer and training courses for engineers of various companies and institutes including Bechtel Corporation, Impell Corporation, General Electric Company, SEAONC, Westinghouse Corporation, Lawrence Livermore Laboratory, and Tennessee Valley Authority (TVA) in USA; Kraftwerk Union, AG West Germany; Tractional Inc., Belgium, Nuclear Data Corporation, Japan; Atomic Energy Organization, Iran.

Major project work includes seismic analysis and evaluation of responses for: the Diablo Canyon Nuclear Station as part of the Long-Term Seismic Program (LTSP); NRC/EPRI large scale seismic experiment in Lotung, Taiwan; large underground circular tunnel for Super Magnetic Energy Storage (SMES); General Electric ABWR and SBWR standard nuclear plants; Westinghouse AP600 standard nuclear plant; Tennessee Valley Authority (TVA) nuclear structures (Browns Ferry, Sequoyah, Watts Bar); several facilities involving liquid gas storage tanks; Heerma TTP offshore structure in the North Sea; seismic stability and liquefaction study at the ITP, RTF, and K-facilities in the Savannah River Site for the Department of Energy; several transportation projects including numerous Caltrans bridges in California; BART extension lines including tunnel and aerial structures along the Dublin and San Francisco airport lines, Muni

Metro Project, Downtown San Francisco; and Richmond Parkway Project in the San Francisco Bay area.

EXPERIENCE (cont'd)

1983 – 1985: Earthquake Engineering Technology Inc., San Ramon, California, As Project Engineer was responsible for development of a method for nonlinear seismic soil-structure interaction analysis in time domain.

1979 – 1983: University of California, Berkeley. As Research Assistant in the Civil Engineering Department, duties included development of the flexible volume method for dynamic SSI analysis of soil-pile-structure systems; member of SASSI development team.

PROFESSIONAL REGISTRATION:

Registered Civil Engineer, California

PROFESSIONAL ASSOCIATIONS:

Member of American Society of Civil Engineers
Member of EERI, Earthquake Engineering Research Institute.
Member of Sigma Xi, The Scientific Honor Society, University of California, Berkeley

PUBLICATIONS

Technical Papers:

Lysmer, J., Tabatabaie-Raissi, Tajirian, F., Vahdani, S., Ostadan, F., SASSI - A System for Analysis of Soil-Structure Interaction, Report No. UCB/GT/81-02, Geotechnical Engineering Department of Civil Engineering, University of California, Berkeley, April 1981.

Ostadan, F., Dynamic Analysis of Soil-Pile-Structure Systems, Ph.D. Dissertation, University of California, Berkeley, 1983.

Ostadan, F., Udaka, T., Okumura, M., One Dimensional Seismic Response Study Using Different Soil Models, 8th SMIRT Conference, Brussels, Belgium, 1985.

Ostadan, F., Lysmer, J., Dynamic Analysis of Directly Loaded Structures on Pile Foundations,

8th SMIRT Conference, Brussels, Belgium, 1985.

Ostadan, F., Lysmer, J., Simplified Dynamic Analysis of Soil-Pile-Structure Systems, 5th International Symposium & Exhibition on Offshore Mechanics and Arctic Engineering, Tokyo, Japan, 1986.

Technical Papers (cont'd):

Ostadan, F., Tseng, Wen S., Lilhanand, K., Application of Flexible Volume Method to Soil-Structure Interaction Analysis of Flexible and Embedded Foundations, 9th SMIRT Conference, Lausanne, Switzerland, 1987.

Ostadan, F., Tseng, Wen S., Effect of Foundation Flexibility and Embedment on the Soil-Structure Interaction Response, 9th World Conference on Earthquake Engineering, Tokyo, Japan, August 1988.

Ostadan, F., Tseng, Wen S., Effect of Site Soil Properties on Seismic SSI Response of Deeply Embedded Structures, ASCE Foundations Engineering Congress, Evanston, Illinois, June 1989.

Ostadan, F., Tseng, W. S., Sawhney, P. S., Liu, A. S., The Effect of Embedment Depth on Seismic Response of a Nuclear Reactor Building Design, 10th SMIRT Conference, Los Angeles, California, August 1989.

Ostadan, F., Arango, I., Oberholtzer, G., Hsiu, F., Radially Loaded Circular Tunnel Structure, IX Panamerican Conference on Soil Mechanics and Foundation Engineering, Vina del Mar, Chile, August 1991.

Ostadan, F., Marrone, J., Arango, I., Litehiser, J., Liquefaction Hazard Evaluation: Methodology and Application, 3rd U.S. Conference on Lifeline Earthquake Engineering, Los Angeles, California, August 1991.

Ostadan, F., Hadjian, A. H., Tseng, W. S., Tang, Y. K., Tang, H. K., Parametric Evaluation of Intermediate SSI Solutions on Final Response, 11th SMIRT Conference, Tokyo, Japan, August 1991.

Ostadan, F., Arango, I., Litehiser, J., Marrone J., Liquefaction Hazard Evaluation, 11th SMIRT Conference, Tokyo, Japan, August 1991.

Ai-Shen Liu, G. W. Ehlert, R. S. Rajagopal, P. S. Sawhney, F. Ostadan, Seismic Design of ABWR and SBWR Standard Plants, ICONE2, San Francisco, California, March 1993.

R. S. Rajagopal, S. Sawhney, F. Ostadan, Seismic Considerations for the Standardized Advanced Light Water Reactor (ALWR) Plant Design, American Power Conference, Chicago, Illinois, April 1993.

I. Arango, F. Ostadan, Qualification of Liquefaction Hazard and Its Application to Risk Assessment and Urban Zoning, 5th International Conference on Seismic Zonation, Nice, France, October 1995.

F. Ostadan, S. Mamoon, I. Arango, Effect of Input Motion Characteristics on Seismic Ground Responses, 11th World Conference on Earthquake Engineering, Acapulco, Mexico, June 23-28, 1996

Technical Papers (cont'd):

I. Arango, F. Ostadan, M. Lewis, B. Gutierrez, Quantification of Seismic Liquefaction Risk, ASME PVP & ICVT Pressure Vessel and Piping Conference, Montreal, Quebec, Canada, July 21-26, 1996.

F. Ostadan, T. Liu, K. Gross, R. Orr, Design Soil Profiles for Seismic Analyses of AP600 Plant Standard Design, ASME PVP & ICVT Pressure Vessel and Piping Conference, Montreal, Quebec, Canada, July 21-26, 1996.

Computer Programs:

User's Manual, Theoretical Manual, and Verification Manual for Computer Program SASSI.

Installation and Validation Reports for Computer Program SASSI prepared for: EDS Nuclear Incorporated, California; Kraftwerk Union, AG, West Germany; Tractional Incorporated, Brussel, Belgium; Bechtel Corporation; General Electric Company; Westinghouse Corporation; Lawrence Livermore Laboratory.

User's Manual, Verification Manual, and Application Manual for Computer Program NANSSI (nonlinear analysis of soil-structure systems), Kozo Keikaku Engineering, Japan.

User's and Theoretical Manuals for Computer Program ASHLE (Advanced Seismic Hazard/Liquefaction Evaluation), Bechtel Corporation.

C

altran

Dr. Mohsin R. Khan

EDUCATION: B.S., Mathematics, Christ Church College, Kanpur, India (1970)
B.S., Mechanical Engineering, Harcourt Butler Technological Institute, Kanpur, India (1974)
M.S., Mechanical Engineering, Clarkson College of Tech., USA (1975)
Ph.D. Solid Mechanics (Structures), Clarkson College of Tech., USA (1978)

REGISTRATION/MEMBERSHIP:

Professional Mechanical Engineer - State of California
American Society of Mechanical Engineer
IEEE-344 Committee (Current Member)
EPRI- STERI (1995-96)

HONORS: Award of Merit from Bechtel Power Corporation
Received several Performance Recognition awards at Pacific Gas & Electric Company

EXPERIENCE: Altran Corporation, San Francisco – Engineering Manager

- Mohsin is the Manager of Altran's Structural Mechanics group in San Francisco. In his current assignments, his responsibilities include both providing technical and project management leadership. The Structural Mechanics group includes structural analysis and design, piping analysis and design, and equipment analysis and design and testing functions.

- Performed Structural Probabilistic Risk Assessment of MHM Crane for the Dry Cask Project. This is for the DOE's Hanford site in Washington.

- Providing equipment seismic specifications for testing and design review support for the CVDF (Cold Vacuum Drying Facility) equipment for the Dry Cask Project. This is also for the DOE's Hanford site in Washington.

- Provided design and analysis for the Indian Point 1 Power Plant's Spent Fuel Pool project. This involved in providing a steel liner and the leak chase system to the existing pool walls to protect it from leakage; designing a partition wall with a water tight steel gate and redesigning the new Fuel Racks. Detailed dynamic analyses of the Spent Fuel Pool Racks using ANSYS program were performed.

- Provided structural design reviews for the Humbolt Bay Power Plant (HBPP) Stack removal project. This involved in assessing the effect of soil pressure on the existing building structural walls due to placement of heavy crane loads during the stack removal.

- Provided design upgrade for the DCPP Refueling Water Storage Tank Project. This involved several dynamic analyses of various piping branch lines, several tank analyses, and structural support evaluations.

- Provided inspection procedures to comply with the ASME section IWE and IWL requirements and calculations for the steel liner and concrete wall threshold thicknesses for the DCPP containment.

- Provided engineering support and structural evaluation and design modification in support of DCPP Seismic Gap program.

- Developing a Design Basis Criteria Document for the HBPP Dry Cask/ISFSI project. This design basis criterion will be used by the HBPP for the bid evaluation to select a Dry Cask vendor for the decommissioning purposes.

- Development of the criteria and the specifications for dynamic shake table tests for the RVLIS system. This involved several cabinet finite element analyses and shaker table testing to meet the seismic design bases requirements for the DCPP site. This involved witnessing the shake table tests, identification of problems which develop during the shake table tests, participation in the resolution of them (e.g., design changes to strengthen the components), and interpretation and acceptance of the test results for documentation in engineering calculations.

Pacific Gas & Electric Company, San Francisco (Nuclear Power Generation)

- Principal Engineer, Company's Structural/Equipment Seismic, Vibration and Dynamic Analysis and design expert

- Supervise and provide technical guidance to a group of engineers in the area of vibration and seismic analysis of Mechanical, Electrical, Instrumentation and Controls, and Heating and Ventilation equipment for Diablo Canyon Nuclear Power Plant (DCPP).

- Developing Finite Element models of equipment and perform computer analyses to qualify the equipment for DCPP design bases seismic events. This involves reviewing the results of the analyses for completeness accuracy, validity and documenting the results in Design Calculations.

- Extensive use of computer programs - STRUDL, ANSYS, BSAP, SAP90,SAP2000 and several in-house programs. Occasional use of NASTRAN and STARDYNE.

- Development of the criteria and the specifications for dynamic shake table tests for instrumentation, electrical devices, and control panels/cabinets to assure that the tests will envelope the specific load conditions that the components would be subjected to during design bases seismic events at DCPP. This involves witnessing the shake table tests, identification of problems which develop during the shake table tests, participation in the resolution of them (e.g., design changes to strengthen the components), and interpretation and acceptance of the test results for documentation in engineering calculations.

- Provide extensive technical support around the clock including weekends to resolve construction constraints/problems during the design implementation and provide alternative design solutions acceptable to client (DCPP) without increasing the outage cost and duration

- Provide technical interface with Westinghouse and other vendors in the area of NSSS equipment, structural dynamic/seismic qualification. This requires monitoring the vendor's technical work, cost and also help in meeting the stringent project schedules.

- Provide technical support including seismic calculations and their interpretation to Diablo Canyon's Long Term Seismic Program consultants for Mechanical, Electrical, I&C, and HVAC equipment. This involves Probabilistic Risk Assessments and Fragility Evaluations.

- Performed structural dynamic analyses to qualify the Manipulator and the Spent Fuel Pool Bridge Cranes for DCPP units 1 and 2. This involved modifying the cranes for the upgrade pupose and to meet high seismic demand loads.

- Provide structural/seismic evaluation for the dedication activities for Replacement Parts Program. This is an extensively tedious and technically involved process and poses a constant challenge to accept commercially procured parts to be used at Diablo Canyon Nuclear Power plant which is in a very high seismic zone.

- Perform LOCA (Loss of Coolant Accident Analysis) and Seismic analyses for Containment Isolation Purge and Exhaust valves.

- Involved in review of Dynamic Qualification of High Density Spent Fuel Racks for Diablo units 1 and 2. Also performed an independent Finite Element analyses to check vendor's work during the licensing hearing process due to outside intervenors. Provided expert testimony on High Density Spent Fuel Rack design.

- Seismic evaluation of as-found conditions of equipment which deviate from the original design bases. For example: electrical cabinets and panels with loose and missing mounting hardware, and damaged equipment and equipment supports. These events are immediately reportable to the NRC if the as-found conditions compromise the public safety. Therefore, prompt resolution of these problems becomes extremely important. The resolution process often times requires unique dynamic non-linear analyses and obtaining test information from various sources.

- Team member of the Diablo Canyon and Humbolt Bay Power Plant's Dry Cask Project. He has unique technical and business experience in the site and vendor selection process for the Dry Cask project. He is familiar with all of the nation's leading Dry Cask vendors, their design features, and their limitations.

- Managed the equipment and structural seismic design and dynamic testing of many capital projects at DCPP. In addition to his work on Dry Casks, he was involved in the Underground Diesel Fuel Tank replacement project, 6th Diesel Generator addition project, Vital 4kv switcher replacement project, Inverter Replacement Project, 125V Battery Replacement Project, Electrical Relays Replacement Project, Various Pump-Motor Replacement Project, Main Annunciator Replacement Project, Main Control Board Modification Project due to Human Factor Effects, Post Accident Monitoring Equipment Replacement Project, and replacing the

buried piping for the ultimate heat sink. Each of these projects was in millions of dollar, and posed significant technical and licensing challenges.

BECHTEL POWER CORPORATION, San Francisco

- Susquehanna Nuclear Power Plant (Mark II) Design evaluation of Containment Building. Structural analysis of box beam platforms, pipe whip restraints, diaphragm slab, hangers, quenchers, etc., using STRUDL, BSAP, ANSYS and other in-house computer programs.

- Equipment (Dynamic) Qualification Group (EQG) Worked on Limerick, Susquehanna-Mark II, Hope Creek-Mark I, Diablo Canyon-PWR, Nuclear Power Plants.

- In charge of all computer activities (Mainframe/Micros) of the group.

- Developed a computer program "BSEIM" that generates artificial random time histories from a given floor response spectra for UNIVAC 1100 series mainframe computer.

- Extensive use of AISC Code and ASME Code. Section III and VIII, and IEEE-344-75 Standard for dynamic qualification of equipment.

- Use of Vibration Testing to qualify Electrical, HVAC and Mechanical Equipment such as cabinets, terminal blocks, valves, dampers, operators, and pumps, etc., per IEEE-344-75.

- Successfully defended NRC Audits on non-NSSS Mechanical Equipment for Diablo Canyon Unit 1 Project.

- Reviewed equipment dynamic qualification of Westinghouse (NSSS) supplied Mechanical Equipment for Diablo Canyon Project.

- Dynamic Analysis (Response Spectra and Time-History) and Static Analysis (Thermal Load, Dead Weight, Pressure, and Nozzle Loads) for a variety of Class I equipment, i.e., valves, component cooling water surge tanks (horizontal), boric acid tanks (vertical), safety injection pump and motor, auxiliary feedwater pumps and motors, auxiliary feedwater turbine, diesel generators and radiators, silencers, starting air receiver and turbo charger tanks, priming tanks, lube and diesel oil filters, strainers, containment fan cooler boxes, dampers, heat exchangers, electrical cabinets, radwaste compressors, coolers, and decay tanks, containment hydrogen purge exhaust and supply filters, steam generator blowdown tanks and coolers. Portable fire pumps, and condensers, etc.

- Extensive use of computer programs - STRUDL, ANSYS, BSAP, NASTRAN, STARDYNE, and in-house Bechtel programs.

- Involved in finite element model review for Seismic and Impact Analysis of Vacuum Breaker Valves for the Downcomers of Susquehanna, Limerick and Shoreham Nuclear Power Plants (Vendor - AGCO).

- Interaction with vendors and review of vendor analysis or test reports; review and writing of design or test specifications; coordination with the project and field about design modifications;

coordination with procurement in issuing purchase orders to outside vendors for analysis and testing.

- Supervised a group of 25 engineers; responsible for the review technical guidance and approval of calculations; manpower planning and man-hour estimates; scheduling and employee performance evaluations; finalizing the equipment qualification packages for NRC audit.

- Developed a General Finite Element Structural Analysis Code MICROSAC for structural analysis for Microcomputers (Static and Dynamic Analysis Capability).

Clarkson College of Technology – Assistant Professor

- Taught Particle Dynamics, Static, Strength of Materials, Machine Design as undergraduate courses.

- Taught Finite Element Methods, Theory of Elasticity and Structural Dynamics as graduate courses.

- Author of several papers in ASME, AISC, and AIAA (American Institute of Aeronautics and Astronautics) Journals.

- Developed several special finite element computer programs, structural optimization programs, kinematic analysis programs.

- In charge of all computer activities in the department of Civil and Environmental Engineering.

- Supervised three graduate students towards their M.S. thesis. Thesis topics involved development of Structural Optimization techniques to design Beams, Frames, Aircraft wing structures, Transmission Towers, etc., under stress, displacement, and natural frequency constraints. Also, a special finite element was developed to analyze horizontally curved beams under warping effects using flexibility method.

PUBLICATIONS:

KHAN, M. R. and WILLMERT, K. D., "Vibration Analysis of Mechanisms Using Constant Length Finite Elements," ASME, Paper No. 76-WA/DE-21, 1976.

KHAN, M. R. and WILLMERT, K. D., "General Kinematic Analysis Of Planar Mechanisms Consisting of Four Bar Chains and Slider Cranks," A User Manual, Department of Mechanical and Industrial Engineering, Clarkson College of Technology, Potsdam, N.Y., Report No. MIE-030.

KHAN, M. R. and WILLMERT, K. D., and THORNTON, W. A., Automated Analysis/Design of High Speed Planar Mechanisms," Proceedings of the 5th OSU Applied Mechanisms Conference, Oklahoma, pp. 21-1 to 21-9, November 1977.

KHAN, M. R., WILLMERT, K. D., and THORNTON, W. A., "A New Optimality Criterion Method for Large Scale Structures," Proceedings of the AIAA/ASME 19th Structures, Structural Dynamics and Materials Conference, pp. 47-58, April 1978.

WILLMERT, K. D., THORNTON, W. A., and KHAN, M. R., "A Hierarchy of Methods for Analysis of Elastic Mechanisms with Design Application," ASME Paper No. 78-DET-56, 1978.

THORNTON, W. A., WILLMERT, K. D. and KHAN, M. R.,"Mechanism Optimization Via Optimality Criterion Techniques," Trans. ASME, Journal of Mechanical Design, Vol. 101, No. 3, July 1979, pp. 392-397.

KHAN, M. R., THORNTON, W. A., and WILLMERT, K. D.,"Optimality Criterion Techniques for Structures with Multiple Design Variables per Member," Proceedings of the AIAA/ASME/ASCE/AHS 20th Structures, Structural Dynamics, and Materials Conference, pp. 87-95, April 1979.

KHAN, M. R., WILLMERT, K. D., and THORNTON, W. A., "An Optimality Criterion Method for Large Scale Structures," AIAA Journal Vol. 17, No. 7, pp. 753-761, July 1979.

KHAN, M. R., WILLMERT, K. D., and THORNTON, W. A., "A Computer Program Package for Large Scale Structural and High Speed Mechanism Design," Proceedings of the Engineering Software Conference at the University of Southhampton, England, September 1979.

SYED, M. I., WILLMERT, K. D., and KHAN, M. R., "Optimality Criteria Techniques Applied to Structures Composed of Different Element Types," Proceedings of the AIAA/ASME/ASCE/AHS 21st Structures, Structural Dynamics and Materials Conference, May 1980, pp. 345-351.

KHAN, M. R., "Optimality Criterion Techniques Applied to Frames Having Nonlinear Cross-Sectional Properties," Proceedings of the AIAA/ASME/ASCE/AHS 22nd Structures, Structural Dynamics, and Materials Conference, pp. 233-241, 1981.

KHAN, M. R., and WILLMERT, K. D., "An Efficient Optimality Criterion Method for Natural Frequency Constrained Structures," Journal of Computers and Structures, Vol. 14, No. 5-6, pp. 501-507, 1981.

KHAN, M. R., and AINSO, H., "Optimal Design of Nonplanar Diaphragms," A Technical Report for Bendix Corporation, Utica, N.Y., 1979.

AINSO, H., and KHAN, M. R., "Effects of Rim and Hub Flexibility During Axial Loading of Diaphragms," A Technical Report for Bendix Corporation, Utica, N.Y., 1980.

KHAN, M. R., "A General Computer Program for the Beams, Frames and Trusses - A User's Manual," Department of Civil and Environmental Engineering Department, Clarkson College,Potsdam, N.Y. 13676, Report Number 81-3, August 1981.

KHAN, M. R., "A Computer Program for Optimum Design of Natural Frequency Constrained Structures," Department of Civil and Environmental Engineering, Clarkson College, Potsdam, N.Y. 13676, Report Number 81-2, August 1981.

KHAN, M. R., "Optimality Criterion Techniques Applied to Frames Having Nonlinear Cross-Sectional Properties," Department of Civil and Environmental Engineering, Clarkson College, Potsdam, N.Y. 13676, Report Number 81-1, August 1981.

KHAN, M. R., and WILLMERT, K.D., "Finite Element Quasi-static Deformation Analysis of Planar Mechanisms With External Loads Using Static Condensation," ASME Paper No. 81-DET-104.

WILLMERT, K. D., JENG, K. K., MONSEF, M., and KHAN, M. R., "Application of Optimality Criterion Methods to Member Lengths and Cross-Sectional Properties of Frame Structures," Proceedings of the AIAA/ASME/ASCE/AHS 23rdStructures, Structural Dynamics and Materials Conference, AIAA-82-0720-CP, May 1982.

ZACHAROPOULOS, A., WILLMERT, K. D., and KHAN, M. R., "An Optimality Criterion for Structures with Stress, Displacement and Natural Frequency Constraints," Proceedings of the AIAA/ASME/ASCE/AHS 24th Structures, Structural Dynamics and Material Conference, AIAA-83-0939, May 1983.

KHAN, M. R., "BSEIM Computer Program For Generation of Random Synthetic Time Histories From The Floor Spectra," Rev. 1, Civil/Structures Group, Bechtel Power Corporation, January 1984.

KHAN, M. R., "Optimality Criterion Techniques Applied to Frames Having General Cross-Sectional Relationships," AIAA Journal, Vol. 22, No. 5, pp. 669-676, May 1984.

KHAN, M. R., "Improved Method of Generation of Artificial Time Histories, Rich In All Frequencies, From Floor Spectra," Earthquake Engineering and Structural Dynamics Journal, Vol. 15, pp. 985-992, 1987.

KHAN, M. R., "User and Verification Manual For MICRO-SEIM Computer Program," September 1988, Pacific Gas & Electric Company.

KHAN, M. R., "Users' Guide, Theory Verification for GENRRS

Computer Program," February 1989, Pacific Gas & Electric Company.

KHAN, M. R. and SHAKIBNIA, B., "Generic Fragility Seismic Testing of Breakers For Diablo Canyon Power Plant," Proceedings of the 1990 ASME Boiler & Pressure Vessel Conference, Book No. H00616.

KHAN, M. R. and CHEN, WAYNE W.H., "Nonlinear InteractionAnalysis for Seismic Equipment qualification," Proceedings of the ASME, PVP -Vol. 237-2, Seismic Engineering Vol. 2, pp. 115-120, 1992.

KHAN, M. R., CHEN, WAYNE W.H. AND WANG, T. Y.,"Seismic Qualification of Multiple Interconnected Safety Related Cabinets In a High Seismic Zone," Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology,.Vol. K2, 1993.

KHAN, M. R., CHEN, WAYNE W.H., SHAKIBNIA, B, AND DAVIS, R.D. "Torque Test For Bolts/Screws In Maintaining Equipment eismic Qualification," Proceedings of the ASME, PVP Vol. 56-2, Seismic Engineering, Vol. 2, pp. 41-45, 1993.

KHAN, M. R., CHEN, WAYNE W.H. AND CHU, W. S., "Seismic Qualification of a Commercial Grade Diesel Generator In a High Seismic Zone," Proceedings of 4th International Topical Meeting on Nuclear Thermal Hydraulics, Operations and Safety., Vol. 1, pp. 5D-1 - 5D-5, April 1994.

PRESENTATIONS

"Vibrational Analysis of Mechanisms Using Constant Length Finite Elements," ASME Winter Annual Meeting, New York, December 1976.

"Optimality Criterion Techniques Applied to Mechanical Design," ASME Design Engineering Technical Conference, Chicago, September 1977.

"Automated Analysis/Design of High Speed Planar Mechanism," 5th OSU Applied Mechanism Conference, Oklahoma, November 1977.

"A New Optimality Criterion Method for Large Scale Structures," AIAA/ASME 19th Structures, Structural Dynamics, and Materials Conference, Maryland, April 1978.

"A Hierarchy of Methods for Analysis of Elastic Mechanisms with Design Applications," ASME Design Engineering Technical Conference, Minneapolis, September 1978.

"Mechanism Optimization Via Optimality Criterion Techniques," ASME Design Engineering Technical Conference, Minneapolis, September 1978.

"Optimality Criterion Techniques Applied to Structures with Multiple Design Variable per Members," AIAA/ASME/ASCE/AHS 20th Structures, Structural Dynamics, and Materials Conference, St. Louis, April 1979.

"Optimality Criteria Techniques Applied to Structures Composed of Different Element Types," AIAA/ASME/ASCE/AHS 21st Structures, Structural Dynamics, and Materials Conference, Seattle, Washington, May 1980.

"Optimality Criterion Techniques Applied to Frames Having Nonlinear Cross-Sectional Properties," AIAA/ASME/ASCE/AHS 22nd Structures, Structural Dynamic, and Materials Conference, Atlanta, Georgia, April 1981.

"Finite Element Quasi-Static Deformation Analysis of Planar Mechanisms With External Loads Using Static Condensation," ASME Design Engineering Technical Conference, Hartford, Connecticut, September 1981.

"Application of Optimality Criterion Methods to Member Lengths and Cross-Sectional Properties of Frame Structures," AIAA/ASME/ASCE/AHS 23rd Structures, Structural Dynamics, and Materials Conference, New Orleans, Louisiana, May 1982.

"An Optimality Criterion for Structures With Stress, Displacements and Natural Frequency Constraints," AIAA/ASME/ASCE/AHS 24th Structures, Structural Dynamics and Materials Conference, Lake Tahoe, Nevada, May 1983.

"Generic Fragility Seismic Testing of Breakers For Diablo Canyon Power Plant," 1990 ASME Boiler & Pressure Vessels Conference, Nashville, Tennessee, July 1990.

"Nonlinear Interaction Analysis for Seismic Equipment qualification," ASME Pressure Vessels and Piping Conference, New Orleans, Louisiana, June 1992.

"Seismic Qualification of Multiple Interconnected Safety Related Cabinets In a High Seismic Zone," 12th International Conference on Structural Mechanics in Reactor Technology, Stuttgart, Germany, August 1993.

"Torque Test For Bolts/Screws In Maintaining Equipment Seismic Qualification," ASME Pressure Vessels and Piping Conference, Denver, Colorado, July 1993.

"Seismic Qualification of a Commercial Grade Diesel Generator In a High Seismic Zone," 4th International Topical Meeting on Nuclear Thermal Hydraulics, Operations and Safety, Taipei, Taiwan, 1994.

"Failure Analysis of Globe Control Valves with Spring-Diaphragm Actuator for Nuclear Power plant Applications, " Topical Meeting on Nuclear Thermal Hydraulics, Beijing, China, April 1997.

"Seismic Qualification of New Underground Diesel Fuel Tanks at Diablo Canyon Nuclear Power Plant Site, " ASME Pressure Vessels & Piping Division Conference, Orlando, Florida, 1997.

D

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United States Government **Department of Energy**

memorandum

DATE: August 22, 2001

REPLY TO
ATTN OF: Office of Nuclear and Facility Safety Policy:HChander:301-903-6681

SUBJECT: REVISED DOE STANDARD 1020-2001, NATURAL PHENOMENA HAZARDS DESIGN AND EVALUATION CRITERIA FOR DEPARTMENT OF ENERGY FACILITIES, PROJECT NUMBER NPHZ - 0001

TO: Technical Standards Program Managers

The subject technical standard has been revised to conform to latest industry codes/standards and is released for your review and comment. The technical standard can be found at the Technical Standards Program Web Site at http://tis.eh.doe.gov/techstds/. After comments have been resolved, the document will be approved as a DOE standard and listed in the DOE standards Index, DOE-TSL-1.

Please review the document and provide your comments to the preparer, Dr. Harish Chander, EH-53, by the comment due date (45 day coordination period) listed for this project at the above Web Site. Your comments must be designated as either essential or suggested and proposed resolutions to those comments provided. Essential comments are those which, if not addressed, would make the document technically unacceptable to your organization and must be supported by detailed rationale. All comments must be in the form of word-for-word changes to the draft document.

Responses from DOE Area Offices, Laboratories, and M&O Contractors should be returned through the appropriate DOE management or organization channels in sufficient time to permit consolidation by the Operations Office and subsequent transmittal to the preparer before the due date. Comments received after that date will be held for the next revision, unless it is possible to address them without affecting the timely approval of the document. Please contact Dr. Chander if you have any questions on this DOE Technical Standards project. Dr. Chander can be reached at 301-903-6681. Please e-mail questions to Dr. Chander at harish.chander@eh.doe.gov.

Richard L. Black

Richard L.&lack, Director
Office of Nuclear and
Facility Safety Policy

Table C-3 Seismic Performance Goals & Specified Seismic Hazard Probabilities

Performance Category	Target Seismic Performance Goal, P_F	Seismic Hazard Exceedance Probability, P_H	Risk Reduction Ratio, R_R
1	1×10^{-3}**	4×10^{-4}*	
2	5×10^{-4}**	4×10^{-4}*	
3	1×10^{-4}	4×10^{-4}* (1×10^{-3})[1]	4 (10)[1]
4	1×10^{-5}	1×10^{-4} (2×10^{-4})[1]	10 (20)[1]

* The seismic exceedence probability is based on USGS maps generated in 1997 (and included in IBC 2000) for 2% exceedence probabiity in 50 years.

** The design methodology of IBC 2000 for Seismic Use Groups I and III achieves approximately performance goals of PC-1 & PC-2 respectively though it does not meet the relationship shown in equation C-1 for the seismic provisions.

1 For sites such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC which are near tectonic plate boundaries.

Different structures, systems, or components may have different specified performance goal probabilities, P_F. It is required that for each structure, system, or component, either: (1) the performance goal category; or (2) the hazard probability (P_H) or the DBE together with the appropriate R_R factor will be specified in a design specification or implementation document that invokes these criteria. As shown in Table C-3, the recommended hazard exceedance probabilities and performance goal exceedance probabilities are different. These differences indicate that conservatism must be introduced in the seismic behavior evaluation approach to achieve the required risk reduction ratio, R_R. In earthquake evaluation, there are many places where conservatism can be introduced, including:

1. Maximum design/evaluation ground acceleration and velocity.
2. Response spectra amplification.
3. Damping.
4. Analysis methods.
5. Specification of material strengths.
6. Estimation of structural capacity.
7. Load or scale factors.
8. Importance factors/multipliers.
9. Limits on inelastic behavior.
10. Soil-structure interaction (except for frequency shifting due to SSI).
11. Effective peak ground motion.
12. Effects of a large foundation or foundation embedment.

For the earthquake evaluation criteria in this standard, conservatism is intentionally introduced and controlled by specifying (1) hazard exceedance probabilities, (2) load or scale

E

Where is Diablo Canyon's Used Fuel Stored Now?

There are 193 fuel assemblies, each containing 225 fuel rods, in use in each of the two Diablo Canyon reactors at any given time. When fuel assemblies are no longer able to sustain a reaction powerful enough to produce energy, they are withdrawn and replaced with new assemblies.

The used fuel assemblies are taken out of the reactor and placed in metal racks submerged in a pool of water. These specially designed pools are built of concrete reinforced with steel and lined with stainless steel. Diablo Canyon has two of these used fuel pools. Each has several primary and backup cooling systems. The water in each pool cools the fuel rods and also serves as a shield against radiation.

The pools are filling up. As currently configured, they will be nearly full by 2006, decades before any federal storage facility is open. Additionally, Pacific Gas and Electric Company has a license to operate Diablo Canyon until 2025.

We need to plan for, and build, additional space at Diablo Canyon to store used fuel so that Diablo Canyon can continue providing California with much needed electricity through the plant's full licensed life

The Preferred Option for Used Fuel: Dry Storage

There are essentially two options available to deal with used fuel storage at Diablo Canyon: wet storage and dry storage. Since the federal Nuclear Regulatory Commission considers both options safe, we have evaluated the pros and cons of both.

During the past year, we sought and received input from many San Luis Obispo County community and government leaders as well as other county residents. In light of this input, as well as our technical evaluations, we have concluded that dry storage is the preferred solution for additional used fuel storage at Diablo Canyon Power Plant.

Unit 1 is currently licensed to operate until 2021 and Unit 2 until 2025.

Dry storage will take older, cooler fuel assemblies out of the used fuel pools and store them above ground, in specially designed storage containers. After used fuel has been cooled for at least five years in the used fuel pools, it may then be stored in these specialized, leak-proof dry storage containers. Several types of container design are already approved by the NRC for use at nuclear power plants throughout the United States. The containers will have multiple safety features and will be placed in a protected area at Diablo Canyon. At least 10 nuclear power plants in the United States currently use dry storage to supplement their wet storage pools.

Pacific Gas and Electric Company believes dry storage of used fuel is the preferred option because it offers a longer-term storage solution pending the federal government's construction of a permanent national used fuel storage facility.

In the event that the federal government takes even longer than the multi-decade schedule now envisioned before opening a national storage site, dry storage would eventually still be necessary at Diablo Canyon because older nuclear powered plants with more used fuel would be entitled to have access to the storage facility ahead of Diablo Canyon. Onsite storage will also most likely be necessary for interim storage of used fuel once Diablo Canyon ceases operation.

Dry Storage of Used Nuclear Fuel

Nuclear power plants in the United States are increasingly turning to dry storage as the preferred method of increasing onsite used fuel storage at their nuclear power plants. After used fuel has been cooled in pools for at least five years, it may then be stored in specialized sealed containers approved by the Nuclear Regulatory Commission (NRC).

The NRC has approved seven different storage container designs that are safe and environmentally sound for use at any nuclear power plant in the country.

F

ANALYTICAL STUDY OF HI-STORM 100 CASK SYSTEM FOR SLIDING AND TIP-OVER POTENTIAL DURING HIGH-LEVEL SEISMIC EVENT

Technical Report No. 01141-TR-001

Revision 0

Prepared for:

Office of The Attorney General, State of Utah

November 30, 2001

altran

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Report Record

REPORT No.: 01141-TR-001 Rev. No.: 0 Sheet No.: 1

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TITLE: Analytical Study of Hi-Storm 100 Cask System for Sliding and Tip-Over Potential During High-Level Seismic Event

CLIENT: Office of Attorney General, State of Utah FACILITY: Private Fuel Storage

REV. DESCRIPTION: Initial Issue

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Originator(s)	Date	Verifier(s)	Date
Mohsin Khan	12/7/01	Mike Brady	12/7/01
		Behrooz Shakibnia	12/7/01

VERIFICATION: Verification is performed in accordance with EOP 3.4 as indicated below

____ Design review as documented on the following sheet

____ Alternate calculation as documented in attachment ____ or ________

____ Qualification testing as documented in attachment ____ or ________

APPROVED FOR RELEASE:

PROJECT MANAGER: ________ Date: 12/7/01

Mohsin Khan

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1.0 INTRODUCTION

The purpose of this study is to perform sliding and tip-over nonlinear time history analysis of HI-STORM 100 casks. The cask system is planned to sit vertically unanchored on a concrete slab in a 2 by 4 array as described in References 1, 2 and 3. The concrete slab rests on ground soil. This study will use three ground motion time histories for 2000-year return period of the region as provided in Reference 4. The nonlinear dynamic analyses will simulate the cask behavior due to specified input motions applied at the cask base. It will demonstrate the sensitivity of HI-STORM 100 cask response (sliding and rocking) movements due to various input data selected. The results of this study will be compared with the results presented in Reference 1.

2.0 INPUT DATA

The following structural design and analyses information is based on Reference 1, 2 and 3.

Overpack Weight = 267,664 lbs.
Radial Concrete Weight = 163,673 lbs.
Maximum Cask Weight = 360,000 lbs.
MPC Weight (including fuel) = 88,857 lbs.

Length of the Cask = 231.25 inches
Diameter of the Bottom Plate = 132.50 inches
Inside Diameter of the Cask Shell = 72.50 inches
Outside Diameter of the Cask Shells = 132.50 inches

MPC Height = 190.5 inches
MPC Diameter = 68.375 inches
MPC Bottom Plate Thickness = 2.5 inches
MPC Top Plate Thickness = 9.5 inches

Total Vertical Stiffness Between the Pad and Cask = 4.541×10^{8} lbs./inch
Total Horizontal Stiffness Between the Pad and Cask = 4.541×10^{11} lbs./inch

Lowest Coefficient of Friction = 0.2
Highest Coefficient of Friction = 0.8

Two horizontal time histories and one vertical time histories are provided in Reference 4. The amplitudes of these time histories are in terms of gravity (i.e., g) and are at a constant time interval of 0.005 seconds. The time history duration is 30 seconds.

3.0 METHODOLOGY FOR FINITE ELEMENT ANALYSIS

Figure 1 shows a rigid mass than can slide and uplift. Various forces and local stiffnesses needed to solve this problem are also shown. The block moves when inertial forces acting on this mass exceed the frictional force μW. Local contact stiffnesses in the horizontal and

vertical direction (K_H and K_V) are needed in mathematical simulation just before any sliding occurs at any instant of time. There are several structural analyses codes, which can solve this nonlinear analysis problem. SAP2000 and ANSYS structural analysis codes described in References 5 and 6 are used to perform nonlinear time history analyses. The methodologies and the formulation of the finite elements and nonlinear solution methods are described in these references. Three mathematical models considered in this study, are described in section 4.0.

4.0 CASK MODEL STUDIES

Three mathematical cask models with varying degree of complexity are considered in this study as follows:

- The purpose of the first model is to obtain horizontal sliding displacements for the HI-STORM100 cask without any vertical excitation. For this case, two industry standard structural analysis codes, ANSYS and SAP2000, are used to compare the maximum sliding displacements values and the nodal displacement time history traces for benchmarking purposes.
- The second model includes the effect of vertical excitation without any rocking effects due to cask height.
- The third model represents a three-dimensional cask with vertical and horizontal rigid beam finite elements. The mass density of the vertical beam elements are adjusted to obtain approximately the weight of a fully loaded HI-STORM100 cask.

For the last two models, several parametric analyses are performed to show the effect of change of contact stiffnesses and coefficient of friction values on the HI-STORM100 cask motion. The effect of use of various structural damping values (ALFA, BETA, or Modal) is also studied in the third model.

4.1 Rigid Mass Sliding Model

A HI-STORM 100 cask weighing 360 kips is modeled by a single rigid beam element that can slide horizontally. Figure 2 shows the SAP2000 model. The height of beam element is chosen small enough to avoid any rocking effects. The bottom of this element is connected to a nonlinear element that accommodates sliding effects. At the base of the nonlinear springs, horizontal time histories corresponding to 2000-year return period were applied. For checking the adequacy of these input time histories, corresponding response spectra were generated and were compared with those spectra in Reference 4.

The results of nonlinear sliding displacements are shown in Figures 3 and 4. Figure 5 provides the sliding displacements results obtained from ANSYS Computer Code (Reference 6). Table 1 provides the absolute maximum relative displacement values obtained by the two computer codes. This table shows good agreements between sliding displacement values obtained by SAP2000 and ANSYS computer codes.

Table 1

Absolute Maximum Relative Displacements in Horizontal Directions for Sliding Cask

		Stiffness for Non-Linear Elements		Relative Cask Displacements		
Computer Code	Coefficient of Friction μ	Vertical Stiffness K_V (lbs./in.)	Horizontal Stiffness K_H (lbs./in.)	Horizontal Displacement D_x (in.)	Horizontal Displacement D_y (in.)	Vertical Dispalcement D_z (in.)
SAP2000	0.2	0	1×10^5	9.371	5.599	NA
ANSYS	0.2	0	1×10^5	9.371	5.599	NA

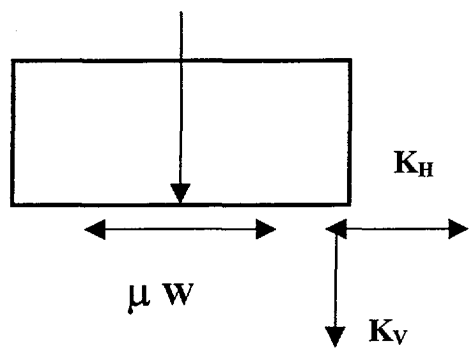

Figure 1
Cask Model for Sliding and Uplift

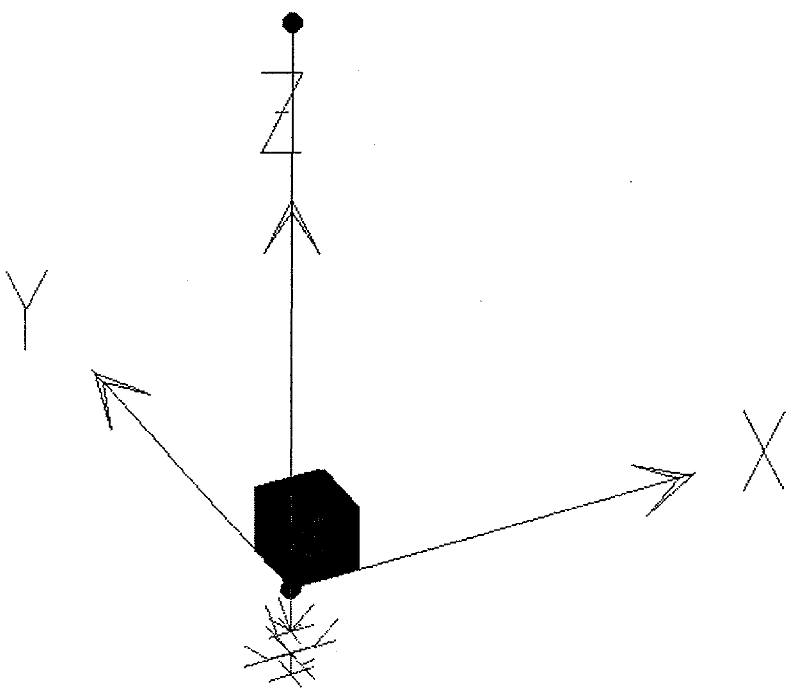

Figure 2
Cask Math Model for Sliding and Uplift

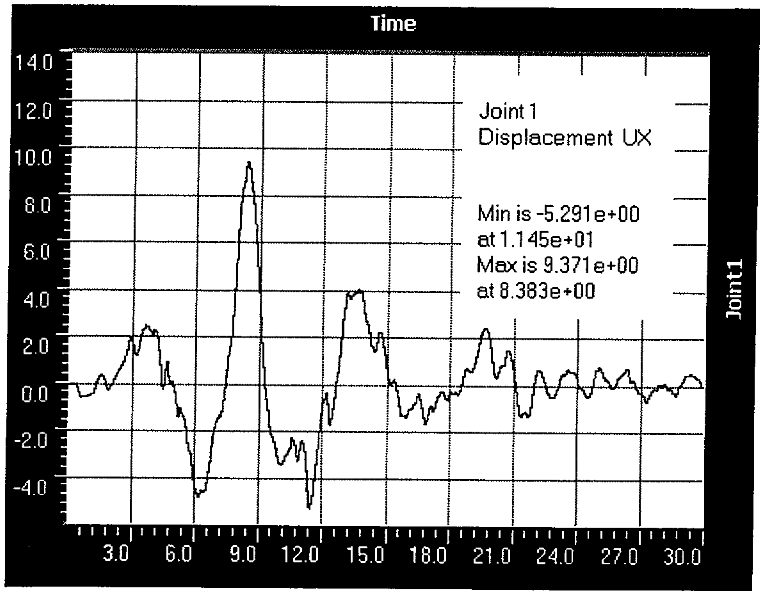

Figure 3
Sliding Cask – Relative Displacement Time History for Horizontal X-Direction

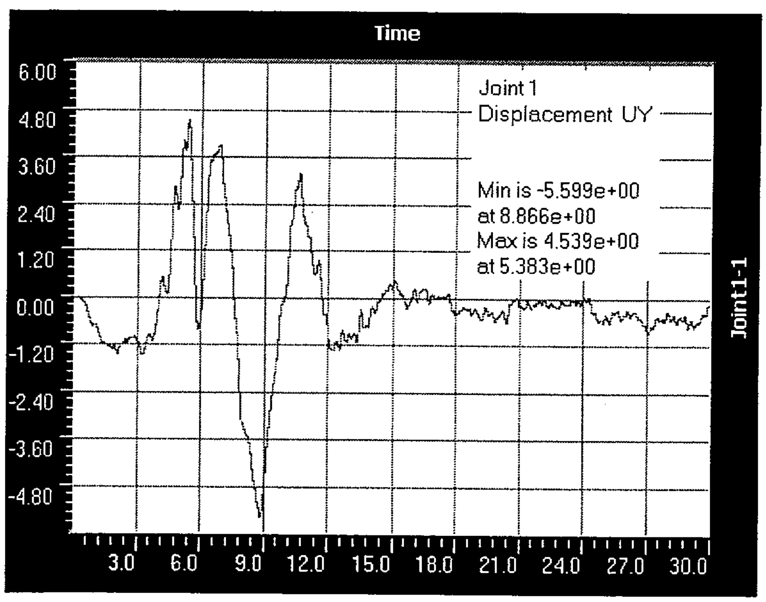

Figure 4
Sliding Cask – Relative Displacement Time History for Horizontal Y-Direction

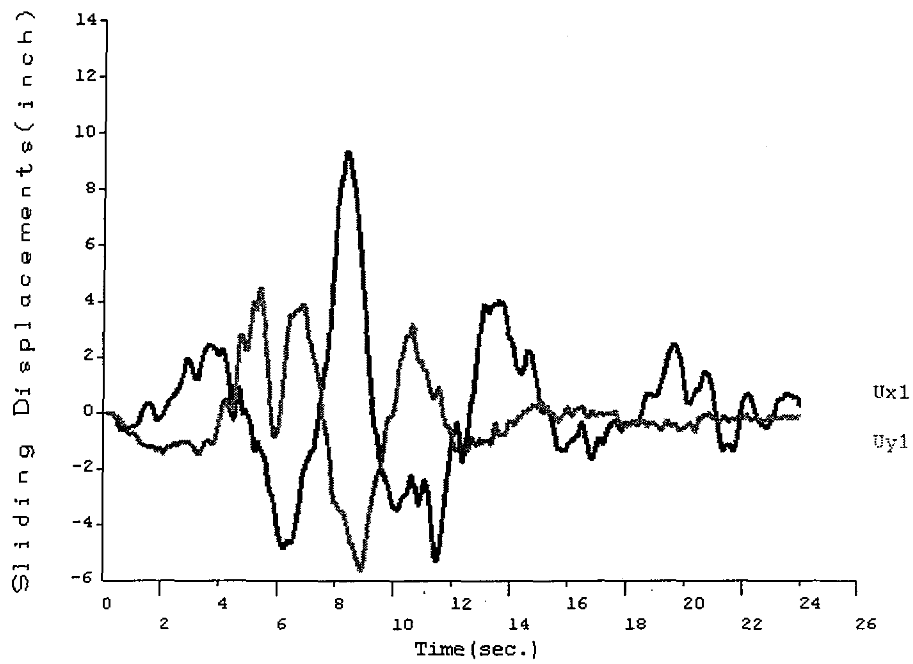

Figure 5
Sliding Cask – Relative Displacement Time History for Horizontal X and Y Directions (ANSYS)

4.2 Sliding and Uplift Model Without Consideration of Rotational Effects

A HI-STORM 100 cask weighing 360 kips is modeled by a small single rigid beam element that can slide and uplift. Figure 2 shows the beam model. The height of this element is chosen small enough to avoid any rocking effects. The bottom of this element is connected to a nonlinear element that can slide and provides compression only nonlinear effect. The rotational effects are ignored. Three time histories corresponding to 2000-year return period were applied at the base of the nonlinear spring.

Starting contact stiffnesses in the horizontal and vertical directions are used as input in performing analyses. These stiffnesses are effective before any sliding or during compression condition. In general, high contact stiffness values are used during numerical simulation of these types of analyses. The real problem with using high stiffnesses is that it artificially treats the solution to be a linear for a significant duration of the input motions. The artificially high stiffnesses also absorb significant amount of energy before sliding actually occurs. The "instantaneous" structural frequencies (before sliding) could also alter the initial sliding velocities, which would affect the final structural response during high level ground input motions. Table 2 summarizes the results of this study. A significant variation in cask displacements are found by selecting different combination of three input parameters; coefficient of friction μ, and local contact stiffnesses in the horizontal and vertical direction (K_H and K_V).

Table 2 shows that as the vertical contact stiffness between the cask and the ISFSI pad increases, the sliding displacements reduce significantly. This effect shows that the cask is approaching a simulated anchored condition to the ISFSI pad and there is no dynamic amplification in the vertical direction. Similarly, as the horizontal contact stiffness increases, the horizontal sliding displacement also reduces. However, the change in vertical stiffness has a greater impact in changing the cask displacements than the horizontal contact stiffness.

The results presented in Table 2 show that the sliding and uplift displacements are not unique and tend to reduce significantly with the increase in contact stiffness values. Reference 3 also gives a range of displacement solutions, which are not unique. The solution given in Reference 3 would change significantly with lower contact stiffnesses and with lower damping values. The convergent sliding displacements should be obtained by selecting a range of contact stiffnesses. A contact stiffness value of 10^8 lbs/in is considered too high for an unanchored cask that can slide and rock. Ideally, these stiffnesses should be small values for an unanchored cask.

Table 2

Absolute Maximum Relative Displacements for Sliding Cask
Associated with Sliding and Uplift Model Without Consideration of Rocking Effects
Due to Cask Height

		Stiffness for Non-Linear Elements		Relative Cask Displacements		
Study Run Number	Coefficient of Friction μ	Vertical Stiffness K_V (lbs./in.)	Horizontal Stiffness K_H (lbs./in.)	Horizontal Displacement D_x (in.)	Horizontal Displacement D_y (in.)	Vertical Dispalcement D_z (in.)
1	0.2	1×10^6	1×10^5	9.16	12.25	2.10
2	0.8	1×10^6	1×10^5	42.74	31.35	1.75
3	0.2	10×10^6	1×10^5	19.25	7.69	1.93
4	0.8	10×10^6	1×10^5	12.70	14.03	1.94
5	0.2	100×10^6	1×10^5	6.38	3.86	0.006
6	0.8	100×10^6	1×10^5	4.74	3.05	0.006
7	0.2	100×10^6	1×10^5	1.91	1.29	0.006
8	0.8	100×10^6	1×10^5	3.80	1.38	0.006
9	0.2	454×10^6 (**)	1×10^5	6.39	3.86	0.006
10	0.8	454×10^6 (**)	1×10^5	4.83	3.06	0.006
11	0.8	454×10^6 (**)	454×10^6	0.057	0.071	0.001

** Stiffness values correspond to those values given in Reference 1.

4.3 Three-Dimensional Sliding and Uplift/Tip-Over Model

A HI-STORM 100 cask weighing approximately 360 kips is modeled by 72 beam elements. Figure 3 shows a three-dimensional (3D) beam model. The cask can slide, uplift and rock or tip-over under the specified seismic input motions. The base diameter of this model is 132 inches and the cask height is 231 inches. The model is graphically generated using the GUI feature of SAP2000. The bottom 8 nodes are connected to nonlinear elements. Three time histories corresponding to 2000-year return period were applied at the base of the nonlinear springs.

Table 3 summarizes the results of this study. A significant variation in cask displacements found by selecting different combination of three input parameters; coefficient of friction μ, and local contact stiffnesses in the horizontal and vertical direction (K_H and K_V). The cask displacements are comparable to those obtained in Table 2. In this case, solutions are also obtained at 0.01%, 1% and 5% structural damping. In reality, the structural damping (ALFA, BETA or Modal) must be small or insignificant value. Cask sliding and uplift displacements should be independent of structural damping for cask elements, since cask is considered as a rigid body. Only friction should be primary energy dissipation mechanism. From the results presented in Table 3, it is shown that the use of higher damping will significantly lower the nonlinear structural response of the cask system.

Table 3

Absolute Maximum Relative Displacements for Sliding Cask
Associated with 3-D Sliding and Tip-Over Model

		Stiffness for Non-Linear Elements		Relative Cask Displacements		
Study Run Number	Coefficient of Friction μ (% Damping)	Vertical Stiffness K_V (lbs./in.)	Horizontal Stiffness K_H (lbs./in.)	Horizontal Displacement D_x (in.)	Horizontal Displacement D_y (in.)	Vertical Dispalcement D_z (in.)
1	0.8 (0.01%)	1×10^6	1×10^5	372.76	229.65	27.24
2	0.2 (1%)	1×10^6	1×10^5	48.69	48.86	2.56
3	0.8 (1%)	1×10^6	1×10^5	306.00	120.45	18.01
4	0.8 (1%)	10×10^6	1×10^5	60.21	46.85	8.59
5	0.2 (5%)	1×10^6	1×10^5	4.38	9.08	2.28
6	0.8 (5%)	1×10^6	1×10^5	21.48	56.85	5.87
7	0.2 (5%)	10×10^6	1×10^5	6.00	4.71	0.15
8	0.8 (5%)	10×10^6	1×10^5	15.11	10.42	0.94
9	0.8 (5%)	100×10^6	1×10^5	27.63	58.27	2.56

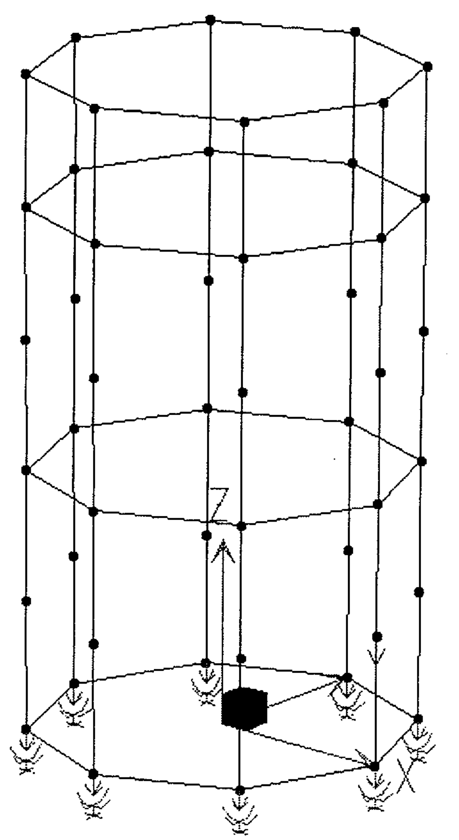

Figure 6
Cask 3-D Math Model for Sliding, Uplift and Rocking

5.0 CASK OVERTURNING/TIPPING

In this section, a calculation is performed to determine the minimum requirement for vertical uplift of the cask body and the kinetic energy (or velocity) required to tip a cask under high level horizontal and vertical earthquake motions. As shown in Figure 7, the cask will tip over when the center of gravity (CG) of the cask body with height (H) and radius (R) moves over one edge.

For HI-STORM 100 Cask:

Height of the Cask (H) = 231.25 inches
Diameter of the Bottom Plate (2R) = 132.50 inches
Maximum Cask Weight = 360,000 lbs.
Cask Velocity = V_M
Horizontal Cask Velocity in X direction = V_X
Horizontal Cask Velocity in Y direction = V_Y
Vertical Cask Velocity in Z direction = V_V
Gravity = 386.4 in./sec.^{2}

$\alpha = \tan^{-1}(H/2R) = \tan^{-1}(231.25/132.50) = 60.2°$

$\beta = 90° - 60.2° = 29.8°$

The value of tipover angle β is conservatively calculated due to selection of cask CG equals to half the cask height.

Maximum Cask Uplift Height = 2 R (sin 29.8°) = 65.85 inches

Change in the C.G Height at Tipping $= \{R^2 + (0.5\,H)^2\}^{1/2} - 0.5\,H = 133.26 - 115.63$
$= 17.63$ in.

Potential Energy (PE) Required for Cask Overturning:

PE = Cask Weight * Change in the C.G height at tipping
PE = (360,000 lbs.) (17.63 in.) = 6,346,800 lbs.-inch

Kinetic Energy (KE) Required for Cask Overturning:

KE = (1/2) * Cask Mass * (Cask Velocity)^2
KE = $(1/2)\,[360{,}000/386.4]\,(V_M)^2$

Considering all three direction of earthquake motions by using SRSS rule for directional response combination , the (Cask Velocity)^2 is:

$$(V_M)^2 = (V_X)^2 + (V_Y)^2 + (V_V)^2$$

Table 3 summarizes the spectral accelerations and velocities at 5% damping from 0.4 to 2.0 Hz frequency range for a 2000-year return period seismic event. This is an important frequency range and it is judged that the instantaneous rocking and sliding frequencies lie in this range for an unanchored cask. From Table 4, the average horizontal velocity in this frequency range for both the fault-normal and fault-parallel direction is approximately equal and the average vertical velocity is approximately half of the average horizontal velocities.

Therefore,

$V_X \cong V_Y$ and $V_V \cong 0.5 * V_X$

Substituting these values,

$$(V_M)^2 = (V_X)^2 + (V_X)^2 + (0.5*V_X)^2 \quad = 2.25 * (V_X)^2$$

For Possible Cask Overturning/Tipping:

KE > PE

$(1/2)\ [360{,}000/386.4]\ (V_M)^2 > 6{,}346{,}800$ lbs.-inch;

$(1/2)\ [360{,}000/386.4] * 2.25 * (V_X)^2 \quad > 6{,}346{,}800$ lbs.-inch; $\Rightarrow V_X > 77.82$ in/sec.

For some cases summarized in Table 3, the cask velocities obtained from the velocity traces during an earthquake are much higher than the calculated value of V_X. For example, Figure 8 shows a velocity trace for Study Run Number 1 in Table 3. In this case the maximum cask velocity is 271 in/sec.. Therefore, it is possible for HI STORM 100 cask to overturn (i.e., tip over) during a high-level seismic event.

Table 4

Spectral Accelerations and Velocities at 5% Damping From 2000-Year Return Period Seismic Input Time Histories

		Fault-Normal		Fault-Parallel		Vertical	
		Accelerations	Velocity	Accelerations	Velocity	Accelerations	Velocity
Period (sec)	Frequency (Hz)	(g)	inch/sec.	(g)	inch/sec.	(g)	inch/sec.
2.500	0.4	0.2020	31.05	0.14228	21.87	0.09851	15.15
2.000	0.5	0.2500	30.75	0.20892	25.70	0.12739	15.67
1.667	0.6	0.3084	31.61	0.28255	28.96	0.14389	14.75
1.429	0.7	0.3604	31.66	0.3794	33.33	0.18618	16.36
1.250	0.8	0.4175	32.09	0.44719	34.38	0.20408	15.69
1.111	0.9	0.4485	30.65	0.50923	34.80	0.23842	16.29
1.000	1.0	0.5851	35.98	0.56703	34.87	0.26506	16.30
0.909	1.1	0.5838	32.64	0.63656	35.59	0.28059	15.69
0.833	1.2	0.6480	33.21	0.71335	36.56	0.33617	17.23
0.769	1.3	0.7524	35.59	0.77482	36.65	0.30952	14.64
0.714	1.4	0.7858	34.52	0.85834	37.70	0.38997	17.13
0.667	1.5	0.7393	30.31	0.94301	38.66	0.39032	16.00
0.625	1.6	0.8890	34.17	0.92933	35.72	0.42693	16.41
0.556	1.8	0.8856	30.26	1.12603	38.47	0.47324	16.17
0.500	2.0	1.0783	33.15	1.17603	36.16	0.56712	17.44
Average Velocity			32.51		33.96		16.06

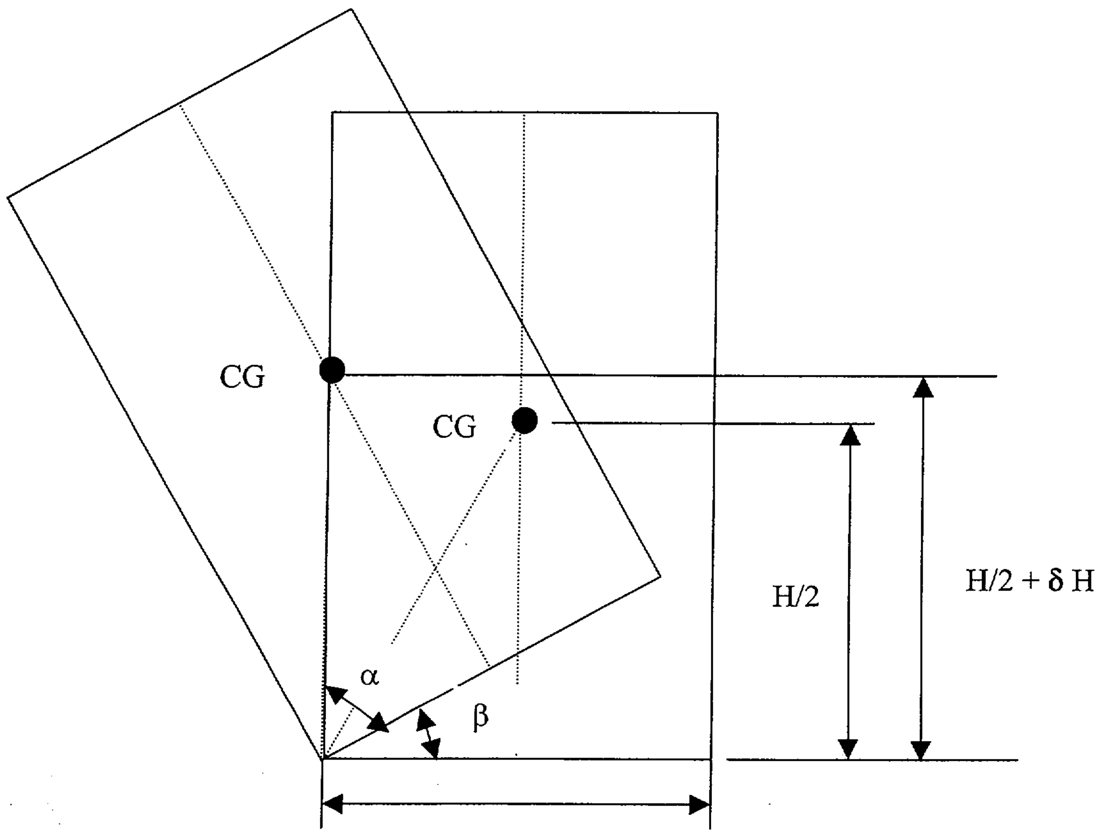

Figure 7
Cask Overturning/Tipping

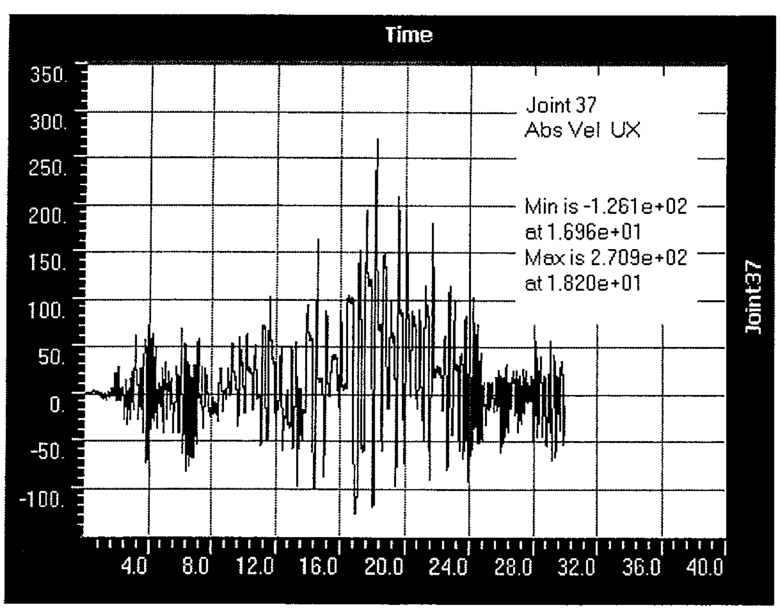

Figure 8
Cask Velocity Trace For Study Run Number 1

6.0 CONCLUSIONS

In this study, evaluation of sliding and tip-over potential of HI-STORM 100 cask under seismic input motion has been performed. The HI-STORM 100 cask movements have been calculated in various ways, from a simple rigid body sliding to a detailed three-dimensional beam model. It has been demonstrated that a wide range of sliding and uplift displacements can be obtained based on the selected input values of the local contact stiffnesses and the coefficient of friction. These local contact stiffnesses are effective before the sliding or uplift begins. In general, high stiffness values are used for solving sliding and tip over type problems. The real issue with this approach is that if these stiffnesses are selected too high, then the program artificially assumes the problem to be linear for a significant duration of the input motion. Using high stiffnesses could absorb some amount of energy before sliding actually occurs. Also, with high stiffnesses the "instantaneous" structural frequencies (before sliding) could also alter the sliding velocities, which would affect the final structural response during high level ground input motion.

Key insights and observations from our study are summarized below.

- The input time histories used for these evaluations were obtained from reference 4 and are based on a 2000-year return period seismic event. Reference 4 time histories did not consider the amplification due to soil structural interaction at the top of ISFSI pad. The soil structural interaction effect would amplify the time history response on top of the ISFSI pad. Therefore, the vertical input motion at the base of the cask would be much higher than those used in this study.

- This study shows that he sliding and rocking displacements are significantly affected by the local stiffness values used as input in the mathematical model simulation. However, the sliding displacements should be independent of the local contact stiffness values used.

- This study shows that using initial high local contact vertical stiffness in Reference 1, 2, 3 significantly minimizes the vertical displacements. This approach also helps in significantly reducing the horizontal sliding displacements.

- For an unconstrained cask that acts as a rigid body, only frictional forces should provide energy dissipation mechanism during sliding and rocking.

- This study shows that the sliding and rocking displacements are significantly affected by using higher damping values during numerical simulation. Reference 1 used a high beta damping value. Use of high structural damping (ALFA, BETA, or Modal) must be avoided since the cask acts as a rigid body. A very small damping value should be used for numerical stability purposes.

- From the numerical study, it is shown that the sliding results presented in References 1, 2, and 3 are unconservative and are not unique. Due to wide variation in cask response obtained based on input data used, it is recommended that the sliding displacements presented in References 1, 2 and 3 be based on mathematical models that have been benchmarked with actual or prototype models based on shake table test data.

- Since the vertical response alters the cask dynamic reactions significantly, the computed cask reactions during cask uplift/drop and impact due to vertical amplifications would be much higher than those used in the pad design. See Reference 1.
- It is possible that for high-level ground input motion, the HI STORM 100 cask could overturn (i.e., tip over).

7.0 REFERENCES

1. Excerpt from Holtec Report No. HI-971631, *Multi-Cask Seismic Response at PFS ISFSI for Private Fuel Storage L.L.C.*, Rev. 0, prepared by Holtec International, May 1997.
2. Excerpt from Holtec Report No. HI-2012653, *PFSF Site-Specific HI-STORM Drop/TipOver Analyses*, Rev 1, prepared by Holtec International, May 2001.
3. Excerpt from Holtec Report No. HI-2012640, *Multi Cask Response at PFS ISFSI From 2000-YR Seismic Event*, Rev. 2, prepared by Holtec International.
4. *Development of Design Ground Motions for the Private Fuel Storage Facility*, Revision 1, prepared by Geomatrix Consultants, Inc., March 2001.
5. SAP2000 User's Manual, Version 7.4, Computers and Structures, Inc.
6. ANSYS Version 5.7 Computer Code, ANSYS Inc.

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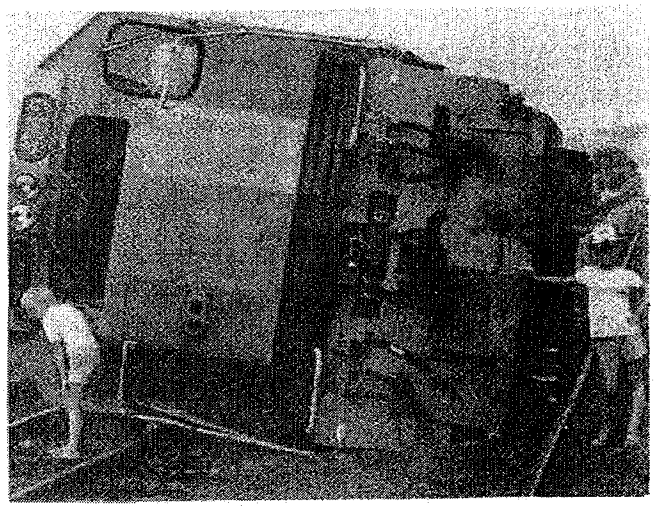


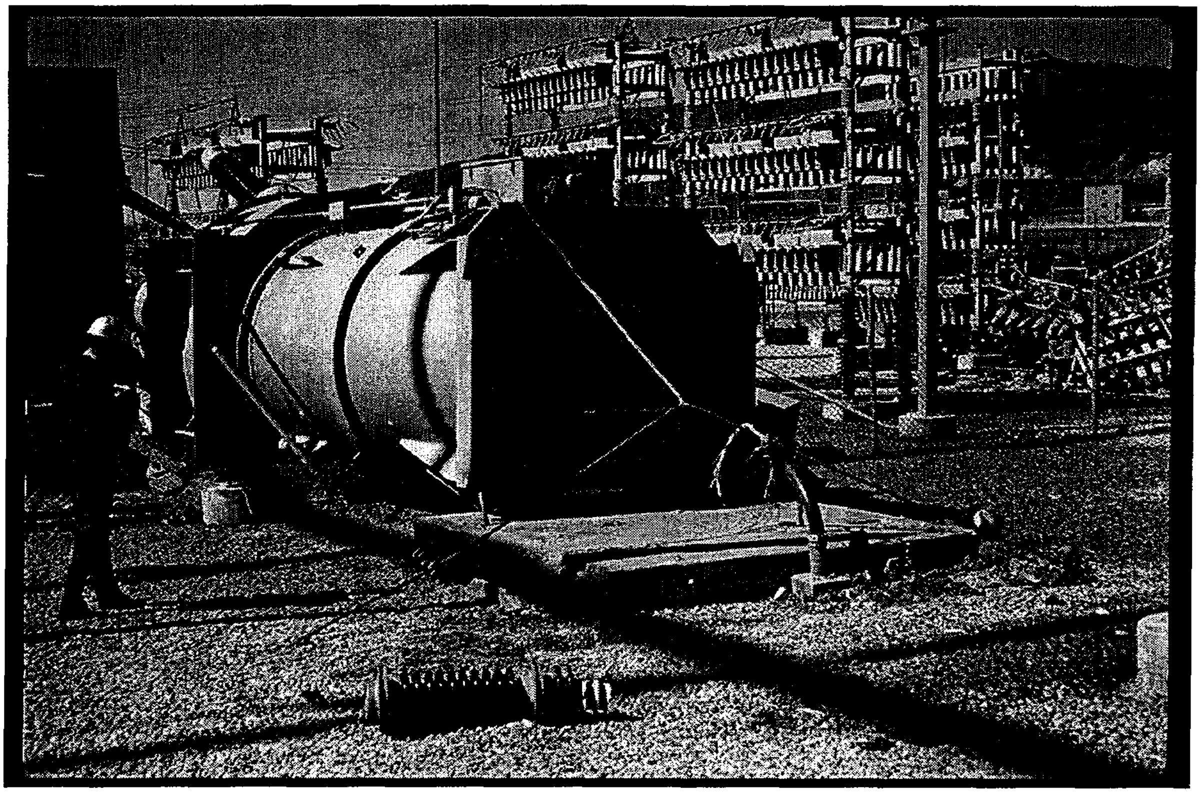
Figure 9: *Edgecumbe.* Overturned 67-ton diesel train engine in the rail yard. The engine was stationary at the time of the earthquake. The driver, who was in the cabin, reported that the engine moved both side to side and up and down before overturning. He escaped uninjured from the top side.

Source of Fig. 9 - Summary of the 1987 Bay of Plenty, New Zealand Earthquake, EQE International, 1987.

The earthquake magnitude was a $M_L = 6.2$. The focal depth of the earthquake was estimated at 6 miles. The earthquake produce approximately 6 miles of discontinuous surface rupture and a complex main scarp about 3.5 miles long striking southwest roughly 0.5 miles east of Edgecumbe. No strong ground motion instrumentation was available, but EQE investigators estimate horizontal pga of 0.5 to 1.0 g in Edgecumbe.

The EQE team also report that damage to smaller unanchored tanks and equipment was widespread. However, all equipment at a steam plant was anchored and the damage was superficial.

Overturned railroad boxcars at banana packing plants near Sixaola on the Panama-Costa Rica border (photo and caption from Slides on the Costa Rica Earthquake of April 22, 1991 - Set III: Performance of Industrial Facilities and Lifelines - Earthquake Engineering Research Institute).



"Overturned equipment was badly anchored."

Location: Sylmar Converter Station
Feb. 9, 1971 San Fernando Earthqukae
M = 6.5

Source of slide:

Steinbrugge Collection, Earthquake Engineering Research Center, University California, Berkeley.

Slide No. S4301

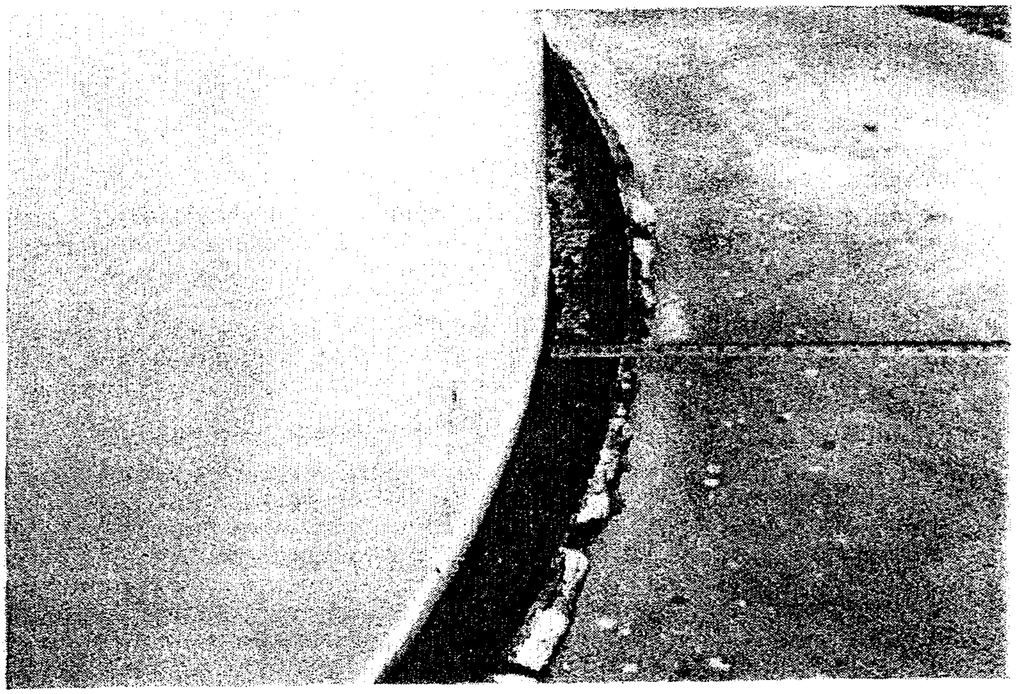


Figure 6: The four gasoline storage tanks located in the southern end of Coalinga did not leak in spite of sliding about 4 inches, as shown in the lower photograph. Sliding was common for unanchored equipment and structures throughout the area.

Source of Fig. 6 - Summary of the May 2, 1983 Coalinga, California Earthquake, EQE Incorporated, 1986.

The earthquake magnitude was a $M_L = 6.7$. It was centered near the town of Coalinga. Available ground motion records in the Coalinga vicinity indicate that peak ground accelerations were high. Depending on the location, peak accelerations ranged from 0.20 g to over 0.60 g.

At the Shell water treatment plant the following was reported:

"<u>Extensive sliding of unanchored</u> tanks with rupture of attached piping. Yielding of supports for anchored tanks." (p. 3).

LEARNING FROM EARTHQUAKES

LESSONS FROM LIFELINE ENGINEERING ON ELECTRIC POWER SYSTEMS:

- THE MOST IMPORTANT DAMAGE AND LOSS CONTROL PROCEDURES INCLUDE:
  1. *SEISMIC TIES* TO PREVENT LARGE RELATIVE DISPLACEMENTS AND POUNDING BETWEEN STEAM GENERATION EQUIPMENT AND SUPPORTING STRUCTURES.
  2. *ANCHORAGE* OF EQUIPMENT AND TANKS TO PREVENT OVERTURNING, SLIDING, AND POUNDING AS WELL AS PIPING-SYSTEM INTERATION PROBLEMS.
  3. *DESIGN* OF SUPPORTING STRUCTURES FOR EQUIPMENT TO ACCOMODATE THE FREQUENCY-DEPENDENT EFFECTS OF SOIL AMPLIFICATION OF GROUND MOTION.
  4. *DESIGN* OF CONSTRUCTION JOINTS TO PROVIDE AN ADEQUATE GAP TO ELIMINATE IMPACT BETWEEN TURBINE PEDESTAL AND POWERHOUSE STRUCTURES.
  5. *DESIGN AND INSTALLATION* OF BATTERY RACKS WITH ADEQUATE ANCHORAGE TO PREVENT COLLAPSE.

From: Slide set - Learning From Earthquakes IV (LFE IV) Earthquake Engineering Research Institute.

This slide shows that a lesson learned from the performance of past earthquakes is:

"Anchorage of equipment and tanks to prevent overturning, sliding, and pounding as well as pipe-system integration problems."

Thus, past experience and common sense suggests that the storage casks at the PFS facility should be anchored to the pads.

A common misconception is that heavy objects are not easily moved by earthquakes. Actually, their large mass means they experience large inertial loads. Newton's second law formula, F = ma, or the inertial force equals the mass times the acceleration, conveys this idea quantitatively. This 93-ton bronze statue of Buddha in Kamakura, Japan slid more than a foot in the 1923 Kanto earthquake. (Photo and caption from Non Structural Damage Slide Set, Earthquake Engineering Research Institute).

From DOE-STD-1020(94), p. C-59

"Engineered anchorage is one of the most important factors affecting seismic performance of systems or components and is required for all performance categories. It is intended that anchorage have both adequate strength and sufficient stiffness to perform its function. Types of anchorage include: 1) cast-in-place bolts or headed studs, 2) expansion or epoxy grouted anchor bolts and 3) welds to embedded steel plates or channels. The most reliable anchorage will be achieve by properly installed cast-in-place bolts or headed studs, undercut type expansion anchors, or welding. Other expansion anchors are less desirable than cast-in-place undercut, or welded anchorage for vibratory environments (i.e., support of rotating machinery), for heavy equipment, or for sustained tension supports. Epoxy grouted anchorage is considered to be the least reliable of the anchorage alternatives in elevated temperature or radiation environments.

EXHIBIT 3

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 (Independent Spent Fuel Storage Installation)	)	ASLBP No. 97-732-02-ISFSI
	)	December 7, 2001

DECLARATION OF DR. MARVIN RESNIKOFF

I, Dr. Marvin Resnikoff, hereby declare under penalty of perjury and pursuant to 28 USC § 1746, as follows:

1. I am a physicist with a Ph.D. in high-energy theoretical physics from the University of Michigan and also the Senior Associate of Radioactive Waste Management Associates (RWMA), a private technical consulting firm based in New York City. I have researched radioactive waste issues for the past 27 years and have extensive experience and training in the field of nuclear waste management, storage, and disposal. Our work at RWMA is about equally divided between three issues related to the matters covered in this deposition: (i) transportation and storage of irradiated fuel, (ii) personal injury law suits involving radiation in which we calculate radiation exposures, and (iii) remediation of radioactive landfills and contaminated sites. A copy of my resume was submitted in conjunction with my Declaration in support of the State of Utah's Request for Admission of Late-Filed Contention Utah RR (Suicide Mission Terrorism and Sabotage) (October 10, 2001).

2. I have considerable experience in reviewing and analyzing cask designs. Since 1975 I have worked on transportation issues, including cask safety, for the States of Utah, Nevada (including Clark and White Pine Counties), Idaho, New Mexico and Alaska. This work began with work for the New York Attorney General's office on the safety of transporting plutonium by plane out of John F. Kennedy International Airport. My role in the case was to determine whether the plutonium shipping container could be punctured and the amount of plutonium that could be released. I was an invited speaker at the 1976 Canadian meeting of the American Nuclear Society to discuss the risk of transporting plutonium by air. On behalf of the State of New York, I also reviewed and provided comments on NUREG-170, "Final Environmental Statement on the Transportation of Radioactive Material by Air and Other Modes." On behalf of the State of Nevada and Clark County, Nevada, I provided comments on the transportation cask safety studies and transportation risk

assessments, such as the Modal Study and references, and more recently NUREG/CR-6672. RWMA has conducted transportation risk assessments for the State of Nevada and has employed various computer codes and formulas to estimate the amount of radioactivity released in and the health and economic consequences of a severe accident, including the computer models RADTRAN, RISKIND, RESRAD, and HOTSPOT. In addition, in hearings before state commissions and in federal court, I investigated proposed dry storage facilities at the Point Beach (WI), Prairie Island (MN) and Palisades (MI) reactors. These are matters that are also addressed in this declaration. For the Council on Economic Priorities, I have co-authored a book on the transportation and storage of irradiated fuel: Resnikoff, M. and Audin, L., The Next Nuclear Gamble (Council on Economic Priorities, 1983).

3. I have considerable training and experience in the field of risk assessment involving nuclear and hazardous facilities, serving as an expert witness in numerous personal injury cases in which I estimated radiation doses and the likelihood these exposures caused cancer. These cases involved uranium mining and milling, oil pipe cleaning, X-rays, thorium contamination and other issues. This work involved the use of computer codes, such as MILDOS, to estimate radiation doses and spreadsheets employing dose conversion factors.

4. I was designated as one of the State's testifying expert for Contention Utah L part B on September 28, 2001. I assisted in the preparation, in part, of State of Utah's Request for Admission of Late-Filed Modification to Basis 2 of Utah Contention L, filed on January 26, 2000 ("First Modification to Basis 2"), and have reviewed another request by the State for Admission of Late-Filed Modification to Basis 2 of Utah Contention L, filed November 9, 2000 ("Second Modification to Basis 2") and submitted Declarations in support thereof. I also participated in the preparation of discovery against the Applicant and the NRC Staff with respect to Utah L part B.

5. I am familiar with Private Fuel Storage, L.L.C.'s ("PFS's") license application, Environmental Report and Safety Analysis Report in this proceeding, as well as the applications for the storage and transportation casks (HI-STORM and HI-STAR) PFS plans to use. I am also familiar with NRC regulations, guidance documents, and environmental studies relating to the storage and transportation of spent nuclear power plant fuel, including NUREG-0800, NUREG-1536, 10 CFR Part 100, EPA's Protective Action Guide, and Federal Register Notice dated December 4, 1996 (61 Fed. Reg. 64257).

6. I have reviewed Applicant's Motion for Summary Disposition of Part B of Utah Contention L (November 9, 2001), its Statement of Material Facts on Which No Genuine Dispute Exists, and attachments thereto. I provide this declaration in support of the State's Response to PFS's Motion for Summary Disposition. The following statements in this declaration are based on my experience, training, and

best professional judgment.

7. PFS makes gross claims without any quantified analyses that at a 2,000 year or 10,000 year DBE there will be no radiation dose increase beyond regulatory limits. *See* PFS Joint Declaration at 23. There are serious flaws in PFS's claims which lead to disputes as to whether radiation dose limits will be met if PFS employs a 2000 year DBE.

8. This declaration relates to whether the PFS design basis for the Holtec International Inc. ("Holtec") HI-STORM 100 cask system provides reasonable assurance that the health and safety of the public and onsite workers will be protected if the casks are subjected to the peak ground accelerations from a 2,000 year mean annual return period earthquake at the PFS site.

9. NRC issued a certificate of compliance ("CoC") for the HI-STORM 100 cask effective May 31, 2000. 65 FR 25241 (2000). By issuing a CoC, NRC determined that the "HI-STORM 100 cask system, as designed and when fabricated and used [for general licenses] in accordance with the conditions specified in its CoC, meets the requirements of 10 CFR Part 72." Id..

10. The site specific conditions at the PFS facility are outside the bounds of the generic CoC for the HI-STORM 100 cask system. Therefore, in order to use the HI-STORM 100 system, PFS must conduct a site specific analysis to determine whether the performance of the casks at the PFS site are adequate to protect health and safety. There are serious shortcomings in PFS's site specific analysis. *See* Joint Declaration of Bartlett, Ostadan, and Khan, Exhibit B ("Utah Joint Dec.").

11. In my opinion, and as discussed below, PFS has not shown that unanchored HI-STORM 100 casks will "reasonably maintain confinement of radioactive material" under off-normal and credible accident conditions at the proposed PFS site as required by 10 CFR § 72.236. Further, PFS and cask designer, Holtec, have not quantified the consequences of a potential 2,000 year mean annual return period, 10,000 year return period, or deterministic earthquakes that could take place at the proposed PFS site.

12. There are significant differences between the facts and conditions used to support the HI-STORM CoC and those at the PFS site, for example:

    a. The calculated ground motions at the PFS facility for a 2,000 year return period earthquake are 0.711 g horizontal and 0.695 g vertical (SAR at 2.6-107, Rev. 22). As described below, the bounding ground motions in the CoC for purpose determining the maximum zero point acceleration that will not cause incipient tipping are bounded by a horizontal acceleration of .445 g

and vertical acceleration of .16 g.

In its HI-STORM CoC analysis, Holtec treated a loaded cask as a rigid body and set up the following inequality,

$G_H + \mu G_v \leq \mu$ (where μ is 0.53, G_H is the resultant horizontal acceleration, and G_V is the resultant vertical acceleration),

stating that the maximum g loading a cask could take without tipping would occur when the horizontal force acting at the center of gravity of the cask just balances the vertical force acting at the pivot point. Any horizontal force greater than this would cause tipping in this rigid body assumption.

In the above formula $\mu = r/H$. In the HI-STORM CoC Holtec reduced the value of r/H from 0.56 to 0.53, thereby giving a bounding horizontal acceleration of .445 g (with .16 g as the corresponding vertical acceleration). CoC, Appx B at 3-8.

As can be seen from the above, the DBE ground motions for the PFS site are significantly higher than those specified in the CoC for the HI-STORM 100 cask.

b. There is an inconsistency between the occupancy time at the controlled area boundary used in the Holtec CoC and that used at the PFS site. The Holtec CoC used a duration time of 8,760 hours per year whereas at the PFS site only 2,000 hours per year was used to compute dose exposure at the fence post. *See* ¶ 11 below.

c. Holtec calculated the dose consequences in a non-mechanistic single cask tip over event, whereas at PFS the entire field of casks could tip over under the accelerations caused by the DBE. *See* Utah Joint Dec.

13. Failure to quantify the consequences of a potential 2,000 year return period, 10,000 year return period, or deterministic earthquake is fatal to PFS' and Holtec's conclusions because the calculated ground motions for a 2,000 year return period earthquake (of 0.711 g horizontal and 0.695 g vertical) at the PFS facility are so far outside the bounds of those used to support the Holtec CoC that it is fair to conclude that there is no quantification of the consequences of what will occur at ground motions of approximately 0.7g.

14. PFS calculated a 5.85 mrem/year dose for a 2,000 hour/year occupancy time at the

controlled area boundary under normal operating conditions.[1] PFS Joint Dec. ¶ 23. PFS significantly underestimated the dose rate. To assure that the public is protected, PFS must calculate a radiation dose assuming a hypothetical individual is located at the site boundary the entire year or 8,760 hours/year because PFS cannot control who is at the site boundary or for what length of time.[2] In the CoC for the HI-STORM 100 System, NRC Staff agreed with my position in response to comment B.18, stating: "The NRC agrees that 8,760 hours should be used [for estimating the dose at the site boundary]." *See* 65 FR 25241, 25245. Thus, using a 8,760 hour/year assumption is consistent with the NRC Staff position in approving the HI-STORM 100 CoC.

15. I calculated the correct annual dose rate assuming a hypothetical individual remained at the site boundary for 8,760 hour. The dose rate is 5.85 mrem/year * 8,760 hours/year ÷ 2,000 hours/year = 25.6 mrem/year, which is in excess of the allowable 25 mrem/year specified in 10 CFR § 72.104(a). This is the dose rate under normal operating conditions, absent a seismic event.

16. In addition to PFS's selection of 2,000 hour per year exposure duration being at odds with the Holtec CoC, it is also unjustified. The PFS facility is expected to have an operational life of at least 40 years. SER at 1-1. The site is located on the northwestern edge of the Skull Valley reservation abutting privately owned property. In my opinion it is nonconservative and unrealistic to analyze dose exposure for 40 hours per week for 50 weeks a year (*i.e.*, 2,000 hours per year). There should be an expectation that residential housing will abut the PFS site boundary. Moreover, by definition an "uncontrolled" area is an area not controlled by PFS.

17. Holtec and NRC Staff considered the HI-STORM tip over analysis as a non-mechanistic event.[3] *See* HI-STORM 100 Safety Evaluation Report ("HI-STORM

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[1] The Holtec dose calculation at the PFS controlled area boundary is inconsistent and less conservative than other Holtec dose calculations which likely used an occupancy rate of 2,080 hour/year. *See* Redmond Tr. at 40.

[2] Although PFS does not control property beyond the site boundary, it calculated a dose rate at a distance of 2 miles from the site boundary. PFS Safety Evaluation Report ("SER") (September 2000) at 7-6.

[3] "In the absence of an identified [cask tipover] hazard" NRC allows a non-mechanistic cask tipover analysis. NUREG-1536, *Standard Review Plan for Dry Cask Storage Systems*, at 2-9. However, a non-mechanistic tipover analysis is no longer acceptable because the HI-STORM 100 casks will likely tipover under peak ground accelerations for a 2,000 year mean annual return period earthquake.

SER") at 11.2.4.1, HI-STORM 100 Safety Analysis Report ("SAR") at 11.2.3. Because the dose at the controlled area boundary is already slightly greater than 25 mrem/year assuming an exposure duration of 8,760 hours/year, any further increase will put this dose that much higher than the limits allowed in 10 CFR § 72.104(a).

18. PFS acknowledges that a tip-over accident could "cause localized damage [including crushing of the concrete and associated micro-cracking] to the radial concrete shield and outer steel shell where the storage cask impacts the surface." *See* PFS Joint Dec. ¶ 25. Holtec in fact states that the "overpack surface dose rate . . . could increase due to the [tipover] damage." HI-STORM 100 TSAR at 11.2-8. Contrary to the HI-STORM 100 TSAR and without any quantified analysis, PFS claims that no "noticeable increase" in radiation dose would occur at the site boundary. PFS Joint Dec. ¶ 25. PFS' radiation dose expert is unaware of any calculations that estimate the radiation consequences of concrete cracking. Redmond Tr. at 46, 47. In my opinion, there is no support for PFS's claim.

19. To determine whether fuel assemblies would be damaged in a tipover event, Holtec calculated the deceleration of the top edge of the canister as the cask struck the cement pad. *See*, e.g., HI-STORM 100 SAR, Section 3.A. In its hypothetical tipover analysis, Holtec identified "a center of gravity over pivot point" configuration as its starting point, assuming that the initial angular velocity was zero. HI-STORM 100 SAR, Rev. 8, Section 3.A.6. There are numerous problems with Holtec's analysis and the conclusion PFS draws from it.

a. During an earthquake, PFS' witnesses conclude "the initial linear velocity of the cask centroid in the plane of precession . . . would not be significantly increased over the [hypothetical] tip-over condition already studied." PFS Joint Dec. ¶ 20. PFS again provides no supporting calculations and in my opinion, PFS's starting premise of zero initial angular velocity is unfounded.

b. If cask tip over results from earthquake accelerations, the initial angular velocity will be greater than zero. *See* Utah Joint Dec. ¶ 87. From this you can conclude that the top of the canister will decelerate at greater than 45 g, in exceedence of the 45 g design basis, thereby damaging the fuel assemblies; also the HI-STORM 100 cask will flatten more than contemplated by PFS. Claims that the "MPC has a very substantial margin built into it" are unsubstantiated; PFS has again failed to support its site specific use of the HI-STORM cask with any calculations or test data. *See* PFS Joint Dec. ¶ 20. Therefore, PFS has not substantiated whether or not the confinement boundary would be breached in a 2,000 year earthquake or a 10,000 year earthquake. *See* id.

20. Since the initial angular velocity is expected to be greater than zero as the cask center

of gravity passes the pivot point, the HI-STORM 100 cask will also flatten more than contemplated by PFS. Although PFS claims that the "MPC has a very substantial margin built into it," it again fails to support its claim with any calculations or test data. *See* PFS Joint Dec. ¶ 20. Furthermore, PFS also acknowledges that the "roundness" of the casks could be reduced following cask tipover. PFS Joint Dec. ¶ 26. However, in the event of cask tipover, PFS has not quantified the amount of concrete flattening or the resultant reduction of gamma and neutron shielding. Thus, the potential consequence of a HI-STORM 100 cask tipover is another unresolved critical safety issue that must be addressed prior to determining or justifying the appropriate site specific design basis earthquake.

21. If a HI-STORM 100 tips over, PFS further states that the roundness of the storage cask could only be reduced in the radial area of the impact. PFS Joint Dec. ¶ 26. PFS witness, Dr. Redmond, then implies that any increase in dose from the reduction in radiation shielding caused by the flattening or localized deformation is inconsequential because the increase in dose will occur between the cask and the ground. Redmond Tr. at 48; PFS Joint Dec. ¶ 26. First, PFS has performed no analysis to show that the deformation will be in contact with the ground. Second, when the HI-STORM 100 casks are in fact uprighted, the flattened area of the cask (localized deformation) will not face the ground. PFS has failed to calculate the potential increase in dose at the site boundary or to workers from such casks.

22. Under a HI-STORM 100 cask tipover event, Holtec has also not quantified the amount of stretching of the metal outer surface, and the amount of cracking of the cement. Cracking will lead to an increased gamma dose at the fence post and an increased neutron and gamma dose to PFS workers since gamma rays and neutrons will pass more easily through this less shielded region. The potential increase in radiation dose at the fence post must be quantified before the design basis earthquake is specified.

23. The analysis performed by Holtec in the HI-STORM TSAR does not bound cask tip-over resulting from an earthquake affecting the PFS facility because the Holtec analysis evaluates only one cask being tipped over. At a facility with up to 4,000 casks, it is highly unlikely that only one HI-STORM 100 cask will tipover as a result of peak ground accelerations from a 2,000 year mean annual return period earthquake affecting the PFS facility. *See e.g.* Utah Joint Dec. ¶ 74.

24. If the HI-STORM 100 casks tipover such that the bottom of a row of casks face the fence post, the direct gamma dose at the fence post will increase. As seen in the drawing, attached hereto as Attachment A, a ring or torus of the bottom of the HI-STORM 100 cask provides reduced shielding. This is not a region where the fuel is located, but indirect gamma rays and neutrons will stream through the bottom of the cask. PFS has not calculated the dose at the boundary from the bottoms of tipped

over HI-STORM 100 casks. Redmond Tr. at 50. In collaboration with my colleague, Matthew Lamb, I performed preliminary rough calculations for the reduced shielding caused by exposure from the bottom of the casks at the site boundary.[4] *See* Attachment B (dose calculations). My calculations show the dose rate due to gamma rays will increase between 1.8 and 18 times that calculated by PFS at the site boundary assuming a 2,000 hour year, and between 7.7 and 77 times that calculated by PFS assuming an 8,760 hour year. Because of the likelihood that HI-STORM 100 casks will tipover during a 2,000 year mean annual return period earthquake, in order to justify that there will be no effect to health and safety from using a 2000 year DBE, in my opinion PFS must calculate a bounding radiation dose at the fence line and to workers.

25. In addition to cracking of the concrete cask, the issue of cask heat-up and loss of concrete shielding must be addressed by PFS. The HI-STORM 100 cask is designed to be cooled by a "chimney effect." Cooler air enters the bottom vent and rises and is released from the top vent. If the casks tip over, the chimney effect is reduced dramatically and this is equivalent to the intake vents being blocked. Holtec calculations show that after 33 hours of 100% air inlet blockage, the concrete temperature will exceed the short-term limit of 350°F specified in the CoC for the HI-STORM 100 cask. *See* HI-STORM 100 SAR, pp. 1.D-4, Table 1.D.1 (Rev 10). The CoC temperature limit is established to ensure the continued effectiveness of the neutron shielding by ensuring the water does not evaporate from the concrete, reducing the amount of hydrogen available for neutron capture. *See* Redmond Tr. at 60-61. PFS has not analyzed the effects of an increase of neutron dose to on-site workers from the prolonged tip over of HI-STORM 100 casks.

26. At the PFS site there is the likelihood that the HI-STORM 100 casks will tip over during a 2,000 year return period DBE. Utah Joint Dec. ¶ 74. In my opinion PFS could not upright all the casks within the time limits imposed by the CoC and this will result in the potential increase in neutron dose to workers.

a. The HI-STORM casks are approximately 20 feet high, 11 feet in diameter and weigh about 175 tons. PFS SAR, Table 4.2-2, Rev. 12. In restoring the casks to their original and upright position, the configuration of the casks on the pad dictates that a crane would have to work from the outside perimeter of the pads towards the center of the pads. Obtaining a crane capable of lifting 175 tons and transporting it to the Skull Valley site, maneuvering around other casks, then uprighting and re-positioning each 175 ton cask on the pad would result in only a few casks, if any, being restored to their original pad position within 33 hours. Workers would then have to operate

[4] I am unaware of any dose calculations performed by Holtec.

in an increased neutron dose environment.

b. The CoC temperature limit is established to ensure the continued effectiveness of the neutron shielding by ensuring the water does not evaporate from the concrete, reducing the amount of hydrogen available for neutron capture. *See* Redmond Tr. at 60-61. In collaboration with my colleague Matthew Lamb, I performed calculations, included herein as Attachment C, that show increased neutron dose due to reduced shielding. These calculations estimate an increase in dose to workers due to neutrons of up to 57.3 times greater than the value calculated by Holtec of 1.88 mrem/hour 1 meter from the cask mid-height if all of the water evaporates from a HI-STORM cask. This would result in a worker dose of approximately 108 mrem/hour. A worker exposed to this for just over 46 hours would exceed the 5 rem/year occupational dose rate specified in 10 CFR § 20.1201, Subpart C.

27. The Altran Report, Attachment F to the Utah Joint Declaration concludes that the HI-STORM 100 casks will tipover under peak ground accelerations induced by a 2,000 year earthquake at the PFS facility. Even if the casks do not tipover, the casks will likely still slide approximately 370 inches in the x direction and 230 inches in the y direction and uplifted 27 inches. *See* Utah Joint Dec. ¶ 26. Contrary to PFS's claims, the casks will not move in phase with each other. Id. ¶¶ 75, 79. Under these conditions the casks will slide and collide with each other. PFS has not evaluated the damage nor calculated dose increase from colliding casks. *See* PFS Joint Dec. ¶¶ 14, 17.

28. Also, the HI-STORM 100 cask will likely be lifted up to 27 inches if subjected to peak ground accelerations induced by a 2,000 year earthquake at the PFS facility. *See* Utah Joint Dec. ¶ 26. The HI-STORM 100 cask was analyzed and determined capable of withstanding only a drop of 11 inches. *See* HI-STORM 100 CoC at 5.0-4. PFS has not demonstrated its requested design basis ground motion exemption will not result in potential damage to the canister or cask.

29. Finally, I have reviewed the cask drop calculations recently supplied by Holtec, HI-2002572, *Evaluation of the Confinement Integrity of a Loaded Holtec MPC Under a Postulated Drop Event* (Nov. 30, 2000). These calculations assume a HI-TRAC cask will drop vertically from a 25-foot height onto a concrete base. It is more likely that the HI-TRAC cask would drop on edge. The shear stresses would then be considerably more severe than in a vertical drop. The NRC Staff admits that "the SAR drop analysis does not include examination of a corner drop." *See* HOLTEC SER at 3-10. If the canister experiences a "corner drop," then PFS has not evaluated whether the canister welds would be impaired, exposing the canister contents to the external environment. This issue must be addressed prior to establishing the design basis

earthquake.

30. Based on the above, I do not agree that the limited analysis performed by Holtec and PFS is conservative or bounding. In the instances discussed above, the HI-STORM cask would be operated under conditions that are outside the parameters analyzed in the SAR and SER, and would lead to doses at the fence post that exceed regulatory limits. Thus, PFS has not shown that its requested design basis ground motion will not endanger life or property or is otherwise in the public interest as required by 10 CFR § 72.7 or will not jeopardize the health and safety of on-site workers.

Executed this 7th day of December, 2001.

By ______________________
Marvin Resnikoff, Ph.D.

earthquake.

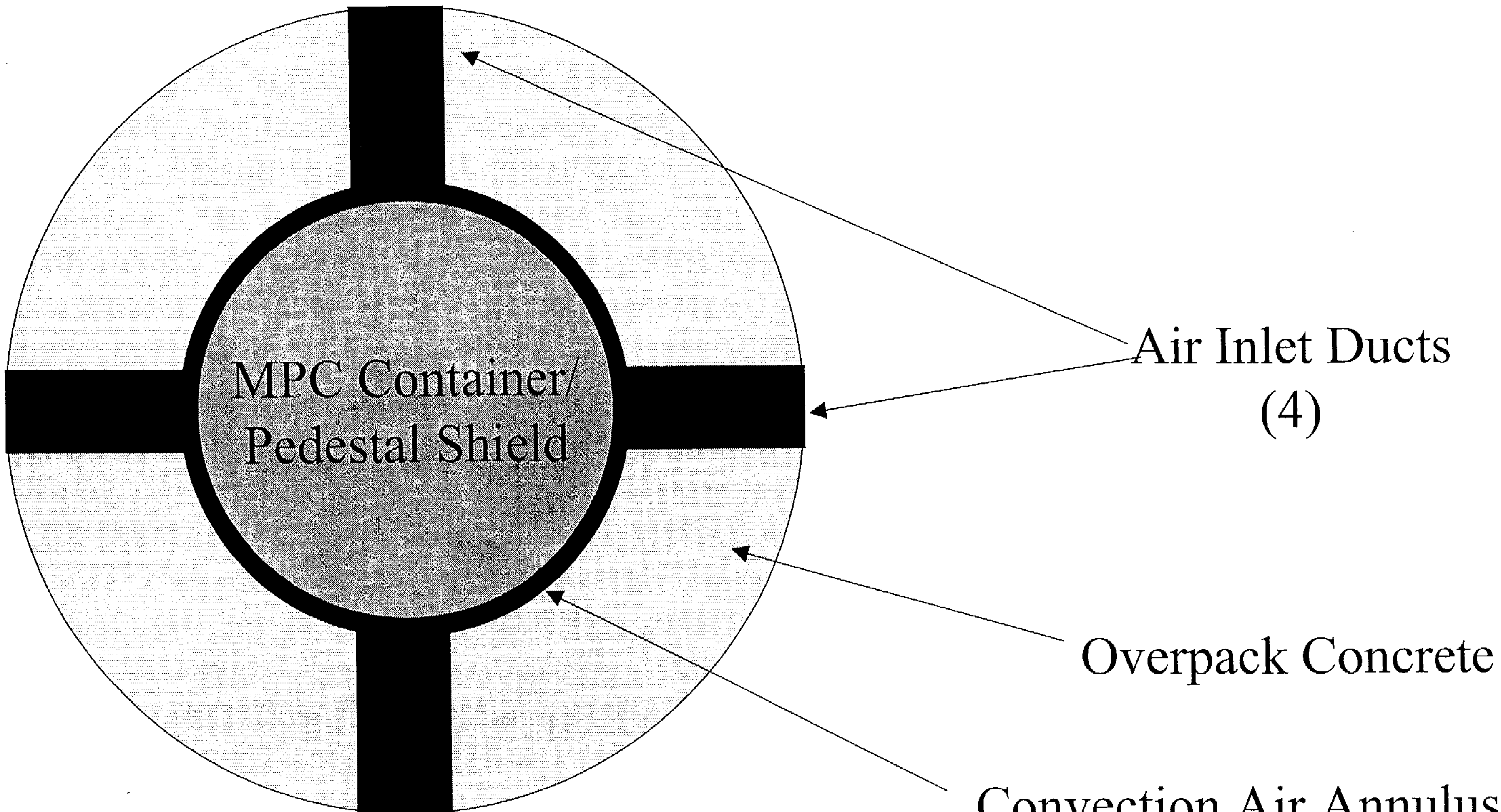
30. Based on the above, I do not agree that the limited analysis performed by Holtec and PFS is conservative or bounding. In the instances discussed above, the HI-STORM cask would be operated under conditions that are outside the parameters analyzed in the SAR and SER, and would lead to doses at the fence post that exceed regulatory limits. Thus, PFS has not shown that its requested design basis ground motion will not endanger life or property or is otherwise in the public interest as required by 10 CFR § 72.7 or will not jeopardize the health and safety of on-site workers.

Executed this 7th day of December, 2001.

By ______________________________
Marvin Resnikoff, Ph.D.

A

Schematic Cross Section of HI-STORM 100 Cask Bottom



Note: Bottom of cask contains approximately 3 inches of steel, not pictured in this drawing

B

Rough Calculations: Dose Emanating from Bottom of Tipped-Over Cask

This calculation attempts to estimate the gamma dose emanating from the bottom of a single tipped-over HI-STORM 100 cask. To do this, I will attempt to calculate the dose rate at the bottom of an unshielded MPC container, given the dose rate at the bottom of a HI-TRAC transfer cask. Once this has been estimated, I will estimate the dose rate on the outside of the bottom of a HI-STORM cask which has been tipped over. I will first calculate the dose rate due to Co-60, then for fuel gammas (assuming all the dose is from Cs-137).

According to the SAR for the HI-STORM 100 cask, the dose rate adjacent to the bottom of a HI-TRAC transfer cask filled with design-basis fuel is 3058.38 mrem/hour. The "fuel gamma" dose rate at the same point is 238.28 mrem/hour. From drawings of the HI-TRAC contained in the HI-STORM SAR, this dose rate is after passing through shielding (2.75 inches of steel and approximately 1 inch of lead, in addition to the meter of air). To calculate the dose before attenuation by the shielding, the following equation is used, solving for D_o.

$$D(x) = D_0 \times e^{-\lambda_s x}$$

The shielding coefficient λ_s can be calculated with the tenth-value layer as follows:

λ_s (1/cm) = ln10/tenth-value layer

Table 1: Tenth Value Layers of Radionuclides

Radionuclide	Gamma Energy	Tenth-value layer			Shielding coefficient		
		Concrete	Steel	Lead	Concrete	Steel	Lead
	(MeV)	(cm)	(cm)	(cm)	(1/cm)	(1/cm)	(1/cm)
Cs-137	0.66	15.7	5.3	2.1	0.1467	0.4345	1.0965
Co-60	1.17, 1.33	20.6	6.9	4.0	0.1118	0.3337	0.5756

1). Co-60

A). Inside Dose Calculation

As was previously mentioned, the dose rate at the bottom of a HI-TRAC cask due to CO-60 was given as 3058.38 mrem/hour adjacent to the cask. Using the above shielding coefficients for steel and lead (ignoring any attenuation in air), we obtain the following value for the dose rate inside the shielding.

$$D_{inside} = \frac{D_{outside}}{e^{-(\lambda_L T_L + \lambda_{st} T_{st})}};$$

where the thickness of steel and lead were previously given (2.75 inches and 1 inch, respectively).

Using these values, we obtain a dose rate inside the shielding of **135,748.5 mrem/hour**.

B). Annulus Area Calculation

A central assumption in this calculation is that the majority of the gamma ray dose emanating from the bottom of the cask will travel through the annulus which, under normal conditions, allows for convective heat transfer. The annulus creates a ring in which it is possible for streams of radiation to pass through without being shielded by any concrete, with the only shielding being provided by the steel baseplate, estimated to be 3 inches thick.

The formula for calculating the area of an annulus is as follows:

$A = \pi(r_o^2 - r_i^2)$;

In this case, r_o = 36.75 inches and r_i = 34.1875 inches, based on drawings in the HI-STORM SAR.

Using these numbers, the area of the annulus is calculated to be 571 in^2.

Next, we need to estimate what percentage of the source term is able to stream through this annulus. As a bounding case, we take the percentage of area occupied by the annulus when compared to the area of the bottom of the HI-STORM that is considered part of the source term. This is bounding because we assume that the entire radiation dose is contained in the area bounded by the outer diameter of the annulus, thus arriving at the maximum fractional area for the annulus.

The area bounded by the outer radius (36.75 inches) is 4243 in^2. Therefore the annulus occupies 13.45% of this area.

C). Computation of Dose outside HI-STORM Annulus

We need to make and assumption regarding the fraction of radiation that can emanate directly through the annulus. As a first approximation, we make the assumption that 13.45% of the radiation emanating from the MPC container travels directly through the annulus, unencumbered by any concrete. This is an overestimate, since the MPC container stops at the inner radius of the annulus, meaning that there is no direct path through it. Therefore, as a second approximation, we reduce the area by a factor of 10.

Next, we recomputed the dose rate, this time on the outside of the HI-STORM cask, after passing through approximately 3 inches of steel. From the previous table, the shielding coefficient for Co-60 through steel is .3337 cm^{-1}. The shielding coefficient for Co-60 in air is 7.12e-5 cm^{-1}, which will be applied to obtain a dose rate at 1 meter from the bottom of the cask. The results are provided below for both cases (13.45% and 1.345% of the radiation emanating through the annulus, respectively.)

Table 2: Tipped-Over Cask Dose Rate from Co-60

Case	% of source term unshielded by concrete	Dose Rate inside shielding (mrem/hr)	Dose rate at 1 meter outside HI-STORM bottom (mrem/hour)
1	13.45	18,258	1426
2	1.345	1,825.8	142.6

D). Estimate of dose at site boundary assuming a line of casks overturning

If a line of casks overturn, we can estimate the dose at the boundary assuming a line source. The largest "line source" would consist of 80 casks overturning, creating a line 1,520 feet (463.3 meters). If we assume all 80 tip over so their bottoms are perpendicular to and facing the site boundary, we can estimate a linear source term for the two cases analyzed above as follows:

Table 3: Development of Line Source Term for Co-60

Case	Single Cask Dose rate @ 1 meter (mrem/hour)	Linear Source term (mrem/meter-hour)
1	1426	246.2
2	142.6	24.62

The dose rate from a linear source term decreases linearly with distance by the following formula:

$I_2 = I_1\theta/h$;

Where theta is the angle in the following diagram. In this case, theta =.79 rad, or approximately 45 degrees and h is equal to 555 meters.

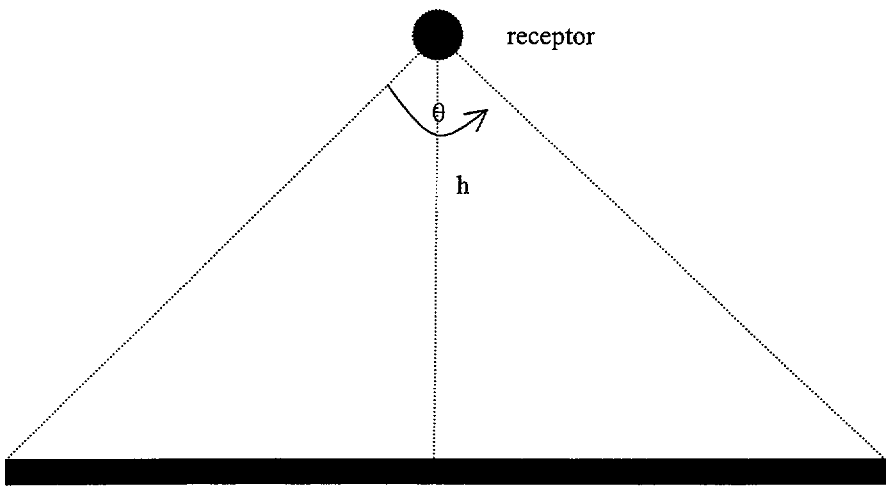


Figure 1: Explanation of angle in line source calculation

Using this formula, we obtain the following dose rates at the site boundary for the two cases.

Table 4: Dose Rate at Controlled Area Boundary from Co-60; No Attenuation in Air

Case	Dose Rate @ 555 meters		
	mrem/hour	mrem/year, 2000 hours/year	mrem/year, 8760 hours/year
1	.35	701	3070
2	.035	70.1	307

However, we did not take into account the shielding by the steel of the casks in this calculation. Therefore, we will discount value using Beer's Law, correcting for the broad-beam geometry.

$I/I_o = Be^{-ut}$

In this case, μ=7.12e-5 cm^{-1} for Co-60 through air and the buildup factor is approximately 6.7 (based on values from table 6.5.1 of *Handbook of Health Physics and Radiological Health*. Using this results in a reduction by a factor of .133. Thus, the final dose rate due to the postulated line source is given as:

Table 5: Dose Rate at Controlled Area Boundary from Co-60

Case	Dose Rate @ 555 meters, assuming attenuation in air		
	mrem/hour	mrem/year, 2000 hours/year	mrem/year, 8760 hours/year
1	.047	93.6	410
2	.0047	9.36	41.0

2). Cs-137

A). Inside Dose Calculation

The fuel gamma dose rate at the bottom of a HI-TRAC cask was given in the HI-STORM SAR as 238.28 mrem/hour adjacent to the cask. Using the above shielding coefficients for steel and lead (ignoring any attenuation in air), we obtain the following value for the dose rate inside the shielding.

$$D_{inside} = \frac{D_{outside}}{e^{-(\lambda_L T_L + \lambda_{st} T_{st})}} \; ;$$

where the thickness of steel and lead were previously given (2.75 inches and 1 inch, respectively).

Using these values, we obtain a dose rate inside the shielding of **80,301.1 mrem/hour**.

B). Annulus Area Calculation

A central assumption in this calculation is that the majority of the gamma ray dose emanating from the bottom of the cask will travel through the annulus which, under normal conditions, allows for convective heat transfer. The annulus creates a ring in which it is possible for streams of radiation to pass through without being shielded by any concrete, with the only shielding being provided by the steel baseplate, estimated to be 3 inches thick.

The formula for calculating the area of an annulus is as follows:

$A = \pi(r_o^2 - r_i^2)$;

In this case, r_o = 36.75 inches and r_i = 34.1875 inches, based on drawings in the HI-STORM SAR.

Using these numbers, the area of the annulus is calculated to be 571 in^2.

Next, we need to estimate what percentage of the source term is able to stream through this annulus. As a bounding case, we take the percentage of area occupied by the annulus when compared to the area of the bottom of the HI-STORM that is considered part of the source term. This is bounding because we assume that the entire radiation dose is contained in the area bounded by the outer diameter of the annulus, thus arriving at the maximum fractional area for the annulus.

The area bounded by the outer radius (36.75 inches) is 4243 in^2. Therefore the annulus occupies 13.45% of this area.

C). Computation of Dose outside HI-STORM Annulus

We need to make and assumption regarding the fraction of radiation that can emanate directly through the annulus. As a first approximation, we make the assumption that 13.45% of the radiation emanating from the MPC container travels directly through the annulus, unencumbered by any concrete. This is an overestimate, since the MPC container stops at the inner radius of the annulus, meaning that there is no direct path through it. Therefore, as a second approximation, we reduce the area by a factor of 10.

Next, we recomputed the dose rate, this time on the outside of the HI-STORM cask, after passing through approximately 3 inches of steel. From the previous table, the shielding coefficient for Cs-137 through steel is .4345 cm^{-1}. The shielding coefficient for Cs-137 in air is 9.31e-5 cm^{-1}, which will be applied to obtain a dose rate at 1 meter from the bottom of the cask. The results are provided below for both cases (13.45% and 1.345% of the radiation emanating through the annulus, respectively.)

Table 6: Tipped-Over Cask Dose Rate from Cs-137

Case	% of source term unshielded by concrete	Dose Rate inside shielding (mrem/hr)	Dose rate at 1 meter outside HI-STORM bottom (mrem/hour)
1	13.45	10,801	391
2	1.345	1,825.8	39.1

3). Estimate of dose at site boundary assuming a line of casks overturning

If a line of casks overturn, we can estimate the dose at the boundary assuming a line source. The largest "line source" would consist of 80 casks overturning, creating a line 1,520 feet (463.3 meters). If we assume all 80 tip over so their bottoms are perpendicular to and facing the site boundary, we can estimate a linear source term for the two cases analyzed above as follows:

Table 7: Development of Line Source Term for Cs-137

Case	Single Cask Dose rate @ 1 meter (mrem/hour)	Linear Source term (mrem/meter-hour)
1	394	67.5
2	39.4	6.75

The dose rate from a linear source term decreases linearly with distance by the following formula:

$I_2 = I_1\theta/h$;

Using this formula, we obtain the following dose rates due to Cs-137 at the site boundary for the two cases.

Table 8: Dose Rate at Controlled Area Boundary from Cs-137; No Attenuation in Air

Case	Dose Rate @ 555 meters		
	mrem/hour	mrem/year, 2000 hours/year	mrem/year, 8760 hours/year
1	.097	194	849
2	.0097	19.4	84.9

However, we did not take into account the shielding by the steel of the casks in this calculation. Therefore, we will discount value using Beer's Law, correcting for the broad-beam geometry.

$I/I_o = Be^{-ut}$

In this case, μ=9.67e-5 cm^{-1} for Cs-137 through air and the buildup factor is approximately 10 (based on values from table 6.5.1 of *Handbook of Health Physics and Radiological Health*. Using this results in a reduction by a factor of .049. Thus, the final Cs-137 dose rate due to the postulated line source is given as:

Table 9: Dose Rate at Controlled Area Boundary from Cs-137

Case	Dose Rate @ 555 meters, assuming attenuation in air		
	mrem/hour	mrem/year, 2000 hours/year	mrem/year, 8760 hours/year
1	.0048	9.52	41.7
2	.00048	0.952	4.17

A

dding up the doses from Co-60 and Cs-137, we obtain the following dose rates at the controlled area boundary"

Table 10: Estimated Dose Rate at Controlled Area Boundary, Multiple Cask Tip-Over

Case	Dose Rate @ 555 meters, assuming attenuation in air		
	mrem/hour	mrem/year, 2000 hours/year	mrem/year, 8760 hours/year
1	.05	103	451
2	.005	10.3	45.1

C

Calculation of Neutron Dose at Elevated Concrete Temperatures

The following calculations are based on information from:

Kaplan, M.F, 1989. *Concrete Radiation Shielding: Nuclear Physics, Concrete Properties, Design and Construction.* Avon, Great Britain: Longman Scientific and Technical.

In order to calculate the loss of neutron shielding associated with evaporation of water in the concrete, it is necessary to discuss briefly the interaction mechanisms at play. Neutrons are grouped into categories based on their energy levels: thermal neutrons have low energies and can be absorbed or captured, while fast neutrons have higher energy and need to be slowed before capture. Table 1, below, shows the energy distribution of fast neutrons used as the design basis fuel in the Holtec HI-STORM SAR.

Table 1: Neutron Energy Distribution of Design Basis Fuel: From HI-STORM SAR

Lower Energy (MeV)	Upper Energy (MeV)	35,000 MWD/MTU, 5-year cooled		45,000 MWD/MTU, 5-year cooled		45,000 MWD/MTU, 9-year cooled	
		Neutrons/s	% of total	Neutrons/s	% of total	Neutrons/s	% of total
0.1	0.4	7.19E+06	3.8%	1.63E+07	3.8%	1.40E+07	3.8%
0.4	0.9	3.68E+07	19.3%	8.33E+07	19.4%	7.15E+07	19.4%
0.9	1.4	3.37E+07	17.7%	7.63E+07	17.8%	6.55E+07	17.7%
1.4	1.85	2.49E+07	13.1%	5.62E+07	13.1%	4.84E+07	13.1%
1.85	3	4.42E+07	23.2%	9.92E+07	23.1%	8.56E+07	23.2%
3	6.43	3.99E+07	21.0%	9.01E+07	21.0%	7.75E+07	21.0%
6.43	20	3.52E+06	1.9%	7.98E+06	1.9%	6.85E+06	1.9%
Totals		1.90E+08	100.0%	4.29E+08	100.0%	3.69E+08	100.0%

For fast neutrons, the *effective removal cross section* (Σ_R) describes the removal of neutrons by a shielding mechanism. It is used in the following equation:

$I=I_o e^{-\Sigma r T}$; where T is the thickness of the shielding.

Neutron cross-sections are greatly affected by the neutron energy and the atomic weight of the various chemical elements in the shielding medium. However, according to Kaplan's text, the effective removal cross section is considered to be approximately constant for neutron energies between 2 and 12 MeV (Kaplan, pp 235) which accounts for approximately 50% of the neutron distribution given above. For comparison, we will assume the cross sections are constant throughout the range of energies listed above.

For materials (such as concrete) which are comprised of a variety of elements, the total effective removal cross section can be calculated as the sum of the weighted averages of the individual effective removal cross sections. Kaplan has listed various effective removal cross sections for components commonly found in concrete. In Table 2, below, we list the elemental makeup of "ordinary concrete" used in the Kaplan text and the concrete to be used for the HI-STORM 100 overpack. The HI-STORM concrete data is taken from Table 5.3.2 of the HI-STORM SAR

Table 2: Elemental Makeup of Concrete and Neutron Removal Cross Sections

	Ordinary Concrete			**Holtec HI-STORM 100 Overpack Concrete**		
Element	g element/cm^3 concrete	Σ_R/ρ (cm^2/g)	Σ_R (cm^{-1})	G element/cm^3 concrete	Σ_R/ρ (cm^2/g)	Σ_R (cm^{-1})
H	0.015	0.598	0.0090	0.0141	0.598	0.0084
O	1.057	0.0346	0.0366	1.175	0.0346	0.0407
Na	0.041	0.0341	0.0014	0.03995	0.0341	0.001362295

Mg	0.085	0.0333	0.0028	--		
Al	0.137	0.0292	0.0040	0.1128	0.0292	0.00329376
Si	0.487	0.0295	0.0144	0.74025	0.0295	0.021837375
P	0.002	0.0283	0.0001	--		
S	0.002	0.0275	0.0001	--		
K	0.015	0.0247	0.0004	0.04465	0.0247	0.001102855
Ca	0.295	0.0243	0.0072	0.19505	0.0243	0.004739715
Ti	0.011	0.022	0.0002	--		
Mn	0.003	0.0202	0.0001	--		
Fe	0.178	0.0214	0.0038	0.0282	0.0214	0.00060348
Concrete Properties	**2.328**		**0.0799**	**2.35**		**0.0820**

The HI-STORM 100 cask contains a concrete layer 67.95 cm thick. Therefore, we can estimate the attenuation of neutrons by this shielding by solving the exponential absorption equation:

$I/I_o = e^{-\Sigma r T}$

For the "ordinary concrete" used in the Kaplan text, $I/I_o = 0.0044$, while for the HI-STORM overpack concrete $I/I_o = 0.0038$. Further, the HI-STORM 100 SAR has specified a neutron dose rate adjacent to the mid-height of the HI-STORM overpack as 1.88 mrem/hour assuming 45,000 MWD/MTU, 5-year cooled MPC-24 fuel. Using this value as I in the above equation we can solve for I_o to estimate the neutron dose rate assuming no shielding.

I_o = 1.88 mrem/hour x $\exp^{.0820 \times 67.95}$ = **495 mrem/hour assuming no neutron shielding by concrete**

Temperature Effects of Neutron Shielding Ability of Concrete

Increased temperature of concrete results in a decrease in the amount of water, which results in an increase in the neutron flux density transmitted through a concrete shield of given thickness. The Kaplan text presents the results of experiments in which the effective removal cross section of concrete were estimated (and experimentally measured) at temperature values of room temperature, 100°C, 200°C, and 300°C. Below, I use these results to estimate the effect on the HI-STORM concrete, assuming that an equivalent loss of hydrogen (by weight %) in the HI-STORM overpack concrete as in the experimental concrete. The same calculations were performed as appear in Table 2 of this report: the relative proportions of the various elements are not included in Table 3 for brevity.

Table 3: Effect of Temperature Increase on Neutron Shielding

		"Ordinary Concrete" used in Kaplan text	HI-STORM 100 Overpack Concrete
Unheated (as cured)	ρ, g/cm^3	2.328	2.35
	H density, g H/cm^3 concrete	0.015	0.0141
	Σ_R (calculated), cm^{-1}	0.0801	0.0820
	Σ_R (measured), cm^{-1}	0.0780	--
100 °C	ρ, g/cm^3	2.258	2.283
	H density, g H/cm^3 concrete	0.007	0.00658
	Σ_R (calculated), cm^{-1}	0.0731	0.0755
	Σ_R (measured), cm^{-1}	0.0735	--
200°C	ρ, g/cm^3	2.238	2.266
	H density, g H/cm^3 concrete	0.005	0.0047

	Σ_R (calculated), cm^{-1}	0.0713	0.0738
	Σ_R (measured), cm^{-1}	0.0724	--
300°C	ρ, g/cm^3	2.227	2.258
	H density, g H/cm^3 concrete	0.004	0.00376
	Σ_R (calculated), cm^{-1}	0.0704	0.0730
	Σ_R (measured), cm^{-1}	0.0702	--
All Water Evaporates	ρ, g/cm^3	2.194	2.242
	H density, g H/cm^3 concrete	0	0
	Σ_R (calculated), cm^{-1}	.0668	.0703
	Σ_R (measured), cm^{-1}	--	--

Previously, we have estimated the unshielded dose rate to be 495 mrem/hour. To estimate what the shielded dose rate is as a function of temperature, we simply repeat the calculation: $I=I_o e^{-\Sigma rT}$ for the varying temperatures. This is done for the HI-STORM 100 cask below.

Table 4: Estimated Dose Rates Due To Neutrons as a Function of Concrete Temperature

Temperature	**Dose Rate Adjacent to Cask Mid-Height (mrem/hour)**
Unheated (as cured)	1.88
100°C	2.94
200°C	3.28
300°C	3.47
All Water Evaporates	4.16

However, the above assumes that thermal neutrons will be attenuated once they are reduced in energy. According to the Kaplan Text, "the concept of an effective removal cross-section is dependent on the presence of hydrogen." (70) If there are insufficient hydrogen atoms to thermalize and consequently contribute to the absorption of a neutron after it has been slowed, then the equations used above may underpredict the amount of radiation emanating from a hydrogen-free shielding material. Usually, concrete contains sufficient hydrogen and is dominated by the collision reactions. When hydrogen is not present, the thermal interactions may dominate from a shielding perspective. If this is the case, the neutron dose rate computed above is likely to be somewhat higher.

For example, the Kaplan text provides values of the thermal diffusion length of a certain type of concrete at two different temperatures: unheated (as cured) and at 100°C). Assuming a shield thickness equal to 67.95 cm, the following I/I_o values for thermal neutrons are presented:

Table 5: Thermal Diffusion Length as a function of Concrete Temperature

Concrete	Temperature	Density (g/cm^3)	L (cm)	I/I_o
0-HW1	Unheated (as cured)	2.33	6.98	5.92×10^{-5}
0-HW2	100°C	2.26	8.97	5.12×10^{-4}

The density of this concrete is very similar to that of the HI-STORM overpack concrete. For our purposes, we assume they are identical in terms of shielding. If we assume a similar loss in hydrogen content (from .015 to .007 g/cm^3 concrete) as a result of heating to 100°C that was witnessed in the "ordinary concrete" discussed in the Kaplan text, we note that a decrease in hydrogen content by approximately 50% leads to a decrease in thermal neutron shielding by an order of magnitude. If we assume a linear relationship between thermal diffusion length and hydrogen content, we can make the following estimates of loss of thermal shielding as a function of temperature (and consequently, hydrogen content).

Table 6: Estimated Thermal Diffusion Length as a Function of Hydrogen Content

Temperature	Density (g/cm^3)	H content (assumed)	L (cm)	I/I_o
Unheated (as cured)	2.33	.015	6.98	$5.92x10^{-5}$
100°C	2.26	.007	8.97	$5.12x10^{-4}$
200°C	2.238	.005	9.66	$8.80x10^{-4}$
300°C	2.227	.004	10.04	$1.15x10^{-3}$
No Hydrogen Left	2.194	0	11.95	$3.39x10^{-3}$

Thus, if the concrete in a HI-STORM 100 cask were to lose all of its water, it is estimated that the amount of thermal neutron radiation passing through would be approximately 57.3 times greater than that calculated in the HI-STORM SAR. In terms of radiation dose, assuming proportionality, this would increase the neutron dose to workers to **1.88 mrem/hour x 57.3, or 108 mrem/hour.**

EXHIBIT 4

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

OFFICE OF THE SECRETARY

November 19, 2001

MEMORANDUM TO: William D. Travers
Executive Director for Operations

FROM: Annette L. Vietti-Cook, Secretary /Annette Vietti-Cook/

SUBJECT: STAFF REQUIREMENTS - SECY-01-0178 - MODIFIED RULEMAKING PLAN: 10 CFR PART 72 -- "GEOLOGICAL AND SEISMOLOGICAL CHARACTERISTICS FOR SITING AND DESIGN OF DRY CASK INDEPENDENT SPENT FUEL STORAGE INSTALLATIONS"

This is to advise you that the Commission has not objected to the staff's plan to revise the approved rulemaking plan for the geological and seismological characteristics for the siting and design of dry cask independent spent fuel storage installations (10 CFR Part 72), subject to the comments provided below.

Central to this rulemaking is the determination of the mean annual exceedance probability of an earthquake at a proposed ISFSI. The proposed rule should solicit comment on a range of probability of exceedance levels from 5.0E-04 through 1.0E-04. Staff should undertake further analysis to support a specific proposal.

The proposed rule should be submitted to the Commission for review prior to publication.
(EDO) (SECY Suspense: 3/22/02)

cc: Chairman Meserve
Commissioner Dicus
Commissioner Diaz
Commissioner McGaffigan
Commissioner Merrifield
OGC
CFO
OCA
OIG
OPA
Office Directors, Regions, ACRS, ACNW, ASLBP (via E-Mail)
PDR

EXHIBIT 5

EXHIBIT 5

Comparison between INEEL ISFSI and PFS proposed ISFSI

Item	INEEL	PFS
Type of Fuel	TMI-2 rubblized core debris[1]	40,000 MWD/MTU (FSER[2] at 7-6)
Type of Cask	NUHOMS (modified) Array of horizontal concrete vault modules[1]	HI-STORM 100 Upright unanchored cylindrical cask (SAR[3] at 4.2-3)
Total Number of Casks	29[1]	4,000 (FSER at 7-4)
Total quantity of stored material	307,000 pounds of rubblized reactor core debris[1]	40,000 metric tons of uranium (FSER at 7-4)
DSHA pga	0.56g (SECY-98-071[4] at 2)	1.15g (horiz); 1.17g (vertical) (Geomatrix 2001[5] at 3)
PSHA pga (10,000 yrs)	0.47g (SECY-98-071 at 2)	~1.2g (horiz); ~1.3g (vertical) (Geomatrix 2001 Fig 6-11, -21)
PSHA pga (2,000 yrs)	0.30g (SECY-98-071 at 2)	0.711g (horiz); 0.695 g (vertical) (SAR at 2.6-107)
DBE pga Return Period for DBE pga	0.36g (SECY-98-071 at 2) ~4,000 years	0.711g (horiz); 0.695 g (vert) 2,000 years (SAR at 2.6-107)
Design Lifetime	<35 years (DOE[6])	40 years (SAR at 1.1-2)

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[1] *See* Attachment A, February 21, 1997 memorandum from M.G. Raddatz, Senior Project Manager, Spent Fuel Project Office, to C.J. Haughney, Acting Director, Summary of Feb. 6, 1997 public meeting to discuss licensing for INEEL ISFSI, and accompanying DOE slides.

[2] NRC Staff's Final Safety Evaluation Report, dated September 2000.

[3] PFS Safety Analysis Report.

[4] *See* Attachment B, SECY-98-071, Exemption Request to 10 CFR 72.102(f)(1) Seismic design requirement for Three Mile Island Unit 2 ISFSI (April 8, 1998).

[5] Geomatrix, *Update of Deterministic Ground Motion Assessment*, Rev. 1, April 2001.

[6] *See* Attachment C, DOE Office of Nuclear Material and Spent Fuel, Spent Nuclear Fuel Program website.

EXHIBIT 6

CONDENSED TRANSCRIPT

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	) Docket No. 72-22
	) ASLPB No. 97-732-02-ISFSI
PRIVATE FUEL STORAGE	)
L.L.C.	) TELEPHONE DEPOSITION OF:
	)
(Private Fuel Storage	) **BRUCE EBBESON**
Facility)	)
	) (Utah Contention L, Part B)
	)

Thursday, November 15, 2001 - 11:15 a.m.

Location: Office of the Attorney General
160 East 300 South, 5th Floor
Salt Lake City, Utah

Reporter: Vicky McDaniel
Notary Public in and for the State of Utah

CitiCourt, LLC
THE REPORTING GROUP
50 South Main, Suite 920
Salt Lake City, Utah 84144
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is the building itself, the crane and the seismic support struts?
A. Right. There may be more, but those are the ones I'm aware of that I had responsibility for.
Q. Okay, thank you. Do you know what the ground motions are for a 2,000-year return period earthquake at PFS?
A. Yes.
Q. And what are they?
A. Both vertically and horizontal peak accelerations of approximately .7 g's, and I don't know the exact number.
Q. Do you know what they are for a 10,000-year return period?
A. Yes. I believe the horizontals are approximately 1.2 something g, and the vertical is 1.3 something, in that order of magnitude. And that was based on a Geomatrix analysis which I saw the results of but not the calculation.
Q. Turning to your declaration on paragraph 9, can you explain how the HI-TRAC can remain attached to the crane throughout transfer operations? Please take time to review any of these paragraphs that I ask questions about.
MR. GAUKLER: Which paragraph is that?

MS. CHANCELLOR: Paragraph 9.
A. Could you repeat the question, please?
Q. Yes. Can you describe how the HI-TRAC can remain attached to the crane throughout transfer operations?
MR. GAUKLER: If you know.
MS. CHANCELLOR: It's in his declaration.
A. Yes. This is information extracted from the FSAR or the SAR by Mr. Cooper, and I accepted it as truth from the SAR.
Q. So you relied on Mr. Cooper for the statement in parentheses in paragraph 9 of your declaration; is that correct?
A. Yes.
Q. Who in turn relied on the SAR; is that correct?
A. Pardon? Yes.
Q. And do you know where in the SAR?
A. No, but I'm sure I could find it.
Q. That's okay.
A. I have reviewed that SAR section.
MS. CHANCELLOR: Just a moment, please.
Q. (BY MS. CHANCELLOR) In paragraph 10 of the declaration, near the end of the last line you refer to SAR Table 5.1-1. Do you see that?

A. Yes.
Q. Would you open the SAR at that table?
A. Yes, I have it.
Q. Okay. And is this revision 6?
A. Yes.
Q. In paragraph 10 of your declaration you state that the estimated time to complete a single transfer operation is approximately 20 hours. Is that correct?
A. Yes.
Q. Okay. And are you relying on Table 5.1-1 revision 6 of the PFS SAR?
A. Yes, which gives a total duration of 19.9.
Q. Okay. I would like to go through table 5.1-1, understand where these numbers come from. Table 5.1-1 has three columns, correct? It's entitled Anticipated Time and Personnel Requirements for HI-STORM Canister Transfer Operations. It's a table with three columns. The first column has the caption "Operation," the second column, "Number of Personnel," and the third column, "Task Duration (Hours)".
A. Correct.
Q. Item No. 1, receive and inspect shipment, measure dose rate takes three personnel a duration of 0.5 hours. Where does -- how did you determine the

number of personnel and the time?
A. That was done by someone else. That's not my area of expertise.
Q. Do you know who the someone else was?
A. No.
Q. Item No. 2, move shipment into canister transfer building, four personnel, 0.5 hours. Do you know the number of personnel or the number of hours?
A. That's the same answer.
Q. Item No. 3, remove personnel barrier, measure cask dose rates and perform contamination survey, three people, 1.6 hours.
A. Correct.
Q. Do you know where those numbers come from?
A. No, I do not.
Q. Remove impact limiters and tiedowns, three personnel, 1.5 hours.
A. No.
Q. No. 5, attach lifting yoke to crane and HI-STAR shipping cask, etc. Three personnel, one hour. Do you know where these numbers come from?
A. No, I don't.
Q. Do you know where any of the numbers come from on Table 5.1-1?
A. No, I do not.

Q. Do you know whether any HI-STORM 100 casks have been loaded with fuel?
A. No, I don't.
Q. Do you know how many personnel will be employed at PFS?
A. No, I don't.
Q. Do you know whether 20 hours is a reasonable time period for the transfer operations in the CTB of a single canister?
A. I don't know that specifically, but sounds like a reasonable number.
Q. What's the basis for your assumption?
A. Well, understanding the -- reading the table and understanding the steps, being familiar with the configuration of the building.
Q. What experience have you had in the transfer of dry cask transportation -- fuel from the dry -- from a dry cask transportation cask into a dry cask storage cask?
A. None.
Q. In paragraph 11 you state that in order to achieve the ultimate capacity of 4,000 casks over a 20-year period loading cycle, the PFSF would receive on average approximately 200 spent fuel casks per year or four casks per week. What is the basis of that

statement?
A. The basis is assuming that the 4,000 casks that have been received would be received in a uniform manner, continuously at the same rate. 4,000 divided by 20 is 200.
Q. Do you have any idea of whether PFS will reach its ultimate capacity?
A. No.
Q. Do you have any idea of whether PFS will actually receive four casks per week?
MR. GAUKLER: Can you speak up, please, Denise? Hard to hear you.
MS. CHANCELLOR: Sorry, Paul. I'm about to lose my voice.
Q. (BY MS. CHANCELLOR) Do you know whether PFS will actually receive four casks per week?
A. No.
Q. Do you know whether transfer operations would actually occur 4,000 hours during the year?
A. No.
Q. If you would turn to the section of your declaration on page 8, "potential building collapse."
A. Yes.
Q. And in paragraph 17 you refer to free-field ground motions from a 10,000-year return period

earthquake, and then in 19 you refer to the additional reserve capacity in the CTB from -- let me rephrase the second part of that question. In paragraph 19 you refer to the CTB being able to withstand forces resulting from a 2,000-year return period earthquake because of additional reserve capacity. I'd like to ask you some questions about your knowledge of seismicity in soils at the CTB to try and understand your --
MR. GAUKLER: Please say it again, Denise.
MS. CHANCELLOR: Sorry, Paul.
Q. (BY MS. CHANCELLOR) I'd like to ask you some questions about your knowledge of seismicity in soils at the PFS site. Do you know whether a major fault dips under the PFS site?
A. Yes.
Q. Would you agree that near fault sites like PFS where a major fault dips under the site, earthquake waves are likely to arrive at an angle?
MR. GAUKLER: Objection. You haven't established any basis that that's his area of expertise.
MS. CHANCELLOR: I'm trying to understand why he is claiming that there is additional reserve capacity beyond a 2,000-year return period earthquake

in the CTB and why he thinks the free-field ground motions due to a 10,000-year return period earthquake will or won't have certain effects on the CTB. So I'm trying to understand what his level of knowledge is about the entire CTB design and structure. So if he's making these claims, I feel that I have a right to get into the entire design of the CTB. And he either knows the answer or it's not his area of expertise. And so that's what I would like to explore.
MR. GAUKLER: He's already identified what his inputs were for his analysis.
MS. CHANCELLOR: Well, I'd like some more specifics. I think it's appropriate to ask him -- he talked about seismicity. I want to know whether he has any opinion about whether earthquake waves will arrive at an angle. If he doesn't, that's fine.
MR. GAUKLER: To the extent you can answer the question, you can answer the question.
THE WITNESS: That is not my area of expertise. I have read reports that were provided by Geomatrix, our consultants, and I know that that's what -- they have considered that, but I don't know that as part of my area of expertise.
Q. In the design of the CTB did you consider any additional rocking and torsional motion on the