Canister Transfer Building

241. The Board starts from the proposition that PFS cannot meet a factor of safety of at least 1.1 without the buttressing effect of soil cement around the foundation perimeter of the CTB basemat, yet not until some distant future date will PFS acquire any data that it may arguably rely upon to support its use of soil cement. Soil Cement *supra*; Bartlett/Ostadan Tstmy, Post Tr. 7268 at 4-6. The design calculations for the sliding stability of the CTB under a 2,000-year design basis earthquake are found at PFS Exh. UU; there are no sliding calculations for a 10,000-year mean return period earthquake. Tr. (Trudeau) at 6348. As the State points out there are no engineering calculations or performance data to support the presumed passive resistance PFS expects to obtain from using such a mass of soil cement around the perimeter of the CTB mat foundation for the 2,000-year design basis earthquake. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 21; *see also* Tr. (Trudeau) at 6264-67. Nor is there any analysis of the effects of separation and cracking caused by out of phase motion of the CTB mat foundation and the soil cement buttress; or how bending and tensile stresses that develop in the soil cement will resist seismic forces without cracking or separation. Tr. (Trudeau) at 6257; (Ebbeson) at 6399; Bartlett/Ostadan Tstmy, Post Tr. 7268 at 21. The base mat of the CTB is expected to settle three inches; the effects of this settlement of the integrity of soil cement and its separation from the CTB on the passive resistance have not been considered by PFS. Tr. (Trudeau) at 6261.

242. Mr. Trudeau testified that he considered soil structure interaction in the CTB dynamic analysis calculation, PFS Exh. VV "inasmuch as the loads came from out structural dynamics people." Tr. (Trudeau) at 6191. However, there has been no dynamic analysis of

the interaction of the soil cement with the CTB mat foundation for the 2,000-year design basis earthquake. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 21. Under the design basis earthquake, the maximum horizontal acceleration response of the CTB mat is 1.047 g. Tr. (Trudeau) at 6192; PFS Exh. VV at 49. The free field peak horizontal ground acceleration response of the adjacent soil cement buttress is 0.71g. Tr. (Trudeau) at 6264. Consequently there is a 47 percent difference between the horizontal response of the CTB and the surrounding soil cement. Id. The soil cement buttress is not structurally tied to the CTB mat foundation, and given the large differences in horizontal acceleration response between those two masses, there is a significant potential for out-of-phase motion resulting from this inertial interaction. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 21; Tr. (Trudeau) at 6265. PFS has not considered the reduction of foundation damping and the concomitant higher seismic loads or the kinematic motion of the CTB caused by the blanket of soil cement around the CTB foundation. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 21. As described in the sections above on Soil Structure Interaction and Pad-to-Pad Interaction, these dynamic interactions can have a significant effect on reducing radiation damping and overestimating seismic loading. The additional concern here is that soil cement will not provide passive resistance. All of these factors affect PFS's design calculation in meeting a factor of safety of at least 1.1. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 21.

243. Similar to the concerns the State raised on the rigidity or flexibility of the storage pad, the State, based on Dr. Ostadan's past experience in dealing with the analysis and design of large mats such as the CTB foundation mat, questions whether the Applicant has appropriately treated the CTB mat as rigid. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 21;

Ebbeson Rebuttal Tstmy, Post Tr. 10790 at 2. Mr. Ebbeson testified that any "potential effect of mat flexibility is accommodated by the factor of safety applied in the seismic stability calculations." Ebbeson Tstmy, Post Tr. 6357 at 14; Tr. (Ebbeson) at 6427. The factor of safety against sliding in Cal. No. G(B) 13, however, has one case at a minimum of 1.15 and another, which PFS claims to be conservative analysis, of 1.26. PFS Exh. VV at 43. Furthermore, there are no design calculations to support the Applicant's assumption that the foundation mat is rigid. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 21. As described in detail on the question of pad rigidity, *supra*, if the mat is not rigid, soil damping used in the dynamic analysis will be excessive and seismic loads underestimated.

Board Finding

244. As use of soil cement is contributing to PFS's demonstration of meeting a factor or safety of 1.1 against sliding, it is essential that there are analyses, data and engineering calculations to support the claimed resistance to sliding, including whether the soil cement will, in fact, perform as intended during an earthquake. PFS has made no such showing.

245. The Board finds that the because the Applicant has not considered the effect of the large mass of soil cement on foundation damping and kinematic motion of the CTB, there may be an underestimation of seismic loads for the CTB, thereby invalidating PFS's dynamic CTB analysis.

246. The Board finds that, given the slim margins in PFS's dynamic analysis, any potential where PFS may have overestimated the effects of radiation damping or underestimated seismic loads is cause for concern. Accordingly, the Board finds that there is

insufficient evidence to show that the Applicant has validly assumed the CTB mat foundation is rigid.

E. Conclusions of Law

247. Based on the evidence presented, PFS has not met it burden of showing that the storage pads, the CTB, their foundations systems, and the storage casks have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake. The Board concludes that PFS has not met the requirements of 10 CFR §§ 72.90, 72.102(c) and (d), 72.120(a) or 72.72.122(b).

CONTENTION PART D: Cask Stability

A. Issue: Has PFS met its burden of showing that the free standing HI-STORM 100 casks experience excessive sliding, uplift, collision, or tip over under design basis ground motions at the PFS site?

B. Regulations/Guidance:

See "Seismic Design and Foundation Stability" above.

C. Findings of Fact - Cask Stabilitiy

248. State expert, Dr. Farhang Ostadan explains that "typically for design," a designer knows the design parameters and specification based on experience. Tr. (Ostadan) at 7311-12. If the analyses is not "right," the designer still has confidence in the design based on past experience. Id. Seismic engineers often rely on design redundancies, such as anchoring the cask, because of the uncertainties in input parameters. Id. at 7340, 7342. A key issue in this case, opined by Dr. Ostadan, is that PFS relies solely on the accuracy of the nonlinear predictions of cask response because of the lack of past experience with PFS's

unique and unconservative design, the lack of redundancy in PFS's design, and the lack of test data to validate the nonlinear seismic analyses of the freestanding cask. Id. at 7312, 7335-36, 7340-41. In other words, Dr. Ostadan is concerned that if Holtec's nonlinear seismic analysis of freestanding cask is wrong, the cask may react to ground motions differently than predicted. Id. at 7342.

249. Dr. Ostadan notes that seismic engineers "know a great deal" about how "conventional" designs perform during earthquakes. Tr. (Ostadan) at 7342-43. In this case, Dr. Ostadan further opines that because of the "lack of appropriate test data and experience data, you may wonder now how credible the results are." Id. at 7343. Based on Dr. Ostadan's experience, he was unaware of any nuclear facility where the designers knew that the facility would be located over a major active fault, such as in this case. Id. Furthermore, Dr. Ostadan knew of no nuclear facility with shallowly embedded foundations that estimated three inches of settlement during the design phase as in this case. Id. at 7351. When considering the unconventional nuclear facility design, the lack of experience and test data, the slim design margins, and the complexity of the nonlinear analyses, Dr. Ostadan emphasized "I would not, if I was the one, solely rely on a nonlinear program for my project. I would be most vulnerable if I do that." Id. at 7353.

250. At the time of the May 2002 hearings, NRC had licensed 23 ISFSIs – 11 site specific licenses and 12 general licenses. Tr. (Guttmann) at 7045. In May, the 23 ISFSIs stored approximately 325 dry storage casks. Id. Of the 23 ISFSI sites, Mr. Guttmann was unaware of the number of sites where the ground motions exceeded or equaled 0.7 g, where the ISFSIs were supported by a soil cement layer. Id. at 7070-71. Mr. Guttmann had no

knowledge of the number of free standing cylindrical casks or HI-STORM 100 casks in storage. Id. at 7069.

251. Only two sites, the Hatch (Georgia) and Dresden (Illinois) reactors, are currently storing an estimated 12 HI-STORM 100 casks[35]. Tr. (Singh) at 5918. The ground motions at Hatch and Dresden are 0.15 g (vertical) and 0.1 g (horizontal) (Tr. (Luk) at 6914-15) and 0.2 g zero period acceleration (State Exh. 121 at 38), respectively. Thus, the Licensing Board finds no site currently storing HI-STORM 100 casks that has estimated ground motions equal to or exceeding the ground motions estimated at the PFS site for either a 2,000-year or 10,000-year earthquake. Furthermore, no evidence was proffered that HI-STORM casks are currently stored on foundations supported by cement-treated soil and relatively soft clay. *See* Tr. (Singh) at 5989.

252. The Licensing Board finds insufficient evidence that the Staff has licensed free standing, cylindrical dry casks, similar to the HI-STORM 100 cask, at sites where the ground motions equaled or exceeded those for the 10,000-year earthquake at the PFS site. Additionally, the Licensing Board also finds insufficient evidence that the Staff has licensed free standing, cylindrical dry casks at sites where the design basis ground motion equaled or exceeded 0.7 g, the proposed design basis ground motion at the PFS site. The Licensing Board finds no evidence that any free standing dry storage casks are stored at sites similar to the PFS site, on shallowly embedded foundations, supported by a cement-treated soil layer

[35]HI-STORM 100 S casks are in storage at the J.A. Fitzpatrick reactor (New York). Tr. (Singh) at 5915. Dr. Singh also anticipated HI-STORM 100 S casks to be used at the Columbia Generating Facility (Washington) and additional HI-STORM 100 casks at the Hatch reactor and Dresden in 2002. Id.

and relatively soft clay foundation, subject to ground motions equal or exceeding 0.7 g.

253. Because PFS has an unconventional design that is unprecedented and unproven with no redundancies, the State claims that comprehensive analysis and testing are necessary to determine whether the HI-STORM 100 cask will excessively slide, uplift, or tip over under the 2,000-year DBE. Khan Tstmy., Post Tr. 7123 at 5-6. The State further claims that PFS has failed to conservatively account for the cumulative effects of potential ground motion on its design and thus, PFS's seismic analysis may significantly underestimate cask behavior. Id. Accordingly, in this section we consider the evidence presented with respect to the opinions and analyses of the seismic behavior of a HI-STORM 100 cask at the proposed PFS site.

Standard

254. Staff witness, Jack Guttmann testified that the Staff's technical licensing decision is based on "standard practices, and standard review plan, commission guidance and polices, and regulations." Tr. (Guttmann) at 6827. Mr. Guttmann further testified that the "regulatory posture" is that the cask does not tip over. Id. at 6977. Thus, until an applicant requests an analysis otherwise, whether the cask tips over is the appropriate standard. Id. The Staff further states "[t]he acceptance criterion was that the casks must be stable in the sense that the center of the top cover of the cask must remain within the original contact circle that the cask makes with the pad." CSER at 5-30. The Licensing Board finds that the issue in this section is whether the Applicant has reasonably demonstrated that the HI-STORM 100 cask will not tip over when subject to the proposed design basis earthquake - a 2,000-year earthquake at the PFS site.

255. To support the PFS license application, the cask vendor, Holtec International Inc. ("Holtec") evaluated the cask stability of its HI-STORM 100 storage cask subject to a 2,000-year DBE at the PFS site in its report entitled *Multicask Response of PFS ISFSI from 2,000-yr Seismic Event (Revision 2)*, Rev. 1 (August 2001) ("Holtec 2,000-year report") (proprietary document, State Exh. 173).[36], [37] Tr. (Gaukler) at 5941.

256. The evidence proffered on the seismic response of the cask is centered on various nonlinear computer analyses conducted on behalf of the three parties. State expert, Dr. Mohsin Khan warns that "nobody can [exactly] predict the nonlinear behavior." Tr. (Khan) at 9358. Moreover, because of the sensitivity in selection of input parameters, nonlinear analyses have sometimes been referred to as obtaining solutions from a "black box." Tr. (Ostadan) at 7335-36. PFS witness, Dr. Allin Cornell also confirmed Judge Farrar's concern that it is "possible to become too enamored of the [computer] models and lose sight of making sure [the models] are anchored in reality." Tr. (Cornell) at 8024. Similarly, PFS witness, Dr. Alan Soler testified, "you can't just say, because the computer program says it's so, that means it's so." Tr. (Soler) at 9775. Dr. Cornell emphasized that nonlinear analyses provide information and insight, but a critical question is "how much information to take from [nonlinear analysis] away towards making subsequent design judgments." Tr. (Cornell)

[36]Holtec International Inc. claimed the exhibit as proprietary information. Tr. (Gaukler) at 5945-46.

[37]This report was preceded by other Holtec cask stability analyses for the PFS site before the Applicant discovered it had severely underestimated the ground motions, which increased from 0.53g (horizontal) and 0.52g (vertical) to 0.711 g (horizontal) and 0.695 g (vertical). Bartlett/Ostadan Tstmy (Part D, dynamic analysis), Post Tr. 7268 at 4.

at 8010. Given this background, we approach a review of the nonlinear analyses with a certain degree of circumspection.

Expert Witness Conflict of Interest.

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

258. At the time of the hearing, Holtec had only 12 storage casks in use, all of which are HI-STORM 100 casks.[38] Tr. (Singh) at 5918. If the PFS facility attains fruition, Holtec, effectively Drs. Singh and Soler, have the potential to sell 4,000 storage casks and other products such as the HI-TRAC canister cask to the PFS project. Id. at 5910-11, 5920. Dr. Singh admitted that sales to PFS could reach the hundreds of millions of dollars by the "crudest estimate." Tr. (Singh) at 5910-11, 5920.

259. As recognized in NRC cases "most expert witnesses do receive compensation from the parties on whose behalf they testify. But their compensation is for their time and expertise, not for their testimony as such." Louisiana Power and Light Company (Waterford

---

[38]As noted earlier, HI-STORM 100S casks are stored at the J.A. Fitzpatrick reactor (New York). Tr. (Singh) at 5915. Additional HI-STORM 100 and 100S casks will be loaded in 2002. *See* footnote ___ *supra*. In addition to the HI-STORM 100 cask system, Holtec markets the HI-STORM 100S and HI-STORM 100SA. Id. at 5914-15. The HI-STORM 100S is a "hugely improved version" of the 100 "to deploy . . . in high seismic regions." Id., and at 5911.

Steam Electric Station, Unit 3), ALAB-732, 17 NRC 1076, 1091 (1983). Here, the financial rewards from the successful outcome of this proceeding in favor of the Applicant are substantial. If PFS is licensed, the financial benefits to Dr. Singh and Dr. Soler, as two of three sole owners of the privately owned company - Holtec, will pale in comparison to the usual expert witness compensation. The Licensing Board also finds that Dr. Singh and Dr. Soler have a substantial interest in both the licensing of the PFS facility and the affirmation by this Board of the Holtec analyses, including those conducted with the DYNAMO code, also owned, in part, by Dr. Singh and Dr. Soler. Based on the Licensing Board's decision concerning the propriety of Holtec's codes and methodologies, we note that the outcome in this case may have far reaching effects on Holtec's business.

260. Bias or interest in the outcome of this case "goes only to the persuasiveness or weight that should be accorded the expert's testimony." Waterford, 17 NRC at 1091 (*citing* 11 J. Moore & H. Bendix, Moore's Federal Practice ¶ 702.30[1] (2d ed. 1982)). The Board finds that Dr. Singh and Dr. Soler have a bias and interest in the outcome of this case. Accordingly, we find it apropos to consider those biases and interest in our deliberation of the weight to accord their testimony and other the evidence relevant thereto.

Holtec's Experience In Performing Non-linear Analysis of Free Standing Casks.

261. Consistent with the 1993 U.S. Supreme Court case Daubert v. Merrell[39] which establishes the standard for expert witness testimony, the Licensing Board finds that the weight given evidence with respect to the cask stability analyses is dependent upon, a) the

[39]Daubert V. Merrell Dow Pharmaceuticals, Inc., 509 US 579 (1993).

proffering witness' relevant education, training, and experience in performing nonlinear seismic analyses of free standing casks subject to site conditions similar to the proposed PFS (e.g., ground motions, soil conditions, etc.); b) whether the expert's testimony is based on sufficient facts; c) whether the testimony relating to cask stability is based on reliable principals and methods and; d) the witness has applied the principles and methods reliably to the facts of the case.[40]

262. Holtec testified that it performed site specific cask stability analyses for the PFS site and five other ISFSIs. Singh/Soler Tstmy, Post Tr. 5750 at 14. At three of the five ISFSIs sites, Holtec analyzed free standing casks: the Dresden site, where zero period acceleration is 0.2 g (State Exh. 121 at 38); the Entergy Northwest (Columbia Generating) site, where the zero period acceleration is about 0.5 g (State Exh. 120 at 18, 29); and the Tennessee Valley site, where the ground motion is approximately 0.5-0.6 g (State Exh. 121 at 38). At the fourth ISFSI, J.A. Fitzpatrick, there is no record evidence of the ground motions. Other than for the PFS site, the only other site where PFS has conducted a nonlinear analysis is at Diablo Canyon, a site with the ground motions were as high as the 2,000-year earthquake at PFS,[41] but this was on the HI-STORM 100SA, the anchored and "hugely improved" version of the HI-STORM 100 cask. Tr. (Soler) at 5930; Tr. (Singh) at 5911. The Board has

[40]"[A] witness qualified as an expert by knowledge, skill, experience, training, or education, may testify thereto in the form of an opinion or otherwise if (1) the testimony is based upon sufficient facts or data, (2) the testimony is the product of reliable principles and methods, and (3) the witness has applied the principles and methods reliably to the facts of the case." Fed. R. Evid. 702 (*emphasis added*); *see also* Daubert, 509 U.S. at 588.

[41]The analyses performed for the Diablo Canyon ISFSI considered a ground motion "around" 0.9 g. Tr. (Soler) at 5929-30, -32.

already noted evidence of the significant site differences between the Diablo Canyon ISFSI site and the PFS site. *See* Contention D, Dynamic Analysis, *supra*. Further the Board finds that Holtec's analysis of the anchored HI-STORM analysis is not comparable to an analysis of an unanchored cask. Thus, the Licensing Board finds that the record does not support that Dr. Soler and other Holtec analysts have any previous experience conducting nonlinear seismic analysis of free standing casks at ground motions that equal or exceed the 0.7 g ground motion for a 2,000-year earthquake at the PFS site.

263. Additionally, there are no known or anticipated sites that store or will store the unanchored casks supported by soil cement or cement-treated soil. Tr. (Singh) at 5989; *see also* (Guttmann) at 7070-71. The Licensing Board also finds that Dr. Soler and other Holtec analysts have no previous experience conducting nonlinear seismic analyses of free standing casks supported by cement-treated soil or soil cement foundations. Based on our finding that the proposed design of free standing, cylindrical casks supported by cement-treated soil and relatively soft clay foundation at 0.7 g peak ground motions is unprecedented, the Licensing Board further finds that no prior cask stability analyses, other than those conducted for the PFS site, provide direct, relevant experience in conducting the analysis for this case.

264. Dr. Singh claimed that Holtec has performed "thousands" of runs simulating freestanding structures; thus, a new model is "verified" against Holtec's data from past results. Tr. (Singh) at 9677. Holtec performed numerous seismic analyses of free standing spent fuel racks. Singh, Soler Tstmy, Post Tr. 5750 at 14. However, in the analysis of spent fuel racks, the racks are submerged in water and there are very small gaps between the racks; thus, Dr. Khan testified that the nonlinear stability analysis of a cask is "very different" from

a free standing spent fuel rack. Tr. (Khan) at 7143. The record lacks sufficient facts to conclude that the analyses of free standing spent fuel racks are relevant to the experience and training necessary to conduct nonlinear seismic analysis of objects potentially subject to large deformation and rotations at high ground motions.

265. In its analysis, Holtec modeled the effects of soil structure interaction through soil springs (linear and rotational) and dampers. Tr. (Soler) at 5993. Dr. Soler and Chuck Bullard, a Holtec employee, authored the various cask stability reports for the PFS site. Tr. (Soler) at 5992. Dr. Soler admitted that neither Mr. Bullard nor he had expertise in analyzing soil dynamics and foundation design (calculating the soil springs and dampers). Id. at 5996-57. Besides the analysis for Tennessee Valley Authority, the only soil dynamic work that Dr. Soler has performed is for this case. Id. at 5995.

266. The Licensing Board finds that Dr. Soler and Mr. Bullard have limited experience in soil dynamics and the foundation design, the calculation of soil springs, and the modeling of soil structure interaction effects in a nonlinear cask stability analysis.

267. The Licensing Board finds, a) Holtec has not performed seismic analyses of free standing casks at sites with ground motions equal to or greater than the 2,000-year earthquake at PFS; b) neither Dr. Soler nor any other identified Holtec analyst are experts in soil mechanics; c) Dr. Soler and another Holtec analyst have calculated the soil springs and dampers at only one other site; d) a lack of evidence that Holtec has prior experience analyzing seismic pad-to-pad interaction; and e) the lack of evidence of the relevance of prior free standing spent fuel rack seismic analysis to the analysis in this case. In sum, we find that Holtec and its witnesses have limited experience in performing nonlinear cask stability

analysis at sites similar to the proposed PFS facility. Holtec's limited experience will be considered in the context of the weight given on various issues.

Applicant's Cask Stability Analyses.

268. The Holtec 2,000-year report describes, in part, Holtec's analysis using its proprietary computer program – DYNAMO – for the nonlinear cask stability analyses. Holtec modeled the cask as a two-body system - the storage overpack and the multipurpose canister ("MPC"). Singh/Soler Tstmy, Post Tr. 5750 at 17-18. The overpack is modeled with 6 degrees of freedom and the MPC is modeled with an additional 5 degrees of freedom. Id. The storage pad was modeled as a rigid body. Id. at A.59. The interface between the cask(s) and the pad are addressed using values for vertical and horizontal contact stiffness and the coefficient of friction. Id. at 17-18. Holtec used soil springs and soil damping coefficients to estimate the effects of the underlying soil and foundation to the cask and pad movement.[42] Id. at A29, A.32.

269. For its 2,000-year report, Holtec used a single set of time histories for a 2,000-year earthquake at PFS for 5% damping. Tr. (Singh) at 9671; (Soler) at 9675; State's Exh. 173 at 4. The 2,000-year report describes nine simulations. Singh/Soler Tstmy, Post Tr. 5750 at A.34. The simulations varied the number of casks from two, four, to eight. Id. at A.34; State Exh. 173 at 7, 8. The simulations also varied the soil properties referenced as "lower range,"

[42]Holtec relied on the methodology specified in ASCE 4-86, *Seismic Analysis of Safety related Nuclear Structures and Commentary*, Tables 3300-1 and 2, and Figure 3300-3. Tr. (Soler) at 5897; Singh/Soler Tstmy, Post Tr. 5750 at 21-22.

"upper range," and "best estimate" soil properties.[43] Singh/Soler Tstmy, Post Tr. 5750 at A.34; State's Exh. 173 at 8. Holtec assumed 5 percent damping between the cask and the pad to simulate energy loss due to impact. Tr. (Soler) at 5879; *see* Singh/Soler Tstmy, Post Tr. 5750 at A.21. Holtec assumed the storage pad was rigid in all simulations. Tr. (Soler) at 5757.

270. In an attempt to thwart the State's criticisms of the Holtec 2,000-year report, the cask vendors performed sundry computer runs and animations not with DYNAMO but with a different computer code, VisualNastran 2001. Applicant Exh. 86 at 14, Tr. (Soler) at 9749. The analyses of eleven runs are described in *PFSF Beyond Design Basis Scoping Analysis* (April 19, 2002) (hereinafter referred to as "Holtec Beyond Design Basis report) (Applicant's Exh. 86).[44]

271. In the Holtec Beyond Design Basis report, the casks were modeled as a six degree-of-freedom, rigid single homogenous cylinder. Singh/Soler Tstmy, Post Tr. 5750 at A.117, 5757; Applicant's Exh. 86 at 15. Holtec also modeled the storage pad as a six degree-of-freedom, rigid body. Tr. (Soler) at 5757; Applicant Exh. 86 at 15. The soil foundation was modeled by six springs (three linear and three rotational) with dampers. Applicant's Exh. 86 at 16. Contact between the cask and the pad was modeled by compression only springs and

---

[43]Geomatrix provided to Holtec the upper range, lower range, and best estimate soil properties. Singh/Soler Tstmy, Post Tr. 5750 at A.32.

[44]Holtec offered testimony concerning the results for it's previous report HI-2012780, *Dynamic Response of Free-Standing HI-STORM 100 Excited by 10,000 Year Return Earthquake at PFS* (November 2001) which was not entered as evidence into the record. Tr. (Soler) at 6002-6003, *see also* Singh/Soler Tstmy, Post Tr. 5750 at A.39. The simulations in the HI-2012780 Holtec report did not include soil structure interaction effects. Tr. (Soler) at 6002.

damper springs. Id. at 11. The damping associated with these dampers was set at 40 percent. Tr. (Soler) at 6065. Holtec did not model the soil cement (or cement-treated soil) beneath the storage pad; however, Holtec claims the effects of soil cement are included in the "lower bound" soil parameters. Tr. (Soler) at 5776.

272. The Holtec Beyond Design Basis report, runs 1 and 2 analyzed a 2,000-year earthquake at the PFS site and runs 3 through 11 analyzed a 10,000-year earthquake at PFS. PFS Exh. 86c. The number of casks in the analyses varied between 1, 2, 4 or 8 casks. Id. Input values in Holtec's model include values for six soil dampers, six soil stiffnesses, coefficient of friction between the cask and pad, the masses and location of the pads, the contact stiffness between the cask and the pad. Tr. (Soler) at 5790. For the simulations (runs 2 through 10) where Holtec "tuned" the soil stiffness to a frequency of 5 hertz, Dr. Soler selected 5 hertz based on his understanding of "deposition testimony" where a State expert witness stated 5 hertz as a "frequency at which there was predominant earthquake energy being input into the motion" and where the State expert saw observable deflections in the pad in ICEC calculations. Id. at 6059.

Reliability and Uncertainty of Applicant's Cask Stability Analyses.

DYNAMO is a Small Deflection Code With Questionable Reliability at Sites With High Seismic Ground Motions.

273. Holtec modified a published "general lumped mass analysis" code to create the predecessor to DYNAMO, the code used to generate the result in the Holtec 2,000-year report. State Exh. 120 at 24-28. DYNAMO admittedly is a "small deformation code" that is not capable of processing "large" cask rotations. State Exh. 120 at 27.

274. Although not quantified, Dr. Soler opines that the maximum angle of rotation that DYNAMO is capable of accurately processing results is less than 15 degrees. State's Exh. 120 at 29-30, *see also* Tr. (Soler) at 5926. Dr. Soler's opinion is that "[a]s long as the deflection [ ] predict[ed] [by DYNAMO] are not too large," he is confident DYNAMO is generating accurate results. Tr. (Soler) at 9930-31. In discussing a small deflection code such as DYNAMO, Dr. Soler opined that "if you attempt to take a code that is written for small deflections and blindly just apply it and get a result that would indicate large deflections, either your program will blow up on you or it will just give you ridiculously large results that have no physical meaning, or it will simply give you wrong results that you may think there's physical meaning to it." State's Exh. 120 at 43, *see also* Tr. at 9490-92. The Board finds no evidence to support Dr. Soler's confidence in DYNAMO producing accurate results in this case.

275. Dr. Soler agreed that the amount of cask tipping or rotation increases as the level of ground motion increases (zero period acceleration) level. Tr. (Soler) at 6032. Except for the PFS site, DYNAMO has not been used to analyze the stability of a free standing cask where the ground motions are equal to or greater than those for a 2,000-year earthquake at the PFS site (0.7 g). Tr. (Singh) at 5936; Khan/Ostadan Tstmy, Post Tr. 7123 at A.11 (*citing* State Exh. 120 at 19, 20, 29). The Licensing Board finds no evidence that the rotational limits of DYNAMO are not exceeded when evaluating ground motions equal to or greater than 0.7g, the 2,000-year earthquake at PFS.

276. To the contrary, using the same input parameters, DYNAMO failed to predict cask tip over, when in a Holtec nonlinear seismic analysis of its HI-STAR 100 cask using

VisualNastran, Holtec determined the HI-STAR cask would in fact tip over at a zero period acceleration (ZPA) of 0.6 g. Tr. (Soler) at 9772-73, 9775, *see also* State Exh. 199. Although the HI-STAR cask has different features than the HI-STORM cask, both were analyzed as free standing casks. Additionally, the Holtec 2,000-year report (State Exh. 173) references the methodology described in the HI-STAR technical paper (State Exh. 199) that discusses DYNAMO's failings as compared to VisualNastran. Tr. at 9782. In fact, Dr. Soler testified the reason, in part, for the technical paper comparison was to be cognizant that "you can't just say, because the computer program says it's so, that means it's so." Tr. (Soler) at 9775.

277. Holtec holds its DYNAMO code as proprietary information which has not been provided to the Staff or the State. Tr. (Singh) at 5923. This Licensing Board and the other parties have had no opportunity to test the reliability and limits of the DYNAMO code due to the proprietary claim held by Holtec. We note that "a trier of fact would be derelict in the discharge of its responsibilities were it to rest significant findings on expressions of expert opinion not susceptible of being tested on examination of the witness." Virginia Electric and Power Co. (North Anna Nuclear Power Station, Units 1 and 2), ALAB-555, 10 NRC 23, 26 (1979).

278. Additionally, Dr. Soler admitted that the contact spring stiffness computations used in State Exh. 173 are not all included in that report but referred to earlier documents that "set forth" the theory. Tr. (Soler) 9780. These "earlier documents" are not in evidence and therefore their reliability and the reliability of the contact spring stiffness computations have not been tested.

279. The Licensing Board finds that the question of whether DYNAMO, as a

small deformation code, generated accurate results in the Holtec 2,000-year report is a "significant finding" in which the opportunity to test the witnesses on cross examination is limited, in part, by the unavailability of the DYNAMO code. There is also an incomplete computation of input parameters in the record. As a result, we will consider the opposing parties ability to test witnesses on cross examination as a factor in weighing the evidence of the reliability of DYNAMO.

280. To support its use, Dr. Singh stated that DYNAMO has "been used in over a thousand discrete structures, qualifying them." Thus, he concluded, DYNAMO is a "well tested program." Tr. (Singh) at 6099-6100. The parties offered no evidence with respect to the type of "discrete structures" qualified by DYNAMO and how those DYNAMO analyses are relevant to this case given the unique and unprecedented design posed by PFS.

281. The Staff cites and accepts the results obtained with DYNAMO in the Holtec 2,000-year report. CSER at 5-30. However, the Staff does not specifically refer to the code used to obtain the Holtec results. *See generally* id. Holtec claims that the Staff also reviewed and accepted DYNAMO performed at other spent fuel storage sites. Singh/Soler Tstmy, Post Tr. 5750 at 14. The Licensing Board finds no evidence in the record concerning, a) the basis of the Staff's acceptance of Holtec's use of the DYNAMO code to accurately predict the dynamic behavior of unanchored casks under high seismic ground motions at the PFS site or sites with similar design characteristics, b) whether the Staff independently validated the results obtained with DYNAMO, and c) whether the Staff's previous acceptance of DYNAMO results have any direct bearing in this case where the Applicant has proposed to place free standing dry storage casks on a shallowly embedded foundation supported by

cement-treated soil in a seismically active location. Notably, the Staff did not have access to the DYNAMO code for any purposes, including verifying the input parameters, the model, or results. Tr. (Singh) at 5923. As a result of the lack of supporting evidence to demonstrate the basis of the Staff's acceptance of results generated by DYNAMO, the Licensing Board finds Holtec's reference to the Staff's previous acceptance of DYNAMO unpersuasive in this case.

282. During his testimony, Dr. Singh had with him DYNAMO's training manual in which he testified the manual contained over a dozen cases in which DYNAMO simulated a "wide variety of problems [such as] harmonic resonance, bifurcation, [and] . . . dynamic responses of nonlinear structures." Tr. (Singh) at 9679. Dr. Singh implies that the DYNAMO training manual represents that DYNAMO has been validated for both fuel rack and cask stability analyses. Tr. (Singh) at 9678-80. Dr. Singh professes that DYNAMO has been validated for dynamic responses of nonlinear structures. This Licensing Board's interest with respect to DYNAMO rests solely in any verification of its capability to accurately analyze the nonlinear seismic response of a free standing cask at the PFS site. We find it significant that the Applicant failed to proffer supporting documentation from the DYNAMO training manual if, in fact, Holtec has documented the scope and relevance of Dr. Singh's claims in this matter. Thus, this Licensing Board finds that not a scintilla of evidence has been offered by the Applicant that DYNAMO has been validated by the training manual.

283. Additionally, Holtec testified that DYNAMO results for "problems that had no simple analytical solutions were also evaluated [with DYNAMO] and shown to give good agreement with numerical solutions using finite element codes such as ANSYS." Singh/Soler Tstmy, Post Tr. 5750 at 20. However, Holtec also stated that ANSYS was not reliable, and in

fact, Holtec found "in the case of a simulation of an earthquake on a freestanding structure, [ANSYS] was [giving] unstable, actually incorrect results." Tr. (Singh) at 6099. In light of Dr. Singh's testimony as to the credibility of ANSYS to accurately model the seismic behavior of freestanding structures – the issues at the heart of this case – the Licensing Board finds that the comparison between DYNAMO and ANSYS unreliable.

284. Holtec testified that DYNAMO produced results in "good agreement" with known solutions for a "series of classical problems." Singh/Soler Tstmy, Post Tr. 5750 at 20. According to Holtec, the classical problems demonstrated DYNAMO features such as compression only behavior, friction resistance, etc. Id. No evidence was offered that demonstrates the relevance of the classical problems to the unique issues under consideration here. Thus, the Licensing Board finds that the capability of DYNAMO to reach "good agreement" with known classical solutions, albeit essentially unidentified known classical solutions, is inadequate to demonstrate the reliability of DYNAMO to accurately predict the seismic behavior of free standing casks under the 2,000-year earthquake at the PFS site.

285. Holtec further testified that DYNAMO was created and used to perform seismic analyses of spent fuel racks and was used in a number of free standing spent fuel rack analyses. Singh/Soler Tstmy, Post Tr. 5750 at 14-15. However, in the analysis of spent fuel racks, the racks are submerged in water and there are very small gaps between the racks; thus, Dr. Khan testified that the nonlinear stability analysis of a cask is "very different" from a free standing spent fuel rack. Tr. (Khan) at 7143. Moreover, the Board finds no evidence that the analyses of free standing spent fuel racks were conducted at ground motions equal or greater than a 2,000-year earthquake at the PFS site. The Board also finds no evidence of any angles

of rotation of free standing spent fuel racks in the confines of a spent fuel pool in comparison to free standing casks. Absent details substantiating the relationship between nonlinear analyses for free standing dry casks and free standing spent fuel racks, the Licensing Board finds unconvincing Holtec's testimony that prior acceptance of the DYNAMO code in spent fuel rack analyses is relevant in this case.

286. The Licensing Board finds that PFS has not produced evidence to demonstrate, (a) the capability of DYNAMO to produce accurate nonlinear seismic analyses of free standing casks under PFS site conditions, or (b) to support the comparison of DYNAMO results with known classical solutions, the comparison of DYNAMO results with results obtained with ANSYS, and the Staff's acceptance of DYNAMO results for both free standing casks and spent fuel racks.

287. The results generated by DYNAMO have not been benchmarked against full or bench scale data. State's Exh. 121 at 93-95. In the absence of validating physical data, the Licensing Board finds that the evidence discussed previously does not establish that the unquantified rotational limitations of the small deflection code – DYNAMO – were not exceeded in the Holtec 2,000-year report. We now turn to the details of the various analyses proffered during the proceeding, including the reliability of the Holtec animations performed with VisualNastran.

Testability of Holtec VisualNastran Results

288. Many times throughout cross examination, Dr. Soler could not specify specific details or results of his various nonlinear analyses because he did not personally seek the requested results; he observed the data visually and did not record the results; he did not

know the inner workings of VisualNastran; or he needed additional time to locate the details. Tr. (Soler) at 5770-71, 5773-76, 5779, 5791-5803; 6021. For example, as documented in the transcript, untracked casks in Dr. Soler's animations appeared to move greater distances than the tracked cask. Id. at 5761. Dr. Soler did not have the ability to identify the actual deflection or angle of rotation of other casks. Id., e.g., at 5779. Thus, although VisualNastran is a publically available code, the ability of parties, in particular the State, and the Licensing Board to test the reliability of Dr. Soler's testimony based in the nonlinear analyses was severely restricted.

289. Additionally, Dr. Soler admitted that no document in evidence lists every input value for each of his simulations. Tr. (Soler) at 5791; *see also* Tr. at 5796. Furthermore, Dr. Singh admitted that the Holtec Beyond Design Basis report does not list "each numerical value." Tr. (Singh) at 5796. Dr. Soler could not provide the critical damping used in his analyses of case 11 and relied upon "whatever ASCE 486 would ask you to use for the soil properties given to us, that is what we used." Tr. (Soler) at 5788-89. Furthermore, Dr. Soler was unaware of "the equations for equilibrium of rigid bodies [which] is built into the [VisualNastran] code." Tr. (Soler) at 5968.

290. During the hearing, the Licensing Board noted and now finds that the Holtec Design Basis report is no more informative, reliable or with foundation than the animation itself without the underpinning data. Tr. at 5853-54. As we assured the State when we admitted Holtec's animations over its objections, we now address, in general, the reliability of the animations. Tr. at 10552-54. As demonstrated by the State, the results of cask behavior could not be quantitatively determined from the animations alone. Without supplemental

documentation or narration, the animations merely represent one analyst's simulation of cask behavior. Given that Dr. Soler supplemented the record with actual input values and results, we find that the animations are not the best evidence. We also find that the visual animations themselves are dangerously prejudicial in that a trier of fact or future tribunal could in fact rely on the animations and not adequately weigh the facts in this case.

291. The Licensing Board finds the reliability of Holtec opinions which rely of the results generated using VisualNastran is a "significant finding" in which the opportunity to test the witnesses on cross examination is limited, in part, to the inability of the parties to test the VisualNastran results through cross examination. Similar to our finding with respect to DYNAMO, we will also consider the opposing parties' ability to test witnesses during the cross examination as a factor in weighing the evidence of the reliability of VisualNastran results.

292. Dr. Singh testified that the Staff's grant of a certificate of compliance for the Holtec's HI-STAR 100 shipping cask was supported in part by analyses generated with VisualNastran. Tr. (Singh) at 6112-13. The Licensing Board finds no evidence in the record concerning the basis of the Staff's acceptance of Holtec's use of the VisualNastran code; whether the Staff independently validated the results obtained with VisualNastran; and whether the Staff's previous acceptance of VisualNastran results have any direct bearing in this case where the Applicant has proposed to place free standing dry storage casks on a shallowly embedded foundation supported by cement-treated soil in a seismically active location. The Licensing Board finds no evidence that VisualNastran has been independently validated with test data for the sliding and uplift of free standing casks in an area with high

seismic ground motions. Based on the lack of supporting evidence, the Licensing Board finds the Staff's previous acceptance of VisualNastran with respect to the HI-STAR 100 cask unpersuasive in this case.

Non-Linear Analysis Input Parameters.

293. PFS witness, Dr. Wen Tseng, agreed with Dr. Ostadan that nonlinear analyses are sensitive to variation of input parameters. Tr. (Ostadan) at 7335, Tr. (Tseng) at 5695. According to Dr. Ostadan, small changes in input parameters may induce substantial changes in the results. Tr. (Ostadan) at 7352. PFS witness, Dr. Cornell confirmed that nonlinear dynamic analysis can be sensitive to some input parameters. Tr. (Cornell) at 8009. In light of no contradicting testimony, the Licensing Board finds that nonlinear analyses are sensitive to some input parameters.

294. No witness disagreed that the results of a finite element model such as a cask stability analysis are dependent upon the quality of input data. Tr. (Khan) at 7157; (Singh) at 6030; (Luk) at 11508. The Licensing Board finds that the acceptance of nonlinear cask stability results is dependent upon a showing that the data input into the models are reasonably conservative, accurate, and comprehensive to account for all effects to free standing cask movement under seismic ground motions.

Khan Report

295. At the request of the State, Dr. Mohsin Khan conducted a parametric study by modeling aspects of the seismic reaction of the HI-STORM 100 cask to evaluate Holtec's seismic analysis of free standing casks at the PFS site. Khan/Ostadan Tstmy, Post Tr. 7123 at A.18. For his parametric study, Dr. Khan utilized a finite element structural analysis code,

SAP2000, to model a single HI-STORM 100 cask as beam elements in which the base of the cask is connected to the storage pad using nonlinear elements. Id. at A.21. Dr. Khan's methodology, analysis, results, and conclusions are described in *Analytical Study of HI-STORM 100 Cask System Under High Seismic Condition*, Technical Report No. 01141-TR-000, Revision 0 (December 2001) (State's Exh. 122) (hereinafter referred to as "Khan Parametric Study").

296. Dr. Khan performed case studies using three mathematical single cask models with varying degrees of complexity.[45] Khan Tstmy, Post Tr. 7123 at A.21. In the second and third case studies, Dr. Khan varied input parameters such as contact stiffness, the coefficient of friction, and the damping. Khan Tstmy, Post Tr. 7123 at A.21.

297. In the second case which discounted rocking effects, for a coefficient of friction of 0.8, the horizontal cask displacement varied from 42.74 inches to 0.057 inches with varying contact stiffness from 1 x 10^6 pounds per inch to 454 x 10^6 pounds per inch, respectively. *See* State's Exh. 122, Table 2 at 11. Similarly, in the three dimensional case, the horizontal and vertical displacement varied with the values of contact stiffness, coefficient of friction, and structural damping. Id., Table 3 at 13.

Contact Stiffness.

298. As a result of his study, Dr. Khan determined that nonlinear mathematical models are highly sensitive to the assumed contact stiffness between the cask and the storage pad. Id. at A.16. Dr. Khan explains that local contact stiffness is needed in a mathematical

[45]The first case study modeled horizontal sliding without any vertical excitation. Khan Tstmy, Post Tr. 7123 at A.21. The second case study modeled horizontal and vertical excitation absent rocking effects due to the cask height. Id. The third case study modeled a three-dimensional cask with vertical and horizontal beam elements. Id.

simulation before any sliding occurs. Id. at A.24. After sliding occurs, the horizontal displacement is a function of the intertial forces overcoming the coefficient of friction times the mass. Id. Thus, displacement of the cask from seismic ground motion should not be very sensitive to the contact stiffness values. Id.

299. Additionally, Dr. Khan maintains that in nonlinear analytical solutions, high contact stiffness values also absorb significant amounts of energy before sliding actually occurs by reducing instantaneous velocities for the next successive iteration in the nonlinear analysis. Id. As a result, high contact stiffness could underestimate vertical displacement of the cask. Id.

300. In its model, Holtec used contact stiffness to "define the stiffness of the vertical-only 'compression springs' at the interface of the cask and the pad." Singh, Soler Tstmy, Post Tr. 5750 at A.137. Holtec used a single vertical contact stiffness value for its simulations in the Holtec 2,000-year report. Id. at A.137, Tr. (Soler) at 6042. Notwithstanding the State's challenge to Holtec's contact stiffness value, Dr. Soler opined that "we got acceptable answers in the 2,000-year return earthquake, so there was no incentive for us there to lower the contact stiffness." Id. at 6043. In its simulations of the 10,000-year earthquake,[46] Holtec used a vertical contact stiffness of 18,864,480 lbs per inch.[47]

[46] *See PFSF Beyond Design Basis Scoping Analysis*, HI-2022854; and *Dynamic Response of Free-Standing HI-STORM 100 Excited by 10,000 Year Return Earthquake at PFS*, HI-2012780, (November 2001).

[47] Dr. Singh and Dr. Soler's prefiled testimony erroneously states the 10,000-year earthquake analyses were conducted with a contact stiffness of 40,130,000 pounds per inch; however, Dr. Soler informed the Licensing Board and the parties that the contact stiffness was in fact 18,864,480 pounds per inch. Tr. (Soler) at 9561-75.

Tr. (Soler) at 9575.

301. Prior to his parametric study, Dr. Khan had not had occasion to select a contact stiffness value for sliding or tipping. Tr. (Khan) at 7217. Similarly, neither Dr. Soler nor Dr. Singh have proffered evidence that they have prior experience selecting a contact stiffness value for a sliding or tipping analysis of a free standing cask where the ground motions equal to or exceed those for a 2,000-year earthquake at PFS. Tr. (Singh) at 6936. The Licensing Board finds that neither Dr. Khan nor the Holtec witnesses have proffered evidence that their recommended contact stiffness value has been validated or benchmarked by test data or other cask stability analysis with similar ground motions.

302. A vertical contact stiffness of 450 x 10^6 lbs per inch for unanchored casks is too high, opines Dr. Khan, because the contact stiffness makes the vertical frequency of the cask too rigid which underestimates the vertical displacement of the cask. Khan/Ostadan Tstmy, Post Tr. 7123 at A.28. Dr. Khan testified that "[o]nce [cask] sliding begins, the high [contact] stiffness values artificially treat the solution as linear [*e.g.*, as if the cask is anchored to the pad] without amplifying it in the upward direction and give non-unique or invalid results." Id. at A.28, A.31. A high contact stiffness corresponding to a high response spectra frequency will never amplify the cask motion. Tr. (Khan) at 7231. Holtec notes its contact stiffness corresponds to a frequency in the rigid range of 111 hertz. Tr. (Singh) at 9634-35.

303. In the absence of test data, it is Dr. Khan's opinion that to conservatively capture the dynamic behavior of the cask, including cask rotation or rocking, the appropriate contact stiffness for unanchored casks must correlate with a frequency that falls within the amplified range of the response spectra curve. Khan/Ostadan Tstmy, Post Tr. 7123 at A.31;

Tr. (Khan) at 9362, 9374, 9482. The rotational stiffness or rotational springs in the model will move the cask with a certain damping at an associated frequency. Id. at 9482. If the contact stiffness does not correlate with the frequency in the amplified region of the response spectra, then the mathematical code will treat the problem as linear as if the cask is anchored to the pad. Khan/Ostadan Tstmy, Post Tr. 7123 at A.31.

304. Paramount to Dr. Khan's opinion is that the Applicant has offered no test data to support its nonlinear cask stability results. If the real dynamic behavior of the structure is unknown, Dr. Khan is adamant that structural analysis design philosophy mandates that the structure's behavior is analyzed using the "peak of the spectra times the weight and other factors into consideration." Tr. (Khan) at 7236. Thus, for design purposes in the absence of test data, to estimate the dynamic response of the cask, a range of contact stiffness is selected that correlates with the rocking frequencies in the earthquake response spectra that give the maximum dynamic response. Id. at 7215, 7208.

305. Dr. Khan opined that contact stiffnesses in the range of 1×10^6 pounds per inch and 10×10^6 pounds per inch correspond to frequencies in the amplified spectral range of the response spectra. Khan Tstmy, Post Tr. 7123 at A.32.

306. The Licensing Board notes that Dr. Singh testified that "[w]henever a problem cannot be physically modeled [such as with shake table testing], the engineer's only recourse is to make it conservative." Tr. (Singh) at 9685. Notwithstanding the need to make the model conservative, Dr. Singh disagreed with Dr. Khan's philosophy that in the absence of test data, the seismic response of the cask must be evaluated at a range of contact stiffness values corresponding to the natural frequency of the amplified region of the response spectra.

Tr. (Singh) at 9617-18. Further, Dr. Singh testified that contact stiffness is not a parameter or a function of the earthquake, but that it is an "intrinsic property of the bodies that are subjected to the earthquake." Id. at 9618.

307. In a 1998 note by Max DeLong of Northern States Power regarding NRC Staff question 3-11 to Sierra Nuclear, the Staff states "[t]he response spectrum for the acceleration time history chosen for the nonlinear analysis or confirmatory testing must be enveloped by the response spectrum. . . . Furthermore the duration of the seismic event must be consistent with high acceleration levels. Large earthquakes that have high acceleration levels are associated with strong ground motion durations." Tr. (Khan) at 9792-93, State Exh. 197A.[48]

308. Fundamental to the conflicting testimony concerning the value of contact stiffness is Dr. Khan's steadfast opinion that absent test data to validate the results of a nonlinear analysis, typical design philosophy requires the designer to match the rocking frequencies in the amplified region of the response spectra. In contrast, without regard to the purpose of Dr. Khan's philosophy, Holtec adamantly professes that the dynamic contact stiffness value must render a "realistic" static deflection value.

309. Holtec testified that contact stiffness is the force applied at the interface points of contact between the cask and the storage pad that would be required to move either

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

the cask or pad a unit distance[49]: K = W/d; where K is the contact stiffness, W is the weight of the cask, and d is the deformation by the pad under the cask. Id. Dr. Khan agrees that this formula would calculate the static, not dynamic, contact stiffness. Tr. (Khan) at 7211, 7237. In a dynamic response, the physical behavior changes as the cask moves; as a result, the load deflection characteristic on the pad would also change so the stiffness could vary with respect to time. Khan/Ostadan Tstmy, Post Tr. 7123 at A.31; Tr. (Khan) at 7243. Dr. Soler agreed, in part, that there is no contact stiffness when there is separation between the cask and pad. Tr. (Soler) at 6053, 9645. As represented in Holtec's model, the contact stiffness value at each contact point changes with time if the contact point on the cask is not physically on the pad due to rocking or uplift. Id. at 9645-47. However, Dr. Soler maintains that "contact stiffness should not be a function of the input motion." Id. at 6050.

310. Contrary to the Holtec witnesses, Dr. Khan's opinion is that a simple deflection calculation, K = W/d, cannot be used to determine a single unique contact stiffness value for a dynamic analysis where the cask can potentially rock, uplift, and slide. Khan/Ostadan Tstmy, Post Tr. 7123 at A.27, 31; Tr. (Khan) at 7235. A range of possible contact stiffnesses should be evaluated and validated with test results. Khan/Ostadan Tstmy, Post Tr. 7123 at A.32; Tr. (Khan) at 7235.

311. Dr. Singh testified that contact stiffness does not change in any significant manner whether the event is dynamic or static. Tr. (Singh) at 9628. However, Dr. Soler admitted that there is no contact stiffness when there is separation between the cask and pad

---

[49]Singh/Soler Tstmy, Post Tr. 5750 at A.136.

during dynamic motion. Tr. (Soler) at 6053, 9645. Significantly, Holtec has not validated its contact stiffness results with actual test data, such as shake table data. Tr. (Soler) at 6054.

312. Absent test data, both Dr. Khan and Dr. Soler agree that there is no single correct contact stiffness value that is appropriate for the nonlinear analyses of the cask. Tr. (Soler) at 6049. The Licensing Board does not find a disagreement between the parties as to the definition of contact stiffness. However, the crux of the disagreement between Dr. Soler and Dr. Khan is whether Holtec's calculated stiffness can be used to accurately predict the sliding, uplift, and rocking of the cask under high seismic ground motions, and how to accurately incorporate design conservatism into a nonlinear analysis absent validating test data.

313. Holtec concluded "that Dr. Khan's work comes to erroneous conclusions because he has not achieved the correct, converged solution for many of his simulations, and has utilized unrealistic and unsupportable inputs for the simulations." Singh, Soler Tstmy, Post Tr. 5750 at A.124. Holtec claims that a contact stiffness of 1×10^6 pounds per inch is unrealistic because the simple deflection formula ($K = W/d$) results in a 0.36 inch static deflection of the pad. Tr. at 6043-44. Although disagreeing that a static deflection formula is applicable to a dynamic model, Dr. Khan agreed that a 0.36 inch static deflection is not realistic. Tr. (Khan) at 7219.

314. To support his opinion that static contact stiffness is not applicable in a dynamic analysis, Dr. Khan testified that once the program is initiated, the boundary conditions are set so the static deflection prior to seismic excitation is not included in the results. Tr. (Khan) at 7210-11. In a dynamic analysis, the vertical contact stiffness is used to

justify whether there is vertical amplification or rocking. Id. at 7212.

315. To support its position, Holtec points to an ANSYS training model which cautions that too high a value for contact stiffness "causes [computation] convergence difficulties," but "[m]inimum penetration gives best accuracy." Applicant Exh. SS at 3-3. ANSYS further recommends that "[d]etermining a good stiffness value usually requires some experimentation" and to start with a low contact stiffness value. Id. at 3-14. ANSYS suggests that as a check "if you can visually detect penetration . . . the penetration is probably excessive." Id. (emphasis added).

316. Dr. Soler agreed that ANSYS does not provide an example verification problem for either a real or artificial earthquake applied to either pure sliding, pure uplift or any combination of uplift and sliding, nor does ANSYS specify a contact stiffness value. Tr. (Soler) at 6051-52, 5900-01. Thus, it is unclear whether the ANSYS training manual can be properly applied to uplifting, rocking, and sliding simulations such as in this case. However, without determining the applicability of the training manual in this case, we find that the ANSYS training manual does not nullify Dr. Khan's contact stiffness value of 1×10^6 because: first, Dr. Soler testified that "no one" contact stiffness value is necessarily correct (Tr. (Soler) at 6049); second, Dr. Soler testified that whether there is visual penetration is subjective (id. at 6037-38); third, ANSYS also advises that finding a "good" stiffness value usually requires "some experimentation," which we find similar to evaluating a range of stiffness values such as in the Khan Parametric Study; and finally, ANSYS qualifies its advice that if visible penetration is detected then penetration is "probably" but not conclusively excessive.

317. Dr. Soler opined that a contact stiffness value that resulted in a deflection of ".0 something" or ".00 something" inch would be sufficient for the cask stability analysis. Id. at 6038. Dr. Soler did not offer a technical basis for his opinion that ".0 something" or ".00 something" is acceptable deflection in estimating contact stiffness. Dr. Khan also opined that a contact stiffness of 10×10^6 pounds per inch is within the recommended spectral range. Khan/Ostadan Tstmy, Post Tr. 7123 at A.32. While noting that the values have not been validated with test data and that the experts acknowledge more than one contact stiffness value is acceptable, the static deflection of a cask corresponding to a contact stiffness of 10×10^6 pounds per inch is equal to 0.036 inches, which is within what Dr. Soler deemed acceptable.

318. Holtec further postulated that in some of the Khan Parametric Study cases, Dr. Khan exceeded the rotational limits of SAP2000. Tr. (Soler) at 9603-04. SAP2000 is a small deflection structural code that is capable of predicting accurate results if the angles of rotation are not large. Tr. (Khan) at 7174-75, 7183. SAP2000 will "blow up" or stop working if the rotational capability is exceeded. Id. at 7187. In SAP2000, "all nonlinearity is restricted to Nllink elements." Tr. (Khan) at 9346. The SAP2000 Nllink element limitation occurs when a significant amount of rigid body rotation (e.g., cask rotation) is introduced into the analysis. Id. at 9353, 9355. Dr. Khan testified that SAP2000 has no sliding displacement limitations or vertical uplift limitations if the center of gravity remains with acceptable limits. Id. Dr. Khan did not evaluate whether SAP2000 had exceeded the rotational limits. Id. at 9360.

319. The parametric study runs did not exceed the rotational capabilities of

SAP2000, opined Dr. Khan, because the analyses ran to completion and did not blow up. Tr. (Khan) at 7187. Although admittedly not a user of SAP2000, Dr. Soler opined that SAP2000 could run to completion and not stop even if the program had "blown up." Tr. (Soler) at 9615-16, 9604.

320. Dr. Khan agreed that as the coefficient of friction increases, the cask would lift and increase the potential for tipping. Tr. (Khan) at 9513. However, Dr. Khan disagreed that the cask would only tip and not slide in a simulation where the coefficient of friction is 0.8, because once the cask starts rocking, the response cannot be predicted where a coefficient of friction of 0.7 may be more sensitive to tipping than 0.8 Id. at 9513

321. Dr. Soler "presumed" Dr. Khan properly represented the cask by beam elements in his model. Tr. (Soler) at 9914-15. Although he questioned the accuracy of Dr. Khan's parametric study results, Dr. Soler admitted he was not a user of SAP2000 and could not comment "to any degree to certainty" why SAP2000 generated "erroneous" results. Tr. (Soler) at 9604. Notwithstanding Holtec's claims, Dr. Khan checked the time histories of his runs to verify that the cask did experience large rotations. Tr. (Khan) at 7187. The Licensing Board finds insufficient evidence to conclude that any of the Khan Parametric Study runs "blew up" or gave inaccurate results based on exceeding the rotational limitations of the program.

322. Dr. Soler testified that Holtec ran an 8 cask simulation for a 2,000-year earthquake with a contact stiffness of approximately one eighth of 4×10^6 pounds per inch, which resulted in a maximum deflection of cask no 1 of about half an inch to an inch. Tr. (Soler) at 6050-51. Holtec did not offer supporting documentation for this simulation or the

values of all input parameters. Based on the lack of simulation details, the Licensing Board finds Holtec's reference to this simulation unreliable.

323. Holtec also ran nine simulations where it "tuned" the soil stiffness so that the mass of the cask(s) and pad resonate at 5 hertz.[50] Tr. (Soler) at 5767, *see* Applicant's Exh. 86c at 17 (Holtec Beyond Design Basis report). The damping in these simulations was 1 or 5 percent. Applicant's Exh. 86c. The coefficient of friction was 0.2, 0.8, or randomly applied in a range between 0.2 and 0.8. Id. Holtec conducted one of the nine runs at a 2,000-year earthquake and the remaining simulations at the 10,000-year earthquake. Id.

324. Dr. Khan agreed that Holtec's "tuning"the soil stiffness to 5 hertz is a comparable approach to evaluating the cask response at rocking frequencies. However, Dr. Khan emphasized that Holtec did not evaluate a higher or lower frequency than 5 hertz where the dynamic response of the cask may be higher. Tr. (Khan) at 9482. To illustrate his point, Dr. Khan referred to the response spectra for 1 percent damping which shows peak acceleration of 4 g at 4 hertz, whereas at 5 hertz the acceleration is 2.7 g and then at 6 hertz the acceleration goes back up to 3.6 g. Tr. (Khan) at 9499-00, State Exh. 195. Moreover, based on Dr. Luk's results, soil structure interaction will filter some of the frequencies but still significantly amplify the accelerations when compared to free field acclerations. Tr. (Khan) at 9511, 9539; (Luk) at 6934-36. Thus, the actual acceleration at the top of the pad will be higher than the free field accelerations shown in the response spectra curves generated by Dr. Khan. Tr. (Khan) at 9511.

---

[50]The simulations are described in the *Holtec Beyond Design Basis Scoping* report, Applicant's Exh. 86c.

325. As shown in State Exh. 195, accelerations at 5 hertz are exceeded at frequencies above and below 5 hertz. The Licensing Board finds, absent test data, the Holtec simulations with "tuned" soil stiffness at 5 hertz do not reasonably show that all potential cask rocking and uplift are encompassed in the analyses. The Licensing Board further finds that the additional simulations in the *Beyond Design Basis Scoping* report do not validate Holtec's contact stiffness of 464 x 10^6 pounds per inch in the Holtec 2,000-year report or 18.8 x 10^6 pounds per inch in Holtec 10,000-year analyses.

326. Additionally, Dr. Soler testified that he simulated Dr. Khan's "exact problem" for case 3, Study run 3 of the Khan report using VisualNastran.[51] Tr. (Soler) 9603, Applicant's Exh. 225 at 15. The sole purpose of Dr. Soler's simulation was to demonstrate that case 3, study run 3 of the Khan Parametric Study is erroneous. Tr. (Soler) at 9613.

327. Holtec simulated the seismic response of a single cask with a 1 x 10^6 pounds per inch contact stiffness at about 5.2 hertz applied at 8 contact points, and 1 percent damping for a 2,000-year earthquake. Tr. (Soler) at 9605, 9611, Applicant's Exh. 225 at 15. The simulation did not consider the effects of soil structure interaction. Applicant's Exh. 225 ("Holtec Additional Analyses"). To simulate the SAP2000 model, Dr. Soler embedded 8 spheres rigidly attached to the cask. Id. at 16. Dr. Soler admitted that he did not model his cask system exactly the same as Dr. Khan but used the same number of contact elements and locations and used a different representation of the cask. Tr. (Soler) 9750-51. Moreover, Dr. Soler admitted that "presumably" had he run the same model as Dr. Khan with SAP2000,

[51]The analyses are describe in Applicant's Exh. 225, HI-2022878, *Additional Cask Analyses for the PFSF* (June 7, 2002).

Dr. Soler would have obtained the same results as Dr. Khan. Id. at 9755-56, 9757.

328. Also described in the Holtec Additional Analyses, Dr. Soler ran another simulation of the seismic response of a cask with a 1×10^6 pounds per inch contact stiffness applied at 16 points and 1 percent critical damping for a 2,000-year earthquake. Tr. (Soler) at 9613-14.

329. We note that no evidence was offered to demonstrate that Dr. Soler correctly remodeled and simulated the Khan model. We further note that Dr. Soler did not compare his results using the same parameters with his VisualNastran model used in Holtec Design Basis report. Thus, the Licensing Board finds there is insufficient evidence that the Holtec replication of the Khan Parametric Study, Case 3, Study run 3 is accurate or reliable. Although criticizing Dr. Khan's results, Dr. Soler attempted to replicate only one out of twenty runs performed in Dr. Khan's parametric study, case 3, study run 3. Tr. (Soler) at 9758-61, 9929. Thus, we find the Holtec simulations are not relevant to the reliability of the Khan Parametric Study.

330. In its comparison of stiffness and damping values, Holtec created an 8 cask animation on a pad with a 40 million pound per inch contact stiffness, 40 percent damping, lower bound soil springs, for a 2,000-year earthquake. Tr. (Soler) at 9673, Applicant's Exh. 225. Another animation of 8 casks on a pad used a 5 million pound per inch contact stiffness, 40 percent critical damping, lower bound soil springs, for a 2,000-year earthquake. Id. As a result of these three animations, Dr. Soler concludes that by "varying damping or stiffness or both" the difference in results are in the order of inches not feet. Tr. (Soler) at 9676.

331. In State's Exh. 195, Dr. Khan plotted the free field response spectrum in the vertical and two horizontal directions for 1, 3, 5, and 40 percent damping for the 2,000-year return period earthquake at PFS. Tr. (Khan) at 9495, 9498, 9502. In each direction for an associated damping, Dr. Khan plotted the frequency from 0 to 34 hertz against the acceleration to generate response spectrums using the same single standard degree-of-freedom model. Id. at 9495, 9499.

332. The points generated by Dr. Khan along the response spectra curve for 5% damping showed essentially identical correlation when compared to the 5 percent damping response spectra curve prepared by Geomatrix on behalf of PFS. Id. at 9543-47; *see e.g.* State Exh. 196. In the absence of contradicting evidence and the correlation between Dr. Khan's and the Geomatrix response spectra for 5 percent damping, we find Dr. Khan's response specta for 1, 3, 5, and 40 percent damping (State's Exh. 195) reliable.

333. The 40 percent damping response spectra curves are relatively flat and all acceleration is below 1 g between 0 and 33 hertz. Dr. Khan opined that if the acceleration stays below 1g, then the cask will behave in a rigid manner without dynamic amplification. Tr. (Khan) at 9499-9500; State Exh. 195. Recognizing Dr. Khan's curves are free field response curves and do not account for increased accelerations due to soil structure interaction, the Licensing Board finds that at 40 percent damping the cask would not uplift and behave as if anchored to the pad.

334. Dr. Soler also testified the damping value changes because critical damping is a function of stiffness. Tr. (Soler) at 9674. The Licensing Board notes that Holtec did not proffer an animation where it simultaneously lowered both damping and stiffness; hence, Dr.

Soler's conclusion with respect to "both" has no basis. Additionally, in challenging Dr. Khan's results, Holtec did not run a simulation at a contact stiffness of 1 x 10^6 pounds per inch. Thus, the Licensing Board finds that the effects on the cask behavior as a result of the stiffness value used are also a function of the damping value used. Consequently, considering that associated damping is a function of stiffness and the cask behavior during a seismic event is nonlinear, we find that the additional Holtec animations varying either damping or contact stiffness are insufficient to show that the cask behavior is not sensitive at both lower damping and lower contact stiffness, than used in Holtec simulations (*e.g.*, 40 percent damping and 18.8 x 10^6 pounds per inch contact stiffness).

335. Furthermore, since the actual value for the coefficient of friction is unknown, Holtec attempted to bound its analysis by evaluating the two values of the coefficient of friction. *See* State Exh. 173. The Licensing Board notes that Dr. Khan's design philosophy is similar in that it conservatively evaluates a range in the absence of test data.

336. At this point, we reflect back to our concern early on, which was confirmed by Dr. Cornell, that we should not become "too enamored" with the computer models and make sure the models are anchored in reality. Tr. (Cornell) at 8024. In addition, Dr. Ostadan cautioned us that the safety during a seismic event rests solely in the accuracy of Holtec's nonlinear predictions of cask response because of the lack of past experience with PFS's unique and unconservative design, the lack of redundancy in PFS's design, and the lack of test data to validate the nonlinear seismic analyses of the freestanding cask. Tr. (Ostadan) at 7312, 7335-36, 7340-41. We also feel that it is appropriate to balance Dr. Singh's and Dr. Soler's exuberant confidence in their own analyses with substantial financial interest in our

acceptance of their analyses and their limited experience in performing nonlinear seismic analyses for free standing casks at ground motion equal to or greater 0.7 g, such as in this case.

337. The Licensing Board finds the evidence is severely wanting with respect to the key dispute between the parties – whether a static contact stiffness is appropriate in a nonlinear dynamic seismic analysis of free standing casks. Consequently, the Licensing Board finds that in the absence of test data, it accepts Dr. Khan's design philosophy that to account for potential rocking, uplift, and sliding of the cask, the contact stiffness values must correspond to the amplified region of the response spectra.

Holtec Used High Damping Values that Underestimate Cask Movement

338. In the Holtec 2,000-year report, Holtec incorporated an impact damping of 5 percent. Tr. (Soler) at 6095-96. However, for the 10,000-year simulations, Holtec used a 40 percent of critical damping value to represent the loss of energy at the pad and cask interface. Tr. (Soler) at 6065, 6097.

339. Dr. Singh testified that Holtec changed the damping from the 5 percent used in the 2,000-year earthquake DYNAMO runs to 40 percent with the VisualNastran runs because of the increase in ground motion. Tr. (Singh) at 9671. According to Dr. Singh, the impact damping increases with the increase in ground motion. Id. at 9670. The Licensing Board finds no evidence beyond Dr. Singh's single statement to support a finding that the impact damping increases with increase in ground motion.

340. Dr. Khan raised his concern that the dynamic response may be underestimated in a nonlinear horizontal sliding analysis where the assumption is that all the

energy is dissipated if energy is also absorbed by using a high damping value. Tr. (Khan) at 9393-99. Dr. Ostadan concurred that the damping has been overestimated which resulted in reducing seismic loads in the dynamic analyses. Tr. (Ostadan) at 10389.

341. Dr. Singh claims that impact damping would be higher than 40 percent because Holtec calculated greater than 50 percent impact damping for a metal cask on a "thick, very thick concrete foundation" from a simple simulation where a cask was dropped and the amount of rebound was measured. Tr. (Singh) at 6098. The Applicant did not proffer supporting calculations for the impact damping of the metal cask nor did the Applicant explain the details of its assumptions or its relevance to the impact damping for the HI-STORM 100. Notwithstanding Dr. Singh's testimony, later during the hearing, Dr. Soler, agreed that the impact damping of a cask "might not be 40 percent, but it is extremely unlikely that [it] would be as low as 1 percent." Tr. (Soler) at 9912.

342. Additionally, Dr. Singh later referred to NRC sponsored impact experiments where steel "billets" were dropped on a concrete pad in which he claimed "our version of the program" was correlated with the test data. Tr. (Singh) at 9660-61. Holtec used the LS-Dyna code for its tip over and drop analyses. Tr. (Soler) at 9761. However, Dr. Singh was unaware of the actual impact damping results in the LS-Dyna tip over and drop analysis. Id. at 9761-62. Again, neither the Applicant nor the Staff offers any supporting documentation concerning the impact tests, which Holtec program was correlated with NRC data, or how the NRC impact tests relate to damping of HI-STORM casks during a seismic event. As we have stated throughout this opinion, our findings must be supported by sufficient facts in the record. Accordingly, this Licensing Board finds insufficient evidence in the record to rely on

the "metal cask" test data showing greater than 50 percent damping or the NRC "billet" impact experiment.

343. Dr. Soler proffered an animation of three spheres dropped from a height of 18 inches where each sphere had either 1, 5, or 40 percent damping. Tr. (Soler) at 9662 , Applicant's Exh. 225 (Holtec Additional Analyses). The 40 percent damping sphere came to rest after two bounces, the 5 percent damping sphere stopped after approximately 14 bounces, and the 1 percent damping sphere stopped after more than 73 bounces. Id. at 9665.

344. In another animation, Dr. Soler replaced the sphere image with a cylinder representing a cask where he dropped the cylinders from a height of 18 inches and each cylinder had either 1, 5, or 40 percent damping. Id. at 9665-66. In this case the cylinders with 40 percent damping and the 1 percent damping bounced three times and more than 73 times, respectively, before stopping. Id. at 9666-67.

345. In a reactionary flurry during the hearing, Holtec generated a number of additional animations, including an 8 cask animation with a 40 million pound per inch contact stiffness, 5 percent damping, lower bound soil springs, for a 2,000-year earthquake. Tr. (Soler) at 9673, Applicant's Exh. 225. Another 8 cask animation used a 40 million pound per inch contact stiffness, 5 percent critical damping, lower bound soil springs, for a 2,000-year earthquake. Id.

346. The response spectra curves, generated by Dr. Khan in State's Exh. 195, demonstrate that the acceleration is sensitive to the frequency of the system. State Exh. 195, Tr. (Khan) at 9496, 9501. For the reasons cited earlier, we again find Dr. Khan's response spectra for 1, 3, 5, and 40 percent damping (State's Exh. 195) reliable. For the 1 percent

damping curve, and to a lesser extent the 3 and 5 percent damping curves, the response spectra curves show acceleration varies with respect to frequency. State's Exh. 195. Additionally, the response spectra curve for 40 percent damping shows relatively no amplification which means the analysis treats the cask as a rigid or anchored system. Tr. (Khan) at 9496.

347. The 40 percent damping response spectra curves are relatively flat and the acceleration is below 1 g from 0 to 33 hertz. Recognizing Dr. Khan's curves are free field response curves and do not account for increased accelerations due to soil structure interaction, the Licensing Board finds that at 40 percent damping the cask would not uplift and behave as if anchored to the pad.

348. At 1, 3, and 5 percent damping, the response spectra curves show higher acceleration in excess of 1g at various frequencies which will result in cask uplift. Id. at 9496-97, State Exh. 195. The response spectra curves at various damping also show that if the contact stiffness is tuned to 33 hertz, there is relatively no increase in acceleration on any curve. Tr. (Khan) at 9502, State Exh. 195.

349. Holtec testified that at a static contact stiffness of 454 million pounds per inch and assuming the cask were connected to the pad, the natural frequency of the cask is 111 hertz and thus, outside of the frequency of an earthquake. Tr. (Singh, Soler) at 9635-36. A contact stiffness of 454 million pounds per inch treats the cask as if it were connected to the pad. Id. The calculated natural frequency assuming the cask is connected to the pad is known as the "rigid range." Tr. (Singh) at 9636. Dr. Singh further professed that to reduce the stiffness to correspond with the amplified region of the response spectra, such as at 5

hertz and 1 percent damping, will give results that "bear no semblance to [ ] reality." Id. at 9637.

350. Dr. Soler also testified that the damping value changes because critical damping is a function of stiffness. Tr. (Soler) at 9674. The Licensing Board notes that Holtec did not proffer an animation where it simultaneously lowered both damping and stiffness; hence, Dr. Soler's conclusion with respect to "both" has no basis. In association with our earlier finding, the Licensing Board also finds that the effects on cask behavior as a result of the damping value used are also a function of the stiffness value used. Thus, we re-emphasize our earlier finding that considering that the associated damping is a function of stiffness and the cask behavior during a seismic event is nonlinear, we find that the additional Holtec animations varying either damping or contact stiffness are insufficient to show the cask behavior is not sensitive at both lower damping and lower contact stiffness, than used in Holtec simulations (e.g., 40 percent damping and 18.8 x 10^6 pounds per inch contact stiffness).

351. With respect to the Holtec dropping sphere animation, Dr. Khan disagreed that a dropped sphere would be similar to the impact damping between the cask and a pad because the earthquake motion is moving the cask up and down. Tr. (Khan) at 9400-01. Dr. Soler admitted that he expected a cask would not simply bounce vertically up and down but uplift and rock from side to side, depending upon the earthquake. Tr. (Soler) at 9932. Dr. Soler also agreed that during an earthquake, the frequency and peak intensity would change with time. Id. Given that both Drs. Khan and Soler agree that the seismic behavior of a cask would not simply bounce in a pure vertical direction, but potentially also rock from side to side, we find that the bouncing sphere animation and bouncing cask animation are

inconclusive to define the damping experienced by a HI-STORM cask during a seismic event.

352. Additionally, Dr. Soler implies that the ball representing 1 % damping that bounces more than 73 times is unrealistic. While the Licensing Board agrees that a 360,000 lb HI-STORM 100 cask would not likely bounce 73 times if dropped from a height of 18 inches, we must have sufficient evidence to support a finding; hence, we find no evidence was presented that the bouncing ball or bouncing cask animations would, in fact, simulate cask impact during seismic ground motions. Significantly, no evidence was proffered that the ball or cask representing 40% impact damping would better simulate the cask impact damping under seismic ground motion. The Licensing Board finds that the bouncing ball or bouncing cask animations are inconclusive to which damping ratio best represents the HI-STORM cask impact damping when subject to seismic ground motion.

353. Thus, the Licensing Board finds insufficient evidence that impact damping between the HI-STORM 100 cask and the concrete storage pad of 5 percent for a 2,000-year earthquake or 40 percent for a 10,000-year earthquake is reasonable.

Acceptable Angle of Rotation.

354. Without consideration of the effects from soil structure interaction, Holtec predicted a maximum angle of rotation for a single cask of approximately 10 degrees for a coefficient of friction of 0.8 and a 10,000-year earthquake. Tr. (Soler) at 6031. The HI-STORM 100 cask will "theoretically" tip over if the cask tipped at an angle of approximately 29 degrees from vertical. Tr. (Soler) at 6033-34. However, Dr. Singh agreed that the cask could tip over if there is residual momentum when the cask reaches approximately 29 degrees (point where the center of gravity is over the corner of the cask). Tr. (Singh) at 6110.

355. In a paper presentation, Drs. Soler and Singh state that "[a]fter a certain threshold value, the response is maximum tilting of the cask axis increases rapidly with increase in the [zero period acceleration] level."[52] State's Exh. 174, *Seimic Response Characteristics of HI-STAR 100 Cask System on Storage Pads* (January 1998) at 15-16. During cross examination in this case, Dr. Soler disagreed with the quote from his publication in that he did not agree that after a certain point, the maximum tilting would "rapidly" increase as the zero period acceleration increased. Tr. (Soler) at 6032. Additionally in the HI-STAR publication, Drs. Soler and Singh recommend that the maximum rotation of the cask be set to 25 percent of the ultimate cask tip over value. Id. However, in this case, the Applicant did not offer evidence concerning the "ultimate" cask tip over value. The Licensing Board notes that 25 percent of the "theoretical" tip over value of 29 degrees is 7.25 degrees. We note that Dr. Singh testified that the HI-STAR cask is more likely to tip over than the HI-STORM cask because of the HI-STAR's lower height to diameter ratio.

356. For the HI-STORM 100 cask, to ensure an adequate safety factor to prevent cask tip over, Dr. Soler opined that the maximum excursion of the top of the cask should not exceed half the radius (33.16 inches). Id. at 6034-35. When considering the maximum excursion of the top of the cask of 33.16 inches with a cask height of 231.25 inches, the Licensing Board finds that to ensure an adequate margin of safety a maximum allowable rotation angle is 8.15 degrees from vertical.

357. In two reports, Holtec estimated maximum cask rotation angles of

---

[52]The paper was authored jointly by Dr. Soler, Dr. Singh, and Martin G. Smith.

approximately 10 degrees for a 10,000-year earthquake and a coefficient of friction of 0.8 which exceeds the maximum rotation angle of 8.15 degrees suggested by Dr. Soler. Tr. (Soler) at 6031, Applicant's Exh. 86d at 13. In Applicant's Exh. 86d, the rotation angle from vertical was calculated based on "50 percent of peak-to-peak excursion instead of the maximum excursion of the top of the cask from the location at the start of the run. Applicant' Exh. 86d. We note that 50 percent of the maximum peak-to-peak excursion is lower than the maximum excursion recorded at the top of the cask. Thus, the Licensing Board finds that the rotation angle calculated in Applicant's Exh. 86d may not reflect the maximum angle of rotation that occurred during the simulations.

358. Additionally, Dr. Soler admitted, based solely on the animation, that in some runs, casks (*e.g.*, cask 5) other than cask 1, in which he only quantified cask 1's movement, appear to move more than the cask 1. Tr. (Soler) at 5762. In an additional 8 cask simulation for a 2,000-year earthquake, Dr. Soler actually quantified the movement for cask 1 and cask 5 where he showed the maximum excursion of the top of the cask for cask 1 was 3.4 inches and for cask 5 was 10.5 inches. Applicant's Exh. 225 at 18.

359. Dr. Singh claims that the "actual" maximum angle of rotation for the cask would be "much less" due to the "huge conservatisms" built into Holtec's model. Tr. (Singh) at 6035. We find, notwithstanding any actual conservatisms, the record is devoid of any evidence quantifying the "huge conservatisms" claimed by Dr. Singh, consequently, we find any claimed conservatism in the Holtec nonlinear analyses unreliable.

360. Based on the Applicant's existing analyses, the Licensing Board finds that the Applicant has not demonstrated that the HI-STORM 100 cask will not exceed the tip over

angle of rotation with a margin of safety of 8.15 degrees from vertical.

Non-linear Analysis Should Be Validated With Shake Table Data.

361. Dr. Khan opines that the only way to validate Holtec's seismic analysis is to benchmark the cask displacement with actual shake table test data. Khan/Ostadan Tstmy, Post Tr. 7123 at A.34. Consistent with his opinion with respect to the Holtec results that the analyses must be validated with test data, Dr. Khan testified that he could also not claim his parametric study results were correct without first validating his results with test data and calibrating his damping, stiffness, and rocking values. Tr. (Khan) at 7178-79. Moreover, Dr. Khan opined that the nonlinear cask seismic analyses should be validated with test data regardless of the analyst's confidence in his solution. Id. at 9425.

362. As an example of the NRC philosophy supporting the need to validate nonlinear seismic analysis with test data, Dr. Khan referred to NRC Reg. Guide 1.100, which endorses IEEE 344-1987 requirement.[53] Khan/Ostadan Tstmy, Post Tr. 7123 at 13-14. IEEE, section 6 states that "[t]he analysis method is not recommended for complex equipment that cannot be modeled to adequately predict its response. Analysis without testing may be acceptable only if structural integrity alone can ensure the design-intended function." Id. IEEE provides "good guidelines" for nonlinear seismic analysis which have been applied in the qualification of structures. Tr. (Khan) at 9431-32. IEEE requires test data validation for Class 1E electrical equipment defined as "electrical equipment and system that are essential to emergency reactor shutdown, containment isolation, reactor core cooling,

[53]Institute of Electrical and Electronic Engineers, Inc., *Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations.*

and containment and reactor heat removal, or are otherwise essential in preventing a significant release of radioactivity to the environment." Id. at 9428-29, Applicant's Exh. 222.

363. Dr. Khan testified that IEEE's provision that "analysis without testing may be acceptable only if structural integrity alone can ensure the design intended function" is not applicable to cask analysis because a designer cannot rely on its judgment that its design is adequate. Tr. (Khan) at 9137-42. In this case, shake table data would validate the cask dynamic response or whether the cask tips over under various ground motions.

364. To rebut Dr. Khan's suggestion that IEEE provides reasonable guidance in this case, Dr. Singh testified that the intended class of components (Class 1E) is not very large and can be placed on a shake table and that "some" electrical and mechanical equipment have small tolerances which alter their functionality. Tr. (Singh) at 9680-81. The Licensing Board finds in light of Dr. Luk's testimony concerning the availability of a large shake table facility which could accommodate a full scale cask (Tr. (Luk) at 15569-72), Dr. Singh's opinion with respect to the size of components no longer has merit. The Licensing Board further finds that absent verification test data, the nonlinear seismic analysis of the cask system and any implied tolerances of the cask system are uncertain.

365. Holtec also points to its cask tip over analyses to demonstrate that the canister will not be breached. Singh/Soler Tstmy, Post Tr. 5750 at 29. The Applicant did not offer the Holtec cask tip over analyses into the record. Additionally, Holtec assumed an initial angular velocity of zero in its non-mechanistic tip over analysis. Tr. (Bartlett) at 12870-71. Accordingly, if the cask tips over due to seismic excitation, Dr. Steve Bartlett opines, then the angular velocity could be greater than zero and thus, invalidate Holtec's cask tip over analyses

and conclusions that the canister would not breach. Id. at 12913-17. Concurring with Dr. Bartlett, in response to a question if in fact, the cask tips over as a result of a seismic event what would be the initial angular velocity, Dr. Cornell stated, if "that's an interesting question physically, actually. The initial velocity would probably clearly have to be something greater than zero or it would not be moving in that direction, that is tipping over." Tr. (Cornell) at 7978. The Licensing Board finds whether the canister is breached as a result of cask tip over is dependent upon whether the cask in fact tips over during a seismic event. Hence, the Licensing Board finds it is circular reasoning to justify not validating the nonlinear cask analyses based on the structural integrity of the cask system which is itself grounded in the assumption that the nonlinear cask stability analysis shows no tip over. Thus, the Licensing Board further finds Holtec's drop tip over analyses cannot be used to ensure the structural integrity of the cask absent testing.

366. The Licensing Board finds the philosophy encompassed in Reg. Guide 1.100 and IEEE persuasive in that it imposes test validation of nonlinear analyses for "equipment essential in preventing a significant release of radioactivity to the environment." The Licensing Board finds that the HI-STORM cask system is essential in preventing a significant release of radioactivity to the environment. The Licensing Board further finds that to prevent a significant release of radioactivity to the environment the Applicant must demonstrate that the HI-STORM 100 cask will not tip over.

367. Because of uncertainties in the analysis, Dr. Luk confirmed he and the individuals in "his group" view shake table testing as "useful" in confirming his analysis. Tr. (Luk) at 11569, 11572. In fact, during the June 2002 hearings, Dr. Luk updated his previous

testimony that he expected a "true state-of-the-art" shake table test facility that could accommodate a full scale cask would be available at the University of California at San Diego next spring due to a recent grant from the National Science Foundation. Id. at 15569-72. Additionally, the Staff plans to request funding for shake table tests. Id. at 11570. Dr. Luk testified that he could "almost assure [that] the cask will not be damage or destroyed on the shake table." Tr. (Luk) at 7111. Through its funding, the National Science Foundation endorse large scale shake table testing for some purposes. Although there is no direct testimony in the record, the Staff itself must find a benefit to cask shake table tests given its intent to seek funding. Notably, if Dr. Luk misrepresented the Staff's intention, the Staff did not proffer any witnesses to correct Dr. Luk's testimony.

368. Dr. Cornell opined that a scaled down shake table test may introduce uncertainties. Tr. (Cornell) at 8025. Furthermore, Dr. Cornell disagreed that shake table testing was necessary to verify input parameters - "we do lots of nonlinear analyses in advanced earthquake engineering. We have done some shake table testing, and the shake table testing is used to verify the general nature of those models. But it's not used for each and every application." Tr. (Cornell) at 7975.

369. Notwithstanding Dr. Cornell testimony, he agreed that physical test data would reduce the amount of uncertainty in a seismic assessment. Tr. (Cornell) at 7979.

370. In sharp contrast to other expert opinions expressed in this case concerning the need or desire for shake table tests, Dr. Singh emphatically stated "is [shake table tests] necessary . . . absolutely no." Tr. (Singh) at 9682. He further disagreed that shake table tests have any value in verifying cask seismic analyses and that "it is absolutely impossible to run a

shake table test [to simulate the seismic response of a cask] and get a [sic] meaningful data." Id. at 9682, 9684. Furthermore, contradicting other witnesses, Dr. Singh assured the Licensing Board that shake table tests would "confer no new knowledge" or information because shake table testing is "simply not feasible" based on the size of the cask and that the coefficient of friction varies with time. Id. at 9682-84, 9728. Dr. Singh opined that a shake table test would not provide any information because "the condition of a pad on a simulated cask with a scale model or even full size" cannot be replicated on a shake table. Id. at 9728-29. Contrary to other witnesses, Dr. Singh offered that "any engineer trained in basic mechanics would not need the shake table to come up - to get any new information" and shake table tests would serve no useful purpose "whatsoever" in this case. Id. at 9729, 9731-32. Dr. Singh submits that "I know to absolute technical certainty, as long as the laws of nature don't change on us, this cask will not tip over under the earthquakes postulated for the PFS site. There's absolutely no doubt." Tr. (Singh) at 9750.

371. However, Dr. Singh did acknowledge that shake table tests are necessary "when you have some ambiguities and some concerns, some possible uncertainty with respect to performance," albeit he believes there is no uncertainty with Holtec's analysis. Id. at 9731. Moreover, although both Dr. Singh and Dr. Soler initially denied ever discussing performing shake table tests with PFS (Tr. at 9732), in November 1997, Dr. Singh sought funding from PFS to verify their analytical work by conducting scale model tests on a shake table (id. at 9738-39.). *See* State Exh. 197. Dr. Singh testified that because the Staff relied upon a "simpleminded static limit," NRC requested that Holtec conduct scale model shake table tests to support the HI-STORM 100 application for a certificate of compliance at high earthquake

levels. Id. at 9739. Notwithstanding the November 1997 Holtec letter (State Exh. 193) which stated "NRC endorsed the Holtec proposal to experimentally confirm the seismic analysis approach," Dr. Singh denied ever recommending shake table tests to PFS, claiming instead the letter was "politically correct" and a result of "the guy who has the gold makes the rule," implying that NRC, not Holtec, desired the shake table tests. Id. at 9745-48 (*emphasis* added).

372. Although, Dr. Singh does not define "high earthquake levels," it appears Holtec sought funding from PFS and Pacific Gas and Electric. Id. at 9742. Dr. Soler testified that ground motions at the PG&E, Diablo Canyon facility are 0.9 g. Tr. (Soler) at 5932. The peak horizontal ground accelerations at the PFS site are estimated as 1.15 for deterministic, and 0.711 g for a 2,000-year event. Con-SER at 2-34.

373. Moreover, the November 1997 Holtec letter to PFS states "if PFS elects not to support [shake table testing], then we can provide all high seismic material stripped from Rev. 1 of the HI-STORM [topical safety evaluation report] for direct incorporation in the Skull Valley site-specific submittal, and we will proceed with only anchored cask certification on this new docket." Tr. (Singh) at 9744; State Exh. 197 (*emphasis added*).

374. In almost stark contrast to their views with respect to shake table tests or "experimental tests," Drs. Singh and Soler testified that "[t]o properly validate a friction model for a free standing structure, it is necessary to check the model you propose against a known analytical solution or against experimental results" to demonstrate the code can produce well known problems. Singh/Soler Tstmy, Post Tr. 5750 at A.132.

375. When weighing the panoply of testimony concerning the advantages,

disadvantage, and need for shake table testing, the Licensing Board finds that the testimony in favor of shake table testing is more compelling. Moreover, the Licensing Board finds the testimony proffered by Dr. Singh, and to a lesser extent Dr. Soler, inconsistent and unreliable. Further, given Drs. Singh and Soler's financial interest in the outcome of this case, we note that any finding that results in the need to conduct shake table testing negatively impact Holtec's financial interest. Accordingly, we further find the testimony of Drs. Singh and Soler unreliable with respect to shake table tests.

376. No expert disagreed that the results of nonlinear analyses could only be validated with test data. The experts disagreed to various degrees of the actual need for shake table data. Accordingly, the Licensing Board finds that breadth of the testimony agrees that the nonlinear analyses can only be validated with actual test data. Moreover, given that Dr. Luk anticipates a shake table test facility able to conduct full scale tests will be available in the spring, Dr. Luk's assurances that a cask would not be damaged by shake table tests, and the lack of evidence which convinces this Licensing Board of the accuracy and reliability of the Holtec 2,000-year report, we cannot find that the Applicant has met its burden to demonstrate that free standing HI-STORM 100 casks will not tip over under at 2,000-year earthquake at the PFS facility.

The NRC Sponsored Luk Report Does Not Verify the Holtec Seismic Analyses.

377. Commencing in March 1999, the NRC commissioned a generic study to evaluate dry spent fuel cask storage. Tr. (Luk) at 6763. The purpose of the generic study was to see how casks perform under seismic conditions and to provide guidance concerning

freestanding casks.[54], [55] Tr. (Guttmann) at 6835-36. At first the generic study did not include any site specific inputs but later included conditions at Hatch and San Onofre. Tr. at 6763-67, 11614. The Southern Company Hatch plant uses free standing Holtec casks but unlike the PFS site, the Hatch site has very low ground motions – 0.15 g (vertical) and 0.1 g (horizontal). Tr. (Luk) at 6914-15. The San Onofre ISFSI does not use free standing Holtec casks; it uses horizontal modules tied together into one unit. Tr. (Luk) at 6915.

378. The Luk confirmatory analysis specific to the PFS site grew out of the generic cask study and results from the confirmatory PFS site specific analysis will eventually be included in the generic study.[56] Tr. (Luk) 7023-24. For the period March 1999 to November 2002, NRC has financed the generic and PFS site specific studies to the tune of one million dollars – $200,000 of which relates specifically to the PFS site-specific confirmatory analysis. Tr. (Luk) at 6854-56; *see also* Tr. (Turk) at 6884.

---

[54]Although Mr. Guttmann maintained that the Standard Review Plans are still applicable to the entire country, Dr. Luk testified that the generic study was initiated because "[t]he current Standard Review Plan is [ ] adequate to go through the licensing efforts in regions with low seismic loading" but with relatively higher seismic loading in the west, "there is a concern to whether the current Standard Review Plan is adequate to support the . . . licensing review process." Tr. (Luk, Guttmann) at 6838-39.

[55]Sandia National Laboratories was contracted to "establish criteria and review guidelines for the seismic behavior of dry cask storage systems." Staff Exh. P at 3.

[56]The State objected to the admission of the Luk report based on the late notice and its availability. Although, the we denied the State's motion to strike the Luk report (Tr. at 6901-03), the Licensing Board notes it is disturbed by the Staff's untimeliness in providing the results to the parties in light of Mr. Guttmann's testimony that he initiated the analysis to "assist the State in understanding the complexities of the analysis" (Tr. at 6843, 6846-47, 6874) and that the 2,000-year DBE status report, including results, was completed at the end of October 2001 (Tr. at 6861, 6877-79).

379. Under contract with the Staff, Vincent Luk developed a three dimensional coupled finite element model to evaluate the seismic stability of the HI-STORM 100 casks at the PFS site. Luk/Guttmann Tstmy, Post Tr. 6760 at A3(b), A.6. Mr. Guttmann testified that he requested a cask stability analysis for the PFS site to "assist the State in understanding the complexities of the [cask stability] analyses." Tr. (Guttmann) at 6843, *see also* 6846-47, 6849. Mr. Guttmann did not affirm that the Luk analysis would also assist the Staff because the Staff "expected" the Luk analysis to be confirmatory. Tr. (Guttmann) at 6847.

380. Dr. Luk summarized his evaluation in *Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage (PFS) Facility*, Rev. 1 (March 31, 2002) ("Luk report").[57] Luk/Guttmann Tstmy, Post Tr. 6760 at A.3(b). Using the ABAQUS/Explicit code, a single elastic cask was modeled on a flexible concrete pad with soil cement[58] adjacent and beneath the pad, and a soil layer beneath the soil cement. Staff Exh. P at 5-6; Luk/Guttmann Tstmy, Post Tr. 6760 at A.7. ABAQUS is a general purpose nonlinear finite element code. Tr. (Luk) at 6768. Contact elements were modeled at three interfaces - 1) the cask/pad interface, 2) the pad/soil cement interface, and 3) the soil cement/soil interface. Staff Exh. P at 5-6. The model considered six horizontal layers in the soil foundation to a depth of 140 feet from the surface. Luk/Guttmann Tstmy, Post Tr. 6760 at A.12. Dr. Luk used contact elements to

---

[57]The other parties in the proceeding received the Luk report on April 2, 2002, the day after the deadline for filing prefiled testimony.

[58]Note the term "soil cement" beneath the storage pad in the testimony of Vincent Luk and his *Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage (PFS) Facility*, Rev. 1 has typically been referred to as "cement-treated soil" in other portions of the hearing. *See e.g.*, Tr. at 6921-23. Soil cement adjacent to the storage pad was also modeled. Id.

model the interface between the cask and the pad. Tr. (Luk) at 6809. Dr. Luk evaluated a 2,000-year and a 10,000-year earthquake for the PFS site.[59] Id. at 6760; Luk/Guttmann Tstmy, Post Tr. 6760 at A.6.

381. Dr. Luk varied three input parameters (the seismic ground motion, the coefficient of friction in the interfaces, and soil parameters) in running 13 simulations of a single cask on a pad with the surrounding soil cement and soil foundation. Staff Exh. P at 30-32. An additional simulation ("Model Type 2") simply modeled a single cask on a pad with no soil foundation for a 2,000-year earthquake. Id. at 30. At the direction of the Staff, Dr. Luk also ran one simulation of Model Type 3, a single cask with the "dead" loads of seven adjacent casks on the pad and 4 adjacent pads each with a "deal" load of 8 casks for a 2,000-year earthquake. Id.; Tr. (Luk) at 7004, 7025.

382. The Staff supplied Dr. Luk with the input values to be used in the PFS analysis (*e.g.*, single set of time histories for a 2,000-year and 10,000-year earthquake, cask and pad design, material properties of the cask and pad, "upper bound," "lower bound," and "best estimate" soil profiles, and the coefficient of frictions values). Luk/Guttmann Tstmy, Post Tr. 6760 at A.6; Tr. (Luk) at 6771, 6824, 6919-23. Dr. Luk did not independently verify any of the input values. Tr. (Luk) at 6923-24. The Luk model accounted for the mass proportional Rayleigh damping, but not the stiffness proportional damping. Staff Exh. P at 9; Tr. (Luk) at 6793-94. The stiffness proportional damping was ignored to reduce the computational time for each simulation. Tr. (Luk) at 6794. Although excluding the stiffness

[59]The ground motion associated with the 1971 San Fernando Earthquake (Pacoima Dam) was also evaluated. Luk Tstmy, Post Tr. 6760 at A.6.

proportional damping would result in high frequency response effects, the high frequency response effects would not have much impact on the structural response of the cask. Id. at 6794-95.

Potential Conflict of Interest.

383. A review panel consisting of three NRC Staff and four industry representatives provided technical advice and input to the generic and site specific cask stability studies conducted by Dr. Luk and his associates. Tr. (Luk) at 6994-6996, 7052-54. The hearing testimony revealed that industry representatives include representatives from Southern Company, San Onofre Nuclear Generation Station, the Electric Power Research Institute, and a private consultant, Dr. Robert Kennedy. Id. at 6995. Southern Company and Southern California Edison, owner of San Onofre, are members of PFS. FEIS (Staff Exh. E) at Fig. 1.3; Con-SER at 17-1. The role of the industry panel members is to provide recommendations concerning the analysis methodology and range of input parameters used by Dr. Luk. Tr. (Luk) at 7054. The advisory panel, including industry representatives, reviewed the generic Luk finite element model and provided their comments and recommendations. Id. at 7076. Dr. Luk met with the advisory panel on three occasions, including November 2001, to discuss the details of the PFS model. Id. at 7077-78. The panel provided comments on the completed 2,000-year analysis for PFS at the November 2002 advisory panel meeting. Id. at 7082-83. The advisory panel meetings were not open to the public. Id. 7083.

384. Although Dr. Luk testified that he had no knowledge that any of the industry representatives were associated with PFS (Tr. (Luk) at 6995-96), the Licensing Board

presumes the Staff was aware that representative from PFS member companies were on the advisory panel. Tr. (Luk) at 7053-54; 7081-86. However, we note that "fundamental fairness" to the conduct of a licensing proceeding mandates the "disclosure of all potential conflicts of interest," whether or not a party believes them to be material and relevant to a licensing proceeding. Long Island Lighting Company (Shoreham Nuclear Power Station Unit 1), LBP-82-73, 16 NRC 974, 979 (1982). Disclosure is necessary to enable the Licensing Board to determine the materiality of the potential conflict of interest. Id. Due to the lack of disclosure of potential conflicts of interest of the industry representatives on Dr. Luk's advisory panel, compounded by the lateness of the availability of the Luk report to the State, the State has had little or no opportunity to probe the backgrounds of the advisory panel and its influence on the Luk methodology and analysis during discovery. Because of the constraints placed on the State to probe the issue and potential inability to raise it themselves, notwithstanding Dr. Luk's testimony that the advisory panel had not inappropriately interfered with his PFS site work, the Licensing Board cannot find that at least two industry representatives on Dr. Luk's advisory panel have no conflict of interest with the outcome of the Dr. Luk's cask stability analysis for the PFS site. Thus, the Licensing Board finds that it must weigh the potential conflict of interest from PFS member companies in its assessment of the Luk report.

Experience in Modeling the Seismic Response of Free Standing Dry Storage Casks.

385. Dr. Luk's sole experience in modeling the free standing dry storage casks includes the site specific analysis for PFS, Hatch, and San Onofre. Tr. (Luk) at 6764, 6914-16, 7051. The site conditions for Hatch and San Onofre are not similar to those at the

proposed PFS site. Although Dr. Luk modeled ground motions in excess of those estimated for Hatch, the Hatch ground motions are approximately 0.15 g horizontal and 0.1 g vertical. Tr. (Luk) at 6915-16. The Hatch site store 12 casks on a concrete pad in a 2 x 6 array, however, Dr. Luk modeled a square pad with a 2 x 2 array. Id. at 6993. The casks proposed for San Onofre are 3 unanchored, rectangular, horizontal casks tied together, unlike the individual cylindrical HI-STORM 100 casks. Tr. (Luk) at 6915, 7054-56. Dr. Luk acknowledged that the seismic response of the horizontal casks at San Onofre is "very different" when compared to the cylindrical cask proposed for PFS. Id. at 7056. The Licensing Board finds that the Hatch site conditions are different from those proposed for PFS. We further find that the San Onofre site conditions and the facility design are substantially different from those proposed for PFS. Therefore, Hatch and San Onofre do not provide relevant modeling experience for the PFS design or site conditions.

386. Robert Dameron, Anatech Corporation, and Po Lam, Earth Mechanics, are co-authors of the Luk report. *See* Staff Exh. P. Mr. Dameron is an engineer with finite element experience. Tr. (Luk) at 6765-66. Mr. Lam, a trained seismologist (id. at 6765-66), developed the soil foundation model. Id. at 7037.

387. Dr. Luk testified that the most important "state-of-the-art" features in his model are, 1) the interface between the substructures and 2) the modeling of soil structure interaction. Tr. (Luk) at 6979.

388. Dr. Luk has no expertise in soil dynamics – an area that directly relates to two critical areas featured in his model. Tr. (Luk) at 6917; 7036. Without input from Mr. Lam, Dr. Luk developed the soil structure interaction model. Id. at 7037. Later in the hearing, Dr.

Luk corrected his earlier testimony that he relied on co-authors of the Luk report for soil structure interaction expertise. Id. at 6917.

389. Initially Dr. Luk testified that he has no expertise in soil structure interaction but later recanted his testimony to claim he has such expertise. Id. Dr. Luk claims the evaluation of soil structure interaction effects is nothing more than the systematic evaluation to address the dynamic coupling between a structure and soil. Id. at 6917, 7036. We previously described the concept of soil structure interaction in Contention Part D, Dynamic Analysis.

390. Dr. Luk professes to be an expert in soil structure interaction based on his work over the "past few years" in evaluating coupling between components in a nuclear power plant. Id. at 7038. Dr. Luk's own soil structure interaction experience is limited to "the past few years" which would encompass the analyses he performed for the Hatch and San Onofre ISFSIs, we find that the record bare in its support that Dr. Luk alone is qualified to model soil structure interaction effects.

391. The Licensing Board finds insufficient evidence to find that Dr. Luk's associates in his analyses are qualified to accurately model the soil dynamics or the soil structure interaction effects.

392. The Licensing Board also finds that Dr. Luk's experience in the nonlinear modeling of the seismic behavior of cylindrical, free standing casks is limited to his generic study and the Hatch analyses. We therefore find that Dr. Luk does not have experience in the nonlinear modeling of the seismic behavior of cylindrical free standing casks supported by cement-treated soil and a relative soft clay foundation at ground motions equal to or greater

to the 2,000-year earthquake at PFS.

393. Mr. Guttmann admitted he has no experience in performing the seismic analysis of free standing casks. Tr. (Guttmann) at 6917. The Licensing Board finds Mr. Guttmann is not qualified as an expert in technical or scientific matters concerning the nonlinear modeling of the seismic behavior of free standing casks.

The Luk Report Does Not Confirm Holtec's Analyses.

394. The Holtec witnesses agreed that the Luk report did not confirm Holtec's methodology but similarly concluded that the cask would not tip over. Tr. (Singh, Soler) at 6122-23, Tr. (Soler) at 6077, 9755. Dr. Soler testified that the Staff's analyses "studies a different problem than [Holtec] simulated either with DYNAMO or VisualNastran." Tr. (Soler) at 5898. Dr. Soler further testified that the Staff analysis "models certain features of the problem in a different manner than [Holtec]." Id. Dr. Soler did not know why there was a difference in the results between his analysis and the Staff analysis for a 10,000-year earthquake with a coefficient of friction 0.8. Id. at 5898-99. Although Dr. Soler claims the Luk report magnitude of excursions are in the same order with those determined by Holtec (Tr. (Soler) at 5998), Dr. Luk testified that the Holtec and Luk results cannot be compared. Tr. (Luk) at 6949-51.

395. Dr. Luk testified that his and Holtec's methodology are "entirely different" and some of the other important imput parameters or "critically different." Tr. (Luk) at 6950. Dr. Luk opines that his and Holtec's results should not be directly compared due to the different methodologies employed and the different input parameters. Tr. (Luk) at 6950-51.

396. Additionally, Dr. Luk did not compare his soil structure interaction effects for

a 2,000-year or 10,000-year earthquake with those predicted by PFS. Id. at 6940-41. Similarly, Dr. Luk did not compare his deconvoluted time histories for a 2,000-year or 10,000-year earthquake with those predicted by PFS at similar depths. Id. at 6941.

397. Like Holtec's analyses results, the Luk report results have not been benchmarked or compared with any physical data, such as shake table tests. Id. at 6958.

398. When probed about his confidence in the results from a "very complicated model" with large amounts of data, Dr. Luk further testified that the results are dependent on the input parameter. Tr. (Luk) at 6987. The Luk model has 4,124 elements - 864 elements model the cask, 384 elements model the storage pad, 848 elements model the soil cement adjacent and beneath the pad, and 2,000 elements model the soil foundation. Id. at 7027. Given the complexity of Dr. Luk's model, at this juncture we think it appropriate to heed Dr. Cornell's affirmation not to be too enamored with the computer program itself. Tr. (Cornell) at 8024. Furthermore, Dr. Luk acknowledges the value in validating nonlinear results with test data. Tr. (Luk) at 11571-72.

399. Due to the lack of test data to validate his results, Dr. Luk relies on sensitivity analyses and the experience he has gained in his NRC related study to substantiate his model. Id. at 6987-88. In an effort to show his model is accurate, Dr. Luk points to his seismic analysis at Hatch where the ground motion was increased to demonstrate that his model could show cask tip over. Id. at 6988. Although Dr. Luk relies on the Hatch analysis to support the accuracy of his PFS model, he also testified that the PFS model was modified to simulate the soil cement layer at the PFS site. Tr. (Luk) at 7026. Moreover, Dr. Luk admitted that the soil cement (cement-treated soil) layer, with interfaces both above and beneath the

layer, "actually caused quite a bit of difficulties in the simulation portion." Id. at 7028.

400. As a result of the additional interface layer in the PFS model, the Licensing Board finds that no documentation has been proffered to show that simulations with the Hatch model provide assurance that the results obtained from the Luk-PFS model are reasonably accurate. Moreover, the record contains no evidence that the results generated from the Hatch model, in fact, accurately predict the seismic behavior of the cask, storage pad, and foundation. Notwithstanding the fact that no party presented documentation to support Dr. Luk's testimony that his model would show cask tip over, the Licensing Board finds that the Hatch model may in fact show cask tip over at higher ground motions, but based on the apparent complexity of the model, the differences in the PFS and the Hatch model, and the lack of test data to validate any results, there is insufficient evidence to conclude the PFS model developed by Dr. Luk accurately or conservatively estimates cask response, including displacement, angle of rotation, and tip over.

401. Furthermore, Dr. Luk's cumulative experience in modeling and predicting the seismic behavior of free standing casks is limited to the three site specific cases analyzed under contract with NRC - PFS, Hatch, and San Onofre. Tr. (Luk) at 6764, 6914-16, 7051. Dr. Luk testified that he modified the generic model for PFS. Id. at 7026. The record is absent a showing that the experience gained from modeling horizontal, rectangular casks at San Onofre is transferable to modeling vertical free standing casks at PFS. *See* ¶ __ LUK LIMITED EXPERIENCE *supra*. Thus, the Licensing Board finds that Dr. Luk's limited experience in performing seismic cask stability is insufficient to find the Luk model accurately or conservatively estimates cask response, including displacement, angle of rotation, and tip

over

402. In the Model Type 1 simulations, Dr. Luk opined that modeling a single cask on a pad is adequate because that cask rotations will be larger if the casks movement is in phase and independent of other casks. Tr. (Luk) at 6774. Dr. Luk verified his assumption with a single simulation (Model Type 3) where he modeled a single cask on a pad with the dead loads of 7 adjacent casks and 4 adjacent pads with the dead loads of 8 casks per pad. Id. at 6776. Dr. Luk did not model 8 casks on a pad because of additional computer time and memory needed for an 8 cask model. Id. at 6779-81, 6956-57. Dr. Luk assumed that all 8 casks on a pad actually behave independent of other casks on the pad. Id. at 6779-80. However, Dr. Luk did not confirm his assumptions by running a model with 8 casks moving. Id. at 7066.

403. Dr. Luk testified that based on a generic study without casks, he concluded that the storage pads are dynamically independent of one another. Tr. (Luk) at 7030. The Staff offered no documentation to support Dr. Luk's testimony, but claim the sensitivity study consisting of a single simulation with Model Type 3 confirmed Dr. Luk's earlier conclusion that the storage pads are independent of other pads. Id. at 7031.

404. Based on a sensitivity analysis for a 2,000-year earthquake, Dr. Luk claims that the results are indifferent to the location of the single cask in his analysis. Tr. (Luk) at 6956. There was no similar sensitivity analysis for a 10,000-year earthquake. Id. Contrary to Dr. Luk's assumption, in a 10,000-year simulation, cask # 5 moved significantly more than cask # 1 – 10.5 inches versus 3.4 inches, respectively. *See* Applicant Exh. 225 at 29. The Licensing Board finds that the cask stability results may be dependent upon the specific location of the

cask on the pad.

Luk Report Shows Significant Soil Structure Interaction Effects.

405. Dr. Luk concluded a "presence of significant [soil structure interaction] effects, as shown in Figures 17 through 19" in the Luk report. Luk Tstmy, Post Tr. 6760 at A.20; 6930. Dr. Luk testified that soil structure interaction is a "key feature" in his model. Id. at 6770. Figure 17 is raw data analysis results to compare the ground motion acceleration for a single node on a storage pad with the free field[60] ground motion acceleration. Tr. (Luk) at 6803-04. Figure 17 shows increased accelerations due to soil structure interaction compared to the free field ground motion. Id. at 6935-36. Dr. Luk concluded that the presence of a structure increases the ground motion acceleration compared to the free field acceleration. Id. at 7012.

Modeling PFS Foundation Soils

406. The Staff directed Dr. Luk to use a coefficient of friction of 0.31 at the pad and soil cement beneath the pad (cement-treated soil). Tr. (Luk) at 6924-25.

407. Dr. Luk initially contemplated how to incorporate the plastic behavior of soil into his model but, in part, because of the two to three fold increase in computer time this would entail, he instead used an elastic body to simulate foundation soils. Tr. (Luk) at 11548.

408. As part of the PFS site-specific confirmatory analysis, Dr. Luk also had an eye on developing a practical analytical model that could be used by the nuclear industry:

[P]eople, of course, can argue within the technical arena there's no such

[60]Dr. Luk testified that free field acceleration is not influenced by the presence of the concrete pad or the edge of the soil foundation. Tr. (Luk) at 7012.

> behavior as elastic, but we are very much concerned with eventually it is one of our tasks to develop appletical [sic] analysis model that can be used by people in the industry. By modeling the plastic behavior in the model, we are going to talk about -- we're going to change the order of magnitude of the size of the model, maybe by a factor of at least two to three.

Tr. (Luk) at 11549.

409. In the PFS site-specific confirmatory analysis, Dr. Luk treats the interface layers under the storage pad as granular material and models the interface nodes as a frictional material, such as a sand. Tr (Bartlett) at 10530; Luk Report at Table 8. As understood in the geotechnical engineering profession, frictional materials are those without any cohesion — generally sands and gravel that do not have a large component of clay or maybe silts in them. Tr. (Bartlett) at 11402.

410. In Table 8, column 2, of the Luk Report, $\mu 1$ is the interface coefficient of friction between the casks and the storage pad; $\mu 2$ represents the coefficient of friction at two different interfaces: between the bottom of the pad and the top of the cement-treated soil; and between the bottom of the cement-treated soil and the top of the upper Bonneville clays. Tr. (Bartlett) at 10348.

411. To model the interfaces, including interfaces above and below the cement-treated soil ($\mu 2$), Dr. Luk used what he referred to as Coulomb's law of friction, $F = \mu n$, where F is the frictional resistance, μ is the coefficient of friction, and n is the normal stress. Tr. (Luk) at 11510; Tr. (Bartlett) 11407. Dr. Luk observed that "the so-called coefficient of friction at the interface between two bodies is an estimate of the friction in the systems of one body in motion with respect to the other, basically, fitting Coulomb's Law of Friction." Tr (Luk) at 11509-10. In particular, Dr. Luk testified that "Coulomb's Law of Friction is a

description of the frictional resistance at the interface, as material properties at the interface." Tr. at 11510.

412. By treating the interface conditions as frictional material, the State is adamant that Dr. Luk's model does not represent the actual PFS design or the PFS site soils. Tr (Bartlett) at 10375-77. Of particular concern to the State is the way in which Dr. Luk has modeled the interface conditions, $\mu 2$, and also the way in which Dr. Luk's model does not account for the post-yield behavior of the upper Bonneville clays. Tr. (Bartlett) at 11481-82.

413. The actual design of the storage pad system at the PFS site is undisputed: a one to two foot thick cement-treated soil layer, the top of which is bonded to the underside of the concrete storage pads and the bottom of which is bonded to the top of the native soil layer (*i.e.*, upper Bonneville clay). Applicant Exh. JJJ; Trudeau/Wissa Tstmy, Post Tr. 10834 at 24-25; Bartlett Tstmy (Soils), Post Tr. 11822 at 4; Con-SER at 2-59. Unlike structural fill, which derives its resistance to seismic forces from friction, cement-treated soil derives its resistance to seismic forces from cohesion. Tr. (Trudeau) at 10839-40.

414. PFS's design intent to withstand seismic forces from the design basis earthquake is to rely on cohesion from bonding at the interface layers (the upper Bonneville clays interface with the laminated cement-treated soil lifts; and the cement-treated soil with the underside of concrete pad) to transfer the horizontal earthquake forces downward from the storage pad to the underlying clay soils. Trudeau/Wissa Tstmy post Tr. XX at 23; Tr. (Bartlett) at 10375-76).

415. The soils characterization at the PFS site was conducted by Stone & Webster but these soil properties are not used in the Luk model. In addition, Geomatrix developed

dynamic soil properties – upper bound, best estimate, and lower bound for the PFS site. In his model, Dr. Luk uses the dynamic soil properties developed by Geomatrix. Tr. (Bartlett) 11502; Luk Report, Table 8.

416. Clays are not a granular material – they are a relatively soft plastic material. Tr. (Bartlett) at 10377. The clays that PFS is relying upon to transfer earthquake forces – the upper Bonneville clays – derive their strength from cohesion (i.e., the undrained shear strength) not from friction. Id.

417. Cohesion is a material property – it is the shear strength or resistance to sliding within the material. Tr. at 11687. Dr. Luk admitted that his model does not incorporate cohesive strength of the soils. Tr. (Luk) at 6787.

418. Cohesion is generally not thought of as a dynamic property – it is a shear strength property that is measured by a static test. Tr. at 11706. Cohesion is also an interface property – it is the strength of the bond at the interface between two layers.[61] Tr. at 11688.

419. Cohesion was not an inherent property included in the dynamic soil properties that were developed by Geomatrix for the PFS site. Tr. (Bartlett) at 10409; 11691. The dynamic soil properties given to him by the Staff and apparently developed by Geomatrix are: maximum shear modulus, soil density, Poisson's ratio, an estimation of shear modulus degradation and damping degradation as a function of strain. Tr. (Bartlett) at 11710. The State's soils expert testified that he knows of no theory of obtaining cohesion of the upper Bonneville clays from dynamic soil properties developed by Geomatrix. Tr. (Bartlett) 1711.

[61] Strictly speaking the term "adhesion" refers to the condition between two dissimilar materials; "cohesion" is the failure within the material itself. Tr. (Bartlett) at 11417.

420. Here, the Board gives particular deference to Dr. Bartlett. His expertise in soils is unquestioned. Dr. Luk, on the other hand, admits that he has no expertise in soils and that he relied on a seismologist for the soil input to the numerical model. Further Dr. Luk admits that shear strength (i.e., cohesion) is not represented in his model. The Board, therefore, finds that the Luk model does not incorporate cohesion through the use of the dynamic soils properties developed by Geomatrix.

421. Frictional materials like sand or gravel derive their strength from grain to grain contact and, in general, derive their resistance to sliding from the normal force. Such materials act differently from clays under non-seismic and seismic loads. Tr. 10377. In the Luk model, when the horizontal forces are large enough then sliding occurs. But as the following illustrates this model does not describe the conditions at the PFS site.

422. In the case of frictional material, and taking first the simple case of gravity loads from the cask-pad system at PFS, the casks and pad impose a normal stress of about 2 kips per square foot (ksf). For a μ2 value of .31 in Luk Report, Table 8, this would result in a sliding resistance of about 0.6 ksf; for μ 2 equal to 1.0, the sliding force would be about 2 ksf. Looking at frictional materials in the dynamic case, the situation is more complex because of the cyclic nature of the earthquake forces. Tr. (Bartlett) 105312. When accelerations are acting downward there is an effective force acting upward and this decreases the normal stress and sliding initiates earlier than in the gravity case. Conversely, when the acceleration forces are acting upward, there is an effective force acting downward, this increases the normal force and sliding would initiate later than in the gravity case. Tr. at 10532.

423. To illustrate the difference between the Luk Model and the actual materials at

the interface at the PFS site, it is instructive to examine the μ2 value of 0.31 for a cohesive material such as cement-treated soil and for a frictional material at the interfaces under the pad. The shear resistance to sliding of the upper bond strength of the cement-treated soil at PFS is about 7.2 ksf. Tr. (Bartlett) at 11421-22. By contrast, the resistance to sliding in the Luk model, using frictional material, and using the mu2 value of 0.31, is about 0.6 ksf. Id. From this simple example, it is clear that the Luk model overestimates sliding by allowing sliding to occur at a much lower horizontal dynamic force than would occur in a model with cement-treated soil and cohesion at the interface

424. Also, clays act much differently than sand or other frictional material during an earthquake. As stresses develop in clay there is a linear relationship between stress and strain, expressed by shear modulus, until a peak is reached. Once reaching a peak, the clay has no more capacity, it yields, goes into post yield behavior, acts nonlinearly and deforms considerably. Tr. (Bartlett) at 10412. The soil model (*i.e.*, constitutive relation) used by Luk is unable to represent the post-yield behavior of the clay. Tr. (Bartlett) at 11482. Further, Dr. Luk recognized this issue but testified that he had not chosen to use a plastic model for soils. Tr. (Luk) at 11549.

425. The Board finds that the Luk Report does not model the design PFS intends to employ at the site; does not model the actual interface conditions at the PFS site; and by using a frictional material to represent the behavior of the two μ2 interfaces employs an inappropriate constitutive relationship to use in the numerical model.

426. In sum, the properties ascribed in the Luk Model at the interfaces designated as μ2 do not properly represent the strengths of those interfaces, and therefore, the Luk

Model overemphasizes sliding and this potentially could dampen out seismic energy that is delivered to the cask.

Young's Modulus

427. The Luk Report uses a 270,000 psi Young's modulus for the cement-treated soil under the pads. Table 4, Luk Report, NRC Exh. P.

428. None of the parties disagree that Holtec has constrained the Young's modulus of cement-treated soil to less than 75,000 psi. Moreover, meeting a Young's modulus of 75,000 psi is an integral part of PFS's soil cement testing that PFS has yet to conduct. *See e.g.,* PFS Exh. JJJ.

429. Dr. Luk testified he obtained the 270,000 psi from NRC Staff person, Mr. Mahandra Shah. Tr. (Luk) at 11625. There is nothing in the record to describe Mr. Shah's technical background, qualifications or experience. Counsel for the Staff represented that Mr. Shah conducted a literature review in which 270,000 psi was referenced for soil cement and he decided to use that value. Tr. at 11629.

430. The Board is puzzled why a value of 270,000 psi was used in the Luk model when the actual value is known for the cement-treated soil at the PFS site. Further, Young's modulus for cement-treated soil is not some obscure technical reference – it is part of a heated dispute between the State and PFS in Contention, Part C.

431. The question arises as to the effect of misrepresenting Young's modulus for cement-treated soil in the Luk Model. The State's expert testified that nonlinear models are extremely sensitive to input parameters and was unwilling to hazard a guess at the effect. Morever, the issues of changes of input parameters to nonlinear modeling is at the heart of

the dispute between the State and PFS in Holtec's cask stability analysis.

432. The Board finds that even though the correct value of Young's modulus was readily available, Dr. Luk used a value that differs three to four fold from that of cement-treated soil at the PFS site. As such, the Board finds this to be an additional error in the Luk model which adds uncertainty to the accuracy of its results.

Pacoima Dam earthquake time histories

433. Dr. Luk obtained the Pacoima Dam earthquake time histories, used in his model, form NRC Staff person, Mahandra Shah. Tr. (Luk) 6923. Dr. Luk, however, did not independently review the input time history. Id.

434. The acceleration time histories for Pacoima dam are 0.61 horizontal and 0.433 vertical. Tr. (Bartlett) at 10536.

435. Further, Dr. Luk testified that he does not know the location of the earthquake epicenter compared with the observation point or compared with the earthquake time history for PFS site. Tr. (Luk) 7005-09.

436. Significantly, the Pacoima dam time histories were not matched to a target spectrum and as such are not representative of an evaluation earthquake for the PFS site. Tr. (Bartlett) 11702. (SB)

437. For nonlinear analysis ASCE 4-98[62] recommend the use of multiple time histories. § 3.2.2.3 (2)(d) states: "In general, more than one set of acceleration time histories, meeting the requirements of Section 2.3, should be used, and the results of the analyses shall

---

[62]American Society of Civil Engineers, Seismic Analysis of Safety-Related Nuclear Structures and Commentary.

be averaged." State Exh. 118; *see also* ASCE 4-86 PFS Exh. XX. One of the requirements of ASCE Section 2.3 is that earthquake time histories shall be selected or developed so that they reasonable represent the duration of strong shaking conditions expected for the site. PFS Exh. XX.

438. The Board finds that the Pacoima dam earthquake record is not representative of the expected seismic conditions at the PFS site and does not satisfy the using multiple time histories as provided in ASCE 4-98.

439. The Licensing Board finds that the Luk cask stability analyses did not model the appropriate site conditions at the PFS site, thus, the Licensing Board finds the Luk cask stability analyses is not persuasive in demonstrating the HI-STORM 100 cask will not tip over under either a 2,000-year or 10,000-year earthquake at the PFS site.

440. Mr. Guttmann opines that the Luk report "[g]enerally shows that the Applicant's calculations are conservative." Tr. (Guttmann) at 6826. In response to whether " no stone has been left unturned," Mr. Guttmann testified that "as the state came up with concern, we reviewed every one of their concerns" and "addressed everyone of the state's concerns." Tr. (Guttmann) at 7062-64. Given that Mr. Guttmann a) admitted he had not reviewed the Holtec 2,000-year report (Tr. at 6844), b) is not familiar with the issues raised in this matter (Utah Contention L and L part B) and c) that we earlier found he has no expertise in modeling seismic behavior of free standing casks, the Licensing Board affords no weight to Mr. Guttmann's opinion concerning the Applicant's calculations and whether the State's concerns in this contention have been addressed.

<u>Comparison of the Holtec-Luk Results</u>.

441. The proponents of the Holtec and Luk models each claim that their model accurately evaluates the seismic response of casks at the PFS site at a 2,000-year or 10,000-year earthquake. Yet, Dr. Luk submits that the results from his analysis and Holtec's analyses "should not be directly compared." Tr. (Luk) at 6952. The Licensing Board recognizes the Luk and Holtec models are vastly different. Notwithstanding the vastly different nonlinear models, Holtec and Luk both purportedly modeled a HI-STORM 100 storage cask, on a 30 foot x 67 foot x 3 foot concrete storage pad, supported by cement-treated soil and dynamic soil properties developed by Geomatrix subject to single sets of time histories for a 2,000-year and 10,000-year earthquakes (initially developed by Geomatrix). Applicant's Exh. 86c at 13; Staff Exh. P at 7. The Licensing Board finds it difficult to reconcile that Holtec's and Luk's results cannot be directly compare yet both the Applicant and Staff both claim they confirm no cask tip over.

442. Holtec claims its Beyond Design Basis, case 1, confirms its DYNAMO results in its 2,000-year report. Applicant's Exh. 86c at 20-21. The Beyond Design Basis case 1, was an 8 cask simulation run on VisualNastran at a lower bound soil, coefficient of friction of 0.8 for a 2,000-year earthquake. Applicant's Exh. at 86d at 1. The VisualNastran showed a net displacement at the top of the cask of 3.7 inches with a maximum angle of rotation of 0.916 whereas DYNAMO generated a net displacement at the top of the cask of 3.08 inches with a maximum angle of rotation of 0.741. Applicant Exh. 86c at 20.

443. As described in its Additional Analyses (Applicant's Exh. 225), Holtec also ran another 8 cask simulation run on VisualNastran with a contact stiffness of 38,194,576 pounds per inch, 4.9 percent damping, and coefficient of friction of 0.8 for a 2,000-year earthquake.

Applicant's Exh. 225 at 18 (referencing 12). A contact stiffness of 464 x 10^6 pounds per inch and 5 percent damping was used in the DYNAMO runs where as a contact stiffness of 18.8 pounds per inch and 27.5 percent damping was used in Beyond Design Basis VisualNastran, case 1. Applicant's Exh. 86d at 2, A-1, Tr. (Soler) at 9672. The Additional Analyses, VisualNastran simulation produced cask # 1 results of 3.4 inches maximum excursion of top of cask. Applicant's Exh. 225 at 29. Notwithstanding the results for cask # 1, cask # 5 of the same simulation showed a 10.5 inch maximum excursion of the top of the cask. Dr. Soler testified that in the highly nonlinear calculation, "we cannot just say one set of results is double the other. When you change something, you have a factor." Tr. (Soler) at 11676. Contrary to Holtec's claims, while we recognize the behavior is nonlinear, the Licensing Board still finds inconsistencies in the various Holtec results. We further find that Holtec's additional simulations at a 10,000-year earthquake raise additional uncertainties and do not support the Holtec 2,000-year earthquake analyses.

444. The Luk analyses for a 10,000-year earthquake at a $\mu 1$ of 0.8, $\mu 2$ of 1.0 resulted in a maximum rotational angle of 1.16 degrees and maximum horizontal sliding of 7.2 inches and 7.39 inches in the u1 and u2 direction, respectively. Staff Exh. P at 32. In comparison, also for a 10,000-year earthquake with a coefficient of friction of 0.8, 5 percent damping, and the soil tuned to 5 hertz, Holtec predicted a maximum excursion of the top of the cask of 56 inches and a rotational angle of 5.37 degrees. Applicant's Exh. 86d at 13. Additionally, for a 10,000-year earthquake, in case 8, Holtec estimated a rotational angle of 10.13 degrees compared to Luk's maximum angle of rotation at 1.16 degrees. Applicant's Exh. 86d at 13, Staff Exh. P at 32. While the we are cognizant that Holtec and Luk

developed different models, the Licensing Board finds that the results generated by the Luk analyses provide no assurances that Holtec obtained accurate results.

Summary

445. The Licensing Board finds that barring actual test data to validate the results obtained by the cask vendors themselves, it is impossible to quantify the uncertainties in the nonlinear computer analyses. Additionally, based on our preceding findings that the Applicant has not met it burden that a) there is no engineering precedence or seismic performance data, b) the Applicant has not credibly demonstrated the dynamic properties and behavior of the storage pad, c), is ample evidence to suggest that the acceleration of the pads may be greater than that estimated by PFS d) there is insufficient evidence to show that all the pads will settle uniformly and e) Holtec analysis for a simple two pad system demonstrates that there can be significant forces transferred from pad-to-pad; do not substantially alter Holtec's nonlinear finite element cask stability results for both the 2,000-year and 10,000-year earthquakes, the Licensing Board further finds uncertainty in the calculated maximum angle of rotation for the VisualNastran simulations conducted for 10,000-year and 2,000-year earthquakes. Thus, the Licensing Board finds that there is not sufficient probative and relevant evidence to show that the Applicant has met its burden that the HI-STORM 100 casks will not tip over under a 2,000-year DBE or a 10,000-year earthquake at the PFS site.

E. Conclusions of Law

446. Based on the evidence presented, PFS has not met it burden of showing that free the standing HI-STORM 100 casks will not experience excessive sliding, uplift, collision, or tip over under design basis ground motions at the PFS site that the storage pads, the CTB,

their foundations systems, and the storage casks have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake. The Board concludes that PFS has not met the requirements of 10 CFR §§ 72.90, 72.102(c), 72.120(a) or 72.72.122(b).

CONTENTION PART E: Seismic Exemption Request

A. Issue: Has PFS shown by a preponderance of the evidence that there is sufficient conservatism built into PFS's ISFSI design such that its ISFSI design and subsequent consequences from a seismic event will not endanger life or property or the common defense and security and it is otherwise in the public interest to allow PFS a substantially lower design standard than mandated by the existing seismic regulations.

B. Regulations/Guidance

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

2. 10 CFR § 72.104(a). Normal operations and anticipated occurrences annual dose limit must not exceed 75 mrem.

3. 10 CFR § 106(b). Any individual located on or beyond the nearest boundary of the controlled area may not receive from any design basis accident a total effective dose equivalent of 5 rem.

4. DOE Standard 1020-02, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* (January 2002)

5. NUREG/CR 6738, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard-and-Risk-Consistent Ground Motion Spectra Guidelines*, October 2001 (2

volumes).

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

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

C. Findings of Fact

Overview

447. The difficulty facing this Board in evaluating PFS's request to use probabilistic earthquake ground motions with a 2,000-year return period value, equivalent to a 5 x 10^{-4} year mean annual probability of exceedance ("MAPE")[63], for the design basis earthquake at the PFS site is that earthquake science and engineering involve many uncertainties. Ultimately, this Board must decide what is an acceptable level of risk. Should such a decision take into account the operational life of the facility? Strictly annual risk? Does Dr. Cornell's assertion of conservatism in the ISFSI design approach assure a sufficient margin of safety? Has the Staff put forward a well-founded rationale for accepting a 2,000-year return period value with the PSHA methodology? These are some of the questions the Board must address in making its decision.

---

[63]A note is in order on terminology used in these findings. "Return period" is the average time between consecutive events of the same or greater severity and is sometime designated as MRP or mean return period. The annual probability of exceedance of an event is the reciprocal of the return period of that event. In terms of the DOE-STD-1020 paradigm, "performance goal" is used as the annual probability of exceedance of acceptable behavior limits (*i.e.*, behavior limits beyond which damage/failure is unacceptable). *See* Utah Exh. 208; DOE-STD-1020-2002, Table C-3, Staff Exh. QQ.

448. The intra-plate setting of the Central and Eastern United States ("CEUS"), east of the Rocky Mountains, involves very old geology, thick sediment cover, and low levels of seismicity, making it difficult to get a scientific understanding of the nature and cause of earthquakes in the CEUS. Tr. (Cornell) at 7891-92. There is a better understanding of earthquake occurrence along the tectonic plate margins in the Western United States, *i.e.*, along the Pacific coast, because earthquakes are more frequent and thus there are more data. Consequently, there is less uncertainty in the hazard assessments for these areas of the Western United States (near the plate boundaries) than in the CEUS. Tr. (Cornell) at 7892-93; *see also* Tr. (Arabasz) at 9138-39. However, in the Intermountain West, where the Skull Valley site is located, the situation is complex. Tr. (Cornell) at 7896; Tr. (Arabasz) at 9176-77.

449. The State offered expert testimony by a highly qualified seismologist, Dr. Walter Arabasz, on the seismic activity in the region of the PFS site. Since 1977, Dr. Arabasz, who is in charge of the University of Utah Seismograph Stations, has been studying and monitoring earthquakes in Utah and has made this the mainstay of his career. Tr. (Arabasz) at 9200. Dr. Arabasz testified that compared to the rate of earthquake activity on the plate boundaries in California, large active faults in the Intermountain area have relatively longer return periods. In the Salt Lake valley, the return period for a large surface-rupturing earthquake on the Wasatch fault is about 1,400 years whereas in Skull Valley the return period on the Stansbury fault is much longer. Tr. (Arabasz) at 9203. There is sparse information for large earthquake recurrence on the Stansbury fault (5 to 6 miles from the PFS site), the last earthquake having occurred on the order of 8,000 years ago and a prior event 15,000 or more years ago. What is known is that the Stansbury fault has been storing up energy for the past

8,000 years and is capable of delivering a large earthquake, but whether the next event will be tomorrow or thousands of years away is unknown. Tr. (Arabasz) at 9203-04. Armed with this knowledge, the Board is cognizant of the potential energy that may be unleashed at the Skull Valley site yet mindful of the uncertainties in earthquake forecasting. This being the case, the Board considers it prudent to be circumspect when evaluating the safety of the PFS facility.

450. There are two sides to the earthquake safety equation: (a) what is the capacity of the structures and foundations at the PFS site to withstand an earthquake; and (b) what is the demand or design basis earthquake that the capacity must meet. *See e.g.*, Cornell Tstmy, Post Tr. 7856 at 13-15; Arabasz Tstmy, Post Tr. 9098 at 6; Tr. (Arabasz) at 10047-48. For the demand side of the equation, at the current stage of NRC regulatory development on formulating the design basis earthquake for an ISFSI, there is no fixed reference frame for the failure probability of SSCs. By contrast to NRC, the U.S. Department of Energy ("DOE") framework has evolved to the point where DOE has ranked its facilities into four groupings and has established a probability of failure (termed P_F in DOE Table C-3, Staff Exh. QQ) of SSCs as a target performance goal for each of those groupings. Furthermore, ISFSI SSCs of concern at PFS (*e.g.*, casks, foundations composed, in part, of soil-cement) are atypical of those at nuclear power plants, for which there is a greater knowledge base. Tr. (McCann) at 8277. In this proceeding, PFS and the Staff are asking the Board to agree with them that the capacity side of the equation will do all the heavy lifting. Can the Board be confident that the asserted conservatism in design has indeed been achieved, given that the reference frame for ISFSI SSC failure probabilities is, at best, in a nascent state of development? The importance

of the exemption part of the contention (the demand side) is that the Board must determine what is the appropriate design basis earthquake ("DBE") to ensure an adequate margin of safety because sufficient protection depends on both the probability of occurrence of the seismic event as well as the level of conservatism incorporated into the SSC design.

Benchmark Probability for the DBE at the PFS Site

451. Part 72 currently requires PFS to assess the maximum vibratory ground motion that could be experienced at the PFS site using deterministic seismic hazard analysis methodology. Part 72[64] cross references the standard that formerly applied to nuclear power plants, *i.e.*, 10 CFR Part 100, Appendix A. Under the changes in the NPPs' requirement, codified at 10 CFR § 100.23, a NPP applicant now refers to NRC guidance (Reg. Guide 1.165) where the "reference probability" for determining the SSE from a probabilistic seismic hazard analysis is specified to be that probability which has an annual median probability of 1 x 10^{-5} of exceeding the SSE , which is equivalent to a mean annual exceedance probability of 1 x 10^{-4} (or a return period of 10,000 years) for the CEUS; there is the option that an applicant may request and justify the use of a higher reference probability for a site not in the CEUS (*e.g.*, in the western United States). Reg. Guide 1.165 at 1.165-12 (State Exh. 201); Tr. (Cornell) 8001-02.

452. As a starting point, the Board finds that it is reasonable to assume that the reference probability for a hypothetical NPP at the PFS site sets the upper benchmark for

---

[64]*See* 10 CFR § 72.102(b); *see also* § 72.102(f)(1): "For sites that have been evaluated under the criteria of appendix A of 10 CFR part 100, the DE [design earthquake] must be equivalent to the safe shutdown earthquake ("SSE") for a nuclear power plant."

establishing the DBE for the PFS facility.

453. As described in testimony by all parties, if a NPP were to be sited at the PFS site, acceptable design levels would be established using ground motions with a mean annual return period somewhere between 5,000 years and 10,000 years. Tr. (Arabasz) at 10111-14, 10120-24; Cornell Tstmy, Post Tr. 7856 at 42, 47-48; Tr. (Stamatakos) at 12717-18; (McCann) at 8337-38); Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 26-29.

454. The Staff has argued, with some qualification, that based on a survey of five NPPs in the Western United States ("WUS"), the reference probability for a hypothetical NPP at the PFS site would be a mean annual exceedance probability of 2×10^{-4} (5,000-year MRP). Tr. (McCann) at 8326, 8337-38; Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 26-29; Stamatakos Rebuttal, Post Tr. 12648 at 4-5. Two of the five NPPs in the survey are located in California, one is in Arizona, and two are in Washington state. State's Exh. 202.[65] We do not agree with Dr. McCann's assessment and with Dr. Stamatakos' position that the average MAPE of 2×10^{-4} from the five NPPs represents a value that is applicable to the entire WUS. Stamatakos Rebuttal, Post Tr. 12648 at 4-5. First, at least three of the five NPPs in the survey are located near tectonic plate boundaries along the Western coast, have steep hazard curves, and are not simply representative of the Intermountain area. Second, the Palo Verde site in Arizona, in an area of low seismicity and with a mean exceedance probability corresponding to a 26,000-year return period earthquake,[66] is not only an outlier in

[65]Excerpts, Topical Report YMP/TR-003-NP, *Preclosure Seismic Design Methodology for a Geologic Repository at Yucca Mountain*, Rev. 2 (8/97), DOE.

[66]Tr. (Cornell) at 8033;Tr. (Arabasz) at 9177-78, 10096.

the calculation of the sample mean but its MAPE argues against the applicability of a 5,000-year MAPE to the entire WUS. Third, we do not believe that extrapolating an average MAPE from such a small number of NPPs to the Skull Valley site – or to any other hypothetical NPP site in the WUS away from the plate boundary – would withstand critical scrutiny in a NPP licensing hearing. Representing that the sample mean characterizes "nuclear power plants in the Western United States" is defensible only semantically.

455. It is apparent that although the 5,000-year MRP may justifiably apply at WUS NPP sites where there are steep hazard curves, such as near tectonic plate boundaries, it does not necessarily apply in the Intermountain west. For example, DOE-STD-1020-2002 sets a greater probability of exceedance (*i.e.*, a shorter return period) for sites located near tectonic plate boundaries than other DOE sites. Staff Exh. QQ at Table C-3. For PC-4 facilities – equivalent to NPPs – the standard for sites located near tectonic plate boundaries is 2 x 10^{-4} (*i.e.*, a 5,000-year return period) whereas for non-plate tectonic sites the return period is 10,000 years.[67] Id.

456. Dr. Arabasz presented a qualitative analysis of nuclear facility sites in the WUS, including the Basin and Range province, and used a literature review and the steepness of hazard curves at some of those sites to ascertain whether the implied probability of exceeding a SSE corresponded to a 5,000-year MRP or a 10,000-year MRP. Dr. Arabasz is an expert with extensive professional experience in studying and monitoring earthquakes in the

[67]Dr. Cornell, *citing* DOE-STD-1020-94, Table C-3 at C-5, also noted the basis for the different risk reduction ratios for "Western sites" is that the western sites are near tectonic boundaries, where the hazard curves are considerably steeper. Cornell Tstmy Post Tr. 7856, at 16-17, n.5.

Basin and Range province, and the Board gives substantial deference to his analysis.

457. Dr. Arabasz first looked at available information. SECY-98-071, Staff Exh. S, documents the grant of an exemption to the Idaho National Engineering and Environmental Laboratory ("INEEL") to store Three Mile Island, Unit 2 ("TMI-2") fuel, including the following, at p. 2: "Based on 10 CFR 100.23 requirements, as described in Regulatory Guide 1.165, 'Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion,' a future nuclear power plant in the western United States can use as a safe shutdown earthquake the 10,000-year return period mean ground motion." Tr. (Arabasz) at 10093-94. Thus, in the foregoing document, issued in April 1998 – eight months after August 1997 when DOE published Yucca Mountain Topical Report YMP/TR-003-NP in which the average MAPE for five NPPs in the WUS was reported (State's Exh. 202, Table C-2) – the Staff accepted a 10,000-year MRP as an appropriate SSE reference standard for a NPP at the INEEL ISFSI site.

458. Another piece of information before Dr. Arabasz was DOE's effort to equate a design basis ground motion at Yucca Mountain to the SSE reference probability for a NPP. Even though it had calculated an average MAPE of about 2×10^{-4} (5,000-year MRP) for five NPPs in the WUS, as reported in its Yucca Mountain Topical Report YMP/TR-003-NP, Table C-2, State's Exh. 202, DOE chose not to use 5,000 years but 10,000 years as the MRP for the Yucca Mountain DBE. Tr. (Arabasz) at 10095; 10120-10121.

459. For more information relevant to an appropriate SSE reference probability eastward of the plate boundary into the Intermountain area, Dr. Arabasz turned to Kennedy & Short's paper titled <u>Basis for Seismic Provisions of DOE-STD-1020</u> (April 1994), in which

they give an overview of the slopes of the seismic hazard curves and show how they vary across the country. Kennedy & Short, Table A-2 (State Exh. 203), use a value, A_R, to describe "'the ratio of ground motions corresponding to a tenfold reduction in exceedance probability.'" Tr. (Arabasz) at 10099 (*quoting* Kennedy & Short at 2-1). In effect, the ratio is a measure of how much ground motions increase as the annual probability decreases. From seismic hazard curves at several nuclear sites, Kennedy & Short provide ratios for the probability intervals 1×10^{-5} to 1×10^{-4}, designated as A5/A4, and 1×10^{-4} to 1×10^{-3} (A4/A3). As can be seen from State Exh. 203, eastern sites tend to have the relatively highest ratios, high seismic sites near tectonic plate boundaries tend to have the relatively lowest values, and western sites not near tectonic plate boundaries tend to have intermediate ratios.[68] Armed with this information, Dr. Arabasz added to Kennedy & Short Table A-2 after determining the value of A5/A4 and A4/A3 for four of the five western sites of DOE Table C-2, State Exh. 202 (Diablo Canyon values were already determined by Kennedy & Short) and for the Yucca Mountain site. It is apparent that three of the five NPP sites on DOE Table C-2 (Diablo Canyon, San Onofre, and Washington Nuclear Plant 3 near Satsop) are near tectonic plate boundaries and have low ratios of A5/A4 (steep hazard curves) of about 1.5 or less; Palo Verde and Yucca Mountain have A_R ratios more like eastern sites. From comparing Kennedy and Short's Table A-2, State Exh. 203, with Table C-3 of DOE STD-1020-2002 (State Exh. 207), Dr. Arabasz concluded the following for A5/A4 ratios in the range of 1.5:

[68]The lower the ratio, the less relative change in ground motions when you move to the right on the hazard curve and hence the steeper the slope in log-log space. Tr. (Arabasz) at 10101-02.

> Under the DOE framework using Table C-3, one would achieve large risk reduction ratios that would justify the use of the 5,000-year P sub H value [probability of exceeding the seismic hazard]. When we have slopes of the order of 2 in A5/A4 space, for example, under western DOE sites not near tectonic plate boundaries, INEEL [sic, INEL][69], Los Alamos, Hanford, the assumption is that the engineering judgment was made as part of the DOE design approach that these A5/A4 slopes did not justify the 5,000-year return period motion.

Tr. (Arabasz) 10108; *see also* Tr. (Arabasz) at 10105-06.

460. Dr. Arabasz continued his qualitative analysis of non-coastal western sites by using as a proxy for NPP information a review of the 84th percentile deterministic motions for the INEEL, PFS, Yucca Mountain, and Los Alamos sites. He observed, qualitatively, that without exception the ground motion values approach or exceed 10,000 years. Tr. (Arabasz) 10109-14, 10120-24. Dr. Arabasz's presentation credibly shows that as you move eastward from the plate boundary to Hanford, Palo Verde, Yucca Mountain, INEEL, Los Alamos and the PFS site, the appropriate SSE reference probability for a NPP would not appropriately be pegged at 2×10^{-4} (5,000-year MRP) but rather at approximately 1×10^{-4} (10,000-year MRP).

461. Contrary to Dr. Arabasz's testimony, Dr. Stamatakos in his rebuttal testimony maintained that the average MAPE of 2×10^{-4} (5,000-year MRP) for five existing NPPs in the WUS "is applicable to the entire Western United States." Stamatakos Rebuttal Tstmy, Post Tr. 12648 at 4. Responding, in part, to his understanding of testimony by Dr. Arabasz (*see* Tr. (Stamatakos) at 12705-06), Dr. Stamatakos also asserted that "there is not a clear difference between the shapes or slopes of most hazard curves in the intermountain west (including the

---

[69] Quotations from the hearing transcript with obvious transcription errors are handled by putting the corrected wording in the quotation bracketed by sic plus the incorrect transcript wording.

PFS site), and sites that are near tectonic plate boundaries." Stamatakos Rebuttal Tstmy, Post Tr. 12648 at 5. He further explained, "My point is that when I look at the underlying factors that control the hazard curves I don't see a logical connection at least as a geologist necessarily between the shape or slope of the hazard curves and whether or not they are located right on a plate boundary or not." Tr. (Stamatakos) at 12705-06. It is clear to the Board that Dr. Arabasz did not imply that "only tectonic plate nuclear power plants may have a shorter return period than the 10,000 return period (MAPE = 1×10^{-4})" (Stamatakos Rebuttal Tstmy, Post Tr. 12648 at 4), or that somehow the shape or slope of a hazard curve is like a genetic marker that identifies whether a site is near or away from a tectonic plate boundary and, hence, whether the site qualifies or not for a higher seismic hazard exceedance probability (lower MRP) under DOE-STD-1020. *See* State Exh. 207 at Table C-3, footnote. The key point overlooked by Dr. Stamatakos is that under DOE guidance, the steepness of the hazard curve at a site, specified either in terms of a parameter A_R, the ratio of ground motions corresponding to a tenfold reduction in exceedance probability (Tr. (Arabasz) at 10098-99) or equivalently by a slope parameter k_H (Staff Exh. QQ at C-9), determines whether one achieves a risk reduction ratio of 20 or more that can justify, in the case of DOE PC-4 facilities, a 5,000-year reference ground motion versus a 10,000-year ground motion. Tr. (Arabasz) at 10105-06, 10108. In his prefiled testimony, Dr. Cornell quantitatively shows how the risk reduction ratio – a measure of the degree of conservatism inherent in design procedures and acceptance criteria – directly relates to the slope of a PSHA hazard curve. Cornell Tstmy, Post Tr. 7856 at Attachment A. The Board gives no weight to Dr. Stamatakos' rebuttal testimony. It either does not address or is inconsistent with the testimony by Dr. Arabasz and Dr. Cornell which shows that it is not the location of the site per se near a tectonic plate boundary; rather, it is the slope or steepness of the hazard curve that is the important factor in arriving at a risk reduction factor of 20 or more, which in turn is the rationale under DOE Standard 1020 to allow a 5,000-MRP.

462. Another point put forward by Dr. Stamatakos in his rebuttal testimony to justify a SSE reference probability of 2×10^{-4} (5,000-year MRP) for a hypothetical NPP at the PFS site is that the latter meets a "clear definition [in DOE-1020-2002 at C-9] of high hazard sites that fall in the 'near tectonic plate boundary' classification." Stamatakos Rebuttal Tstmy, Post Tr. 12648 at 5. Under cross-examination, Dr. Stamatakos strained to defend construing his "clear definition" but persisted in trying to link the PFS site to ones qualifying for a higher hazard exceedance probability (lower MRP) under DOE-STD-1020. Tr. at 12709-13. Dr. Stamatakos attempted to characterize the PFS site as comparable to sites near tectonic plate boundaries because of its proximity to active faults, high recurrence rates, and high seismicity. Stamatakos Rebuttal Tstmy, Post Tr. 12648 at 5; Tr. at 12712. Dr. Arabasz and Dr. Stamatakos disagree on whether the PFS site would be characterized as having "high seismicity" (Tr. (Stamatakos) at 12721-25) or nearby faults with "high recurrence rates" – or equivalently high slip rates. Tr. (Stamatakos) at 12724-25, 12727-39, 12753-56. The Board finds that at bottom, Dr. Stamatakos's subjective characterization of the PFS site from a geological point of view vis-a-vis DOE-STD-1020 is quixotic (*see* Tr. at (Arabasz) 10231-32); what is fundamentally important for the selection of appropriate seismic hazard exceedance probabilities in DOE-STD-1020 is the engineering analysis and judgment based on the slopes of hazard curves and consequent risk reduction ratios. Staff Exh. QQ at Appendix C. The

Board finds it significant that Dr. Cornell – who has quantitatively analyzed risk reduction ratios and the slope of the PSHA seismic hazard curve at the PFS site (Cornell Tstmy, Post Tr. 7856 at Att. A) – states in his prefiled testimony that "1×10^{-4} per annum, which has been found to be the mean estimate of the annual probability of exceedance of the design basis earthquake (DBE) of the typical nuclear power plant in all regions of the United States, is the appropriate basis from which to establish, via the principles of the risk-graded philosophy adopted by the Commission, the mean annual probability of exceedance of the DBE of an ISFSI anywhere in the country, including specifically at the PFSF site." Cornell Tstmy, Post Tr. 7856 at 48. Again, Dr. Stamatakos' rebuttal testimony does not withstand scrutiny and it does not overcome the evidence that the SSE reference probability for a hypothetical NPP at the PFS site is about 1×10^{-4} (10,000-year MRP).

463. The weight of the evidence presented in the hearing is that a technically defensible SSE for a NPP sited in the Intermountain area would have a return period of approximately 10,000 years and, therefore, the upper-end DBE benchmark for the PFS site should be a MAPE of 1×10^{-4}.

464. A number of regulatory codes are now using a 2,500-year return period as the basis for seismic design. For example, DOE-STD-1020-2002 uses a 2,500-year ground motion for the design of PC-3 facilities – those facilities similar to ISFSIs – not near tectonic plate boundaries. Staff Exh. QQ at Table C-3. Also, the International Building Code 2000 ("IBC 2000") is based on seismic hazard defined in terms of Maximum Considered Earthquake ground motions associated with a 2,500-year return period earthquake. Staff Exh. II at iii.

465. Under the IBC 2000, the building code currently in force in Utah, the design basis for certain buildings is a 2,500-year return period earthquake. According to the code, you first enter the hazard curve at 2,500 years and obtain the ground motions; then you multiply those ground motions by two thirds. An Importance Factor is used for certain structures, such as those that contain hazardous materials; in such cases you multiply ground motions obtained after the two-thirds reduction by 1.5 and this gets you back to the 2,500-year ground motions. Tr. (Cornell) at 7902-05.

466. Dr. Bartlett testified that interstate highway bridges in Utah are constructed using a 2,500-year design basis earthquake. Tr. (Bartlett) at 12807, 12977. Such structures must survive a 2,500-year event with essentially no structural damage. Id. at 12977.

467. It is evident that at the low end, the DBE benchmark for the PFS site sensibly must be at least 2,500 years. Currently, interstate highway bridges in Utah, certain buildings under the IBC 2000 building code, and PC-3 facilities under DOE-STD-1020-2002, all use a 2,500-year DBE. In addition to an inadequate margin of safety, there is a public policy concern that by allowing a 2,000-year DBE for the PFS nuclear facility it will have a lower DBE than that now required by other standards. At a minimum, setting the DBE for a nuclear facility lower than that for other non-nuclear structures or DOE PC-3 facilities poses a real public perception problem – as Dr. Arabasz testified: "[a]bsent the coterie [sic, codery] of the cognoscenti [sic, cognoscente], who can explain it..." Tr. (Arabasz) at 9208. Certainly the evidence presented in this proceeding does not justify a 5×10^{-4} MAPE that PFS has requested and the Staff has endorsed.

468. We further note that when the Commission granted consent to the Staff to go

forward with rulemaking, it mandated the Staff to seek comments and justification for a DBE in the range of 2,000-years to 10,000-years. *See* Staff Requirements Memo (November 19, 2001), Staff Exh. U. Therefore, to date, the Commission has not endorsed a 2,000-year DBE.

Staff's Rationale for PFS's Seismic Exemption

469. In general the Staff now relies on the following to support an exemption to 10 CFR § 72.102(f) and instead allow a 2,000-year MRP earthquake (5 x 10^{-4} MAPE) for the PFS site: (a) Commission statements that the radiological hazards posed by ISFSIs are less hazardous than those posed by NPPs; (b) the reference probability for the safe shutdown earthquake for a NPP being 1 x 10^{-4} MAPE and the average mean annual probability of exceeding the SSE at five existing NPPs in the WUS being 2 x 10^{-4}; (c) 2,000-year MRP in DOE-STD-1020-94 for PC-3 facilities; (d) grant of an exemption to INEEL; and (e) conservatism in PFS's probabilistic seismic hazardous analysis ("PSHA"). *See* Con-SER at 2-50 to 2-51; Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 18-21, and State Exh. 209.

470. The Staff has issued four documents that are illustrative of the Staff's logic aimed at justifying a 2,000-year MAPE for ISFSIs: the December 15, 1999 Safety Evaluation Report ("SER"); the September 29, 2000 SER; the March 2002 Consolidated SER; and the September 26, 2001 rationale for a 2,000-year MRP in Modified Rulemaking Plan, SECY-01-0178. Relevant excerpts from these documents are contained in State Exh. 209. The Staff has consistently relied on the MAPE for PC-3 facilities in DOE-STD-1020 and the grant of an exemption to INEEL among its time-varying justifications for PFS's exemption. Other parts of the rationale for the exemption have fallen by the wayside to be replaced by different justifications.

471. In the December 15, 1999 SER the Staff relied on a statement by PFS's contractor, Geomatrix, that a 2,000-year mean return period is appropriate for the design earthquake at the PFS site. The Board finds that reliance on the Applicant's exemption request does not justify approval thereof.

472. In the 1999 SER, the Staff also relied on an old version of the Uniform Building Code that recommended using peak ground motion values that have a 90-percent probability of not being exceed in 50 years for seismic design to analogize that peak ground motion values that have a 99 percent likelihood of not being exceeded in a 20 year licensed ISFSI would correspond to a 2,000-year MRP. This logic did not appear in the other iterations of the SER but it appears in a somewhat different form in the rationale in the Modified Rulemaking Plan, SECY-01-0178.

473. In the rationale of its Modified Rulemaking Plan the Staff analogized that a 20 year licensed ISFSI with 5×10^{-4} MAPE (2,000-year MRP) would have the same total probability of exceeding its design earthquake during its lifetime as would the Yucca Mountain pre-closure facility with a 1×10^{-4} MAPE (10,000-year MRP)[70] and an operational life of 100 years. State Exh. 209 (at 4). The Board finds that this logic has a different implication for the PFS ISFSI. If the Staff were to compare validly the 40 year operational life of the PFS ISFSI with the 100 year operational life of the Yucca Mountain pre-closure facility, then the total probability of exceeding the design earthquake at both facilities would be identical only if the DBE at the PFS ISFSI had a MAPE of approximately 2.5×10^{-4} (MRP

---

[70] *Cf* ISFSI: $5 \times 10^{-4} \times 20$ years $= 1 \times 10^{-2}$
with Yucca facility: $1 \times 10^{-4} \times 100$ years $= 1 \times 10^{-2}$

approximately 4,000 years). Arabasz Tstmy, Post Tr. 9098 at 14; Tr. (Arabasz) at 9204-09.

474. The Commission's statement that ISFSIs pose a lower radiological risk than NPPs does not in and of itself justify a five fold decrease from 10,000 years to 2,000 years in the MRP for an ISFSI's design ground motions. Moreover, the Board has already found that the Staff is on shaky ground in relying on a 5,000-year MRP for the SSE at a nuclear power plant site in the Intermountain West - as opposed to a western coastal site.

DOE Standard 1020

475. The Staff relies on DOE Standard 1020 both explicitly and implicitly to justify a 2,000-year return period for the DBE of the PFS ISFSI. Tr. (Arabasz) at 10143-45. However, the Staff disavows adopting DOE Standard 1020 (Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 30-31; Tr. (Arabasz) at 9270) and claims to rely on it only as a point of reference and for consideration of risk. Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 30-31; Stamatakos Rebuttal Tstmy, Post Tr. 12648 at 2. The key problem in the Staff's reliance on DOE Standard 1020 to justify a MAPE of 2×10^{-4} (2,000-year MRP) as specified in DOE-STD-94 for a Performance Category-3 facility, is that the Staff eschews the DOE design approach that fundamentally and quantitatively couples the MAPE for any performance category with a target seismic performance goal. Arabasz Tstmy, Post Tr. 9098 at 9-10; Tr. (Arabasz) at 9270; 10144-45. Unlike PFS, whose arguments at least attempt to justify the exemption request in terms parallel to DOE's risk reduction performance standard, the Staff distances itself from this basic part of the DOE paradigm. The Board finds that the Staff's partial reliance on DOE Standard 1020 does not offer support for an exemption to allow a 2,000-year MRP at the PFS site.

INEEL Exemption for TMI-2 ISFSI

476. The Staff also relies on the grant to DOE-INEEL of an exemption from 10 CFR § 72.104(f) for storage in an ISFSI of rubblized fuel debris from the Three Mile Island Unit II reactor (TMI-2 ISFSI). The facts and site conditions at INEEL are different from those at PFS. INEEL is located on federal reservation of vast size – approximately 800-900 square miles – and the nearest resident is tens of miles from the site. Tr. (Chen) at 8185, 8187-88. At INEEL, the TMI-2 ISFSI is located on the Idaho chemical processing plant ("IPCC") site. Ground motions at the IPCC site are 0.30 g for a 2,000-year MRP and 0.47 g for a 10,000-year MRP. State Exh. 127 at p. 4-1. The IPCC – a higher risk facility than the TMI-2 ISFSI – was designed to peak horizontal accelerations of 0.36 g. Id. The TMI-2 ISFSI was also designed to 0.36 g horizontal design value which means its ground motions fall somewhere between a 3,000- to 4,000-year MRP. Tr. (Chen) at 8184; Arabasz Tstmy, Post Tr. 9098 at 12. Fuel at INEEL is stored in 30 horizontal concrete modules, that under earthquake conditions are not expected to slide. Tr. (Chen) at 8186-87.

477. In contrast to the INEEL site, PFS is located within two miles of the nearest resident and the land to the north of the site is contiguous with privately owned land. FEIS at xxxviii (Staff Exh. E); Tr. (Donnell) 12578-81. Furthermore, the Board cannot rule out that someday the land to the north of the PFS site could be developed for residential uses. Tr. (Donnell) at 12579-82. PFS will not store 30 casks but 4,000 casks containing spent fuel from commercial nuclear power plants. The design values at PFS are those for a 2,000-year MRP. Further, PFS uses an unconventional design in which PFS and the Staff consider sliding of the casks and the pads under earthquake conditions to be beneficial because sliding

dissipates seismic energy that the casks and foundations would otherwise have to resist. Tr. (Pomerening) at 6634-35, 6652; (Soler) at 10658.

478. The Board finds that the site specific circumstances relating to DOE's 2,000-year exemption request for the INEEL TMI-2 ISFSI do not make that exemption a compelling precedent, and thus that exemption has little if any bearing in this case.

Geomatrix Probabalistic Seismic Hazard Analysis

479. The State acknowledges that the Geomatrix investigators who conducted a probabilistic seismic hazard analysis ("PSHA") for the PFS site, as contractors for the Applicant, are highly competent. Tr. (Arabasz) at 9322-23. Also, there is general agreement among the parties that Geomatrix conducted an adequate PSHA to depict the potential hazard at the PFS site. *See, e.g.*, Tr. (Arabasz) at 9119-20. The Staff, however, goes on to take the view that Geomatrix produced a "conservative" PSHA. Stamatakos, Chen, McCann, Post Tr. 8050 at 13-17; Tr. (Stamatakos) at 8113, 8220-21, 8225, 12763; Con-SER at 2-38 to -40, 2-48. PFS does not make this claim. The Staff's reliance on the conservative nature of Geomatrix's PSHA to support a grant of a 2,000-year MRP to PFS (Con-SER at 2-38 to -40) and its assertion that the Applicant's conservative estimate of hazard provides an additional margin of safety in the seismic design (Stamatakos, Chen, McCann, Post Tr. 8050 at 17) are founded on erroneous premises, questionable speculation about what the relative PSHA outcome should have been, and one-party analyses subject to scientific challenge.

480. A PSHA typically is an enormous undertaking involving seismic source characterization, ground motion modeling, and hazard calculations. Tr (Arabasz). at 9115-18, 9330. As such there is an incredible spectrum of parameters and values to be aggregated into

the process of calculating the hazard. Id. at 9878. Central to a well executed PSHA is capturing the technically supportable and legitimate range of informed opinion representing the whole scientific community on specific aspects of the PSHA. Id. at 9861-62.

481. The Staff did not conduct its own PSHA; it chiefly reviewed the geological and seismological inputs to Geomatrix's PSHA, evaluated Geomatrix's probabilistic and deterministic hazard results, and performed some independent analysis, notably slip tendency. Tr. (Stamatakos) at 8090-91; (Arabasz) 9861-62; Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 12-18; Con-SER at 2-35. In order to buttress its claim that the Geomatrix PSHA is conservative, the Staff uses the slip tendency analysis conducted by Dr. Stamatakos and his colleagues at Southwest Research Institute (Tr. (Stamatakos) at 8089) and also makes comparisons to PSHA results for sites in and around Salt Lake City. Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 16-17. Scrutiny of the Staff's analysis and its PSHA comparisons does not substantiate the Staff's claim that Geomatrix's PSHA results are conservative.

482. Slip tendency analysis is a modeling technique designed to assess stress states and potential fault activity.[71] As used by the Staff, *i.e.*, for the purpose of assessing potential fault activity, the analysis requires as a starting point a specification of the orientation and relative magnitudes of stresses acting on the local geology of Skull Valley. As the Staff explains in its prefiled testimony:

In slip tendency analysis, the underlying assumption is that the regional stress

[71] *See e.g.*, Morris et al., 1996, cited in Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 14.

> state controls slip tendency and that there are no significant deviations due to local perturbations of the stress conditions. The assumption is supported by a similar slip tendency analysis of the Wasatch fault, which shows the highest slip tendency values for the segments of the fault considered to be most active (Machette et al., 1991). . . . [The] orientation for the principal stresses was based on recent global positioning satellite information (Martinez et al., 1998a).

Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 14. Because the stress state at Skull Valley is unknown, the Staff had to assume the applicability of regional stress information from elsewhere. The Staff reported that it used a horizontal minimum principal stress with an azimuth of 085°, citing Martinez et al., 1998. <u>Id.</u>; Tr. (Stamatakos) at 8087. But the cited Martinez paper (State Exh. 184) does not contain this value; rather, the Staff arrived at this value by subjectively "tuning" the regional stress field in the Wasatch Front area to get maximum slip tendency on parts of faults with known paleoseismic (prehistoric) slip like the Wasatch fault. Tr. (Stamatakos) at 8091-92, 8191.

483. Based on the results of its tuned slip tendency analysis, the Staff argues that the East fault has a relatively low slip tendency value and is therefore less likely to slip than faults or fault segments further from the site. Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 15. This conclusion ignores the Staff's acknowledgment of Geomatrix's finding that, "In all the alternative models <u>and because of the evidence for surface rupture of late Quaternary deposits</u>, the East fault is considered seismogenic and assigned a probability of activity of 1." Staff Exh. Q at 2-17 (*emphasis added*). Based on offsets of those late Quaternary deposits in the immediate vicinity of the PFS site, Geomatrix assessed for the East fault a most likely slip rate of 0.2 mm per year – the same order of magnitude as the most likely slip rate of 0.4 mm per year for the Stansbury fault. State Exh. 185 at Table 6-2; *see, e.g.*, Con-SER

reference, Geomatrix Consultants, Inc. 1999a at 48-49. The Board finds that the evidence of surface rupture of late Quaternary deposits by the East fault is a far more cogent indicator of the fault's seismogenic potential and of the local stress conditions near the PFS site in Skull Valley than what the Staff guesses them to be from its hypothetical, subjectively tuned modeling. At best, the Staff's interpretation of the stress state in Skull Valley would be one competing opinion in a PSHA, subject to challenge by other experts. *See* Tr. (Arabasz) at 9862. Further, corresponding inferences the Staff makes from the slip tendency analysis about conservatism in Geomatrix's assessed site-to-source distances and maximum magnitudes (Con-SER at 2-38; Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 15-16) are also arguable and not established conclusions.

484. The seismogenic potential of the West fault is another basis on which the Staff argues that Geomatrix's PSHA is conservative because the Staff concludes the West fault is a splay of the larger East fault, incapable of independently generating large magnitude earthquakes. Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 13; Tr. (Stamatakos) at 8222; (Arabasz) 9842-43; Con-SER at 2-33. The Board finds that this argument is inconsequential insofar as the Staff acknowledges that alternative models for the geometry and extent of the West fault have little effect on the total hazard, and that the West fault – whether as an independent source or a secondary feature – has a minimal influence on the hazard computed by Geomatrix. Con-SER at 2-46; *see also* State Exh. 185 at Fig. 6-12.

485. Another major line of reasoning the Staff uses to conclude the Geomatrix PSHA is conservative is a comparison to PSHA results in and around Salt Lake City, which leads the Staff to claim that Geomatrix's PSHA may have led to an "overly conservative"

hazard result by as much as 50% or more. Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 13, 16-17. An erroneous premise pervading these comparisons by the Staff is that "fault sources near Salt Lake City are larger and more seismically active than fault sources near the PFS site." Id. at 16; *see also*, for example, Tr. (Stamatakos) at 8112, 8115, 8221-22; (McCann) 8225. In pre-filed testimony Dr. Stamatakos claimed that the Wasatch fault "has a slip rate nearly ten times greater than the Stansbury or East Faults (cf. Martinez et al., 1998; Geomatrix Consultants, Inc., 1999a), and is capable of producing significantly larger magnitude earthquakes than the faults near the proposed PFS Facility site in Skull Valley (cf. Machette et al., 1991; Geomatrix Consultants, Inc., 1999a)." Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 17; Tr. (Arabasz) at 9865.

486. The Stansbury and East faults are the two largest contributors to the total mean hazard at the PFS site for return periods greater than a few hundred years (State Exh. 185 at Fig. 6-12) and have the highest-weighted slip rates of 0.4 mm/yr and 0.2 mm/yr, respectively. Id. at Table 6-2; Tr. at (Stamatakos) 8234, 9876. The mean maximum magnitudes assessed by Geomatrix for these two faults are 7.0 and 6.5, respectively. Con-SER at 2-47; Tr. (Stamatakos) at 8238. The corresponding geological slip rate assessed by Geomatrix for the Salt Lake City segment of the Wasatch fault is 1.1 mm/yr (State Exh. 185 at Table 6-2; Tr. (Stamatakos) at 8234-35; (Arabasz) 9877), and the maximum magnitude for this segment of the Wasatch fault ranges from about 6.7 to 7.1 (State Exh. 185 at Fig. 6.2; Tr. (Stamatakos) at 8240). During testimony Dr. Stamatakos admitted that the Geomatrix data indicate the slip rate on the Wasatch fault is "roughly a factor of three" – not ten – greater than the slip rate on the Stansbury fault. Tr. (Stamatakos) at 8235-36. According to Dr.

Stamatakos, the factor of ten comes from adopting slip rates "up to five millimeters per year" for the Wasatch fault based on global positioning system ("GPS") data from Martinez et al., 1998. Tr. (Stamatakos) at 8236; State Exh. 184. However, he also admitted, "Well, certainly there is some controversy in the scientific community about how you actually interpret the GPS slip rates." Tr. (Stamatakos) at 8237. Attempting to compare slip rates mixing GPS information and conventional geological information is a dubious proposition. Tr. (Arabasz) at 10130-31. In the case at hand, relying on the paper by Martinez et al., 1998, to argue that the Wasatch fault has a slip rate of about 5 mm/yr is not scientifically defensible. Quoting directly from that paper, Dr. Arabasz established first, that the authors themselves are uncertain whether the GPS deformation field they observed is due to loading of the Wasatch fault or to some other cause, including homogeneous crustal extension (Tr. (Arabasz) at 10129-30), and, second, that the authors' uncertainty arises from "a lack of broader GPS coverage and the limitations of the current resolution of the GPS measurements." Id. at 10130.

487. The Staff makes two basic comparisons between Geomatrix's PSHA results and the counterpart hazard results for sites in or near Salt Lake City. First, it notes that, "[T]he results of the Applicant's PSHA for Skull Valley (Geomatrix Consultants, Inc., 2001a) suggest that it is 1.5 times more likely that a ground motion of 0.5g horizontal peak ground acceleration or greater will be exceeded at the PFS site (assuming hard rock site conditions), than at Salt Lake City, based on the USGS National Earthquake Hazard Reduction Program (Frankel et al., 1997)." Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 16; *see also* Staff Exh. JJ at 5. Dr. Arabasz made the point in his testimony that there are significant

shortcomings in this comparison by the Staff. Tr. (Arabasz) at 9864-65. The following facts are relevant to the comparison. The exact location of the Salt Lake City PSHA calculation is uncertain. Tr. (Stamatakos) at 8215-16. The hazard calculation for Salt Lake City is based on the USGS National Earthquake Hazard Reduction Program (id. at 8109), whose hazard calculations would not be acceptable for the SAR at the PFS site. Id. at 8111. Although not explicitly acknowledged by Dr. Stamatakos, the reason for the latter is that the national hazard mapping is done on a regional scale and includes only major active faults. Id. at 8110. Dr. Stamatakos did not know "everything the GS did in this [the Salt Lake City] analysis," but presumed that "the Wasatch fault probably controls a lot of what is in that hazard." Id. at 8110-11. In Geomatrix's site-specific PSHA for the PFS site, the East fault is only 0.9 km from the Canister Transfer Building, has a mean maximum magnitude of 6.5, and is a major contributor together with the Stansbury and East Cedar Mountain faults to the total mean hazard; all three faults are within 9 km of the PFS site. Con-SER at 2-47; *see also* Tr. (McCann) at 8232. Given the slip rates of 0.4 mm/yr, 0.2 mm/yr, and 0.07 mm/yr for the Stansbury, East, and East Cedar Mountain faults, respectively (State Exh. 185 at Table 6-2), there is a combined slip rate of 0.67 mm/yr contributing to the annual earthquake activity rate, which is 74% of the Wasatch fault's slip rate of 1.1 mm/yr. Slip rates, maximum magnitudes, distances, and near-source effects are all part of the complex interplay of parameters in the Geomatrix PSHA. Tr. (Arabasz) 9878-79. The site-specific Geomatrix PSHA hazard results at the PFS site (for rock site conditions) are an integrated outcome of the seismic source characterization, just as the USGS's regional PSHS hazard results are at Salt Lake City (also for rock site conditions). The Board find, as presented in this proceeding, that the Staff's claimed conservatism cannot be evaluated by comparing the two bottom lines. Without independently performing site-specific PSHAs for the two sites, the Staff's inference that the Geomatrix PSHA is conservative by comparison to sites in or near Salt Lake City is only speculation.

488. The second comparison the Staff makes between Geomatrix's PSHA results and counterpart results near Salt Lake City relates to hazard calculations at nine sites in the I-15 corridor in the Salt Lake Valley. Stamatakos/Chen/McCann Tstmy, Post Tr. 8050 at 17. The Staff observes that Geomatrix's 2,000-year horizontal peak ground acceleration (soil hazard) is actually higher than the 2,500-year ground motions (also on soil) at the I-15 sites. Id. The Staff explicitly reviewed Geomatrix's revised ground motion modeling in 2001, which involved development of a detailed shear-wave soil profile to calculate site response, and noted: "This change in the shear-wave profile and site response model led to a significant increase in estimated ground motions at the PFS site." Con-SER at 2-41. In fact, the 2,000-year peak horizontal ground motion increased 35% from 0.528g to 0.711g. Id. at Table 2-2. This misleading comparison is easily dealt with: without stripping off the site responses at the PFS and I-15 sites, the Staff's comparison of PSHA results is meaningless.

489. A proposition raised by Dr. McCann to support the conservative nature of Geomatrix's hazard results is that at very low ground motions the hazard curves at the PFS site have the same annual frequencies of exceedance as the hazard curve for Salt Lake City. Tr. (McCann) at 8224-25; Staff Exh. JJ at 5. The Board finds Dr. McCann's proposition is misleading because to validly compare very low ground motions between the PFS and Salt Lake City hazard curves, one has to examine and compare the methodology used by

Geomatrix in its site-specific PSHA with that used by the USGS in its regional PSHA – specifically, how background seismicity for the lowest magnitudes considered was analyzed and areally smoothed. Dr. McCann also observed that the rate of occurrence of earthquakes exceeding low ground motions at Skull Valley and Salt Lake City comes very close to that for the San Francisco Bay Bridge hazard curve (Staff Exh. JJ at 5) and thus, "[T]he Skull Valley site appears to be challenging some of the more seismically active areas in the country . . ." Tr. (McCann) at 8225. Again, comparison of these hazard curves at the lowest ground motions cannot be done validly without scrutinizing the respective methodologies used. As a check, however, if one grants the similarity of the Skull Valley and Salt Lake City hazard curves at low ground motions (Staff Exh. JJ at 5), one can examine the mean hazard curves computed in a uniform way by the USGS for Salt Lake City and San Francisco, as presented in Figure 3 on page 4 of Staff Exh. JJ.[72] In this figure, for 0.1g peak acceleration, Salt Lake City has approximately the same frequency of exceedance relative to San Francisco as it does compared to the San Francisco Bay Bridge in the figure on page 5 of the same exhibit. Thus, one would conclude, according to Dr. McCann's reasoning, that both Salt Lake City and Skull Valley are "challenging" San Francisco's hazard. The Board finds that the Staff's testimony does not support its claim that Geomatrix's hazard results are conservative.

490. After lengthy testimony and cross-examination, Dr. Stamatakos still holds to the view that Geomatrix provided a conservative seismic hazard estimate for the PFS site. Tr. at (Stamatakos) 12763. Dr. Arabasz, on the other hand, agreed to the adequacy of

---

[72]Note the different appearance of log-log plot compared to Staff's log-linear plot on page 5 therein.

Geomatrix's PSHA (Tr. at (Arabasz) 9119) but would not agree that its hazard results were conservative (e.g., id. at 9861-63, 9878-79, 10128-31).

491. Dr. Stamatakos's bottom-line position is that either Geomatrix provided a very conservative seismic hazard curve or, if the hazard results are accurate, the PFS site deserves to be treated as a tectonic plate boundary site, which would justify a higher reference exceedance probability (lower MRP). Tr. (Stamatakos) at 12753-54, 12763. This appears to be a false dilemma. We have already found that the first part of the proposition is opposed by evidence that Geomatrix's PFSA is not conservative, and the second part is opposed by evidence that the benchmark probability for the DBE at the PFS site is not 2×10^{-4}. The Board gives considerable weight to Dr. Arabasz's view that the large predicted ground motions at the PFS site are due to the unusual closeness of the East fault and the controlling earthquakes (Tr. (Arabasz) at 10228-29), and the hazard at the site would not justify 5,000-year SSE ground motions in the case of a hypothetical nuclear power plant (Tr. 10113-14).

492. We are faced here with two competing opinions: one by Dr. Arabasz and the other by Dr. Stamatakos. On balance we give greater weight to Dr. Arabasz's testimony. We make this judgment based on the depth of Dr. Arabasz's familiarity and experience with earthquake conditions in Utah, seismology, and seismic hazard analysis. We note Dr. Stamatakos's training and professional experience in structural geology and geophysics, and his involvement in multidisciplinary studies at the Center for Nuclear Waste Regulatory Analysis. While his insights and views have merited careful attention, the many substantive challenges to his arguments appear to reflect a lesser degree of experience with the broad scope of PSHA issues that are central here.

493. Whether or not the Applicant has produced a DSHA that fully meets the requirements of 10 CFR 100 Appendix A is a residual issue. Tr. (Arabasz) at 9152-53. Assuming the allowance of a PSHA, the issue has been set aside by stipulation and is not a problem for hearing unless PFS or the Staff attempt to use DSHA results to validate some level of conservatism in the PSHA results. Id. at 9152-54.

494. The Board finds that the Applicant's PSHA is adequate. The Board does not find sufficiently convincing evidence to support the Staff's claim that the Applicant's PSHA hazard results are conservative or overly conservative. In sum, the evidence does not support a finding that the Staff may rely on claimed conservatism in the Geomatrix PSHA or a 5,000-year benchmark probability as rationale for PFS's 2,000-year DBE exemption request.

Establishing Risk Graded Design Basis Earthquake Standard

Performance Goals

495. A typical risk-graded approach to seismic design utilizes a DBE defined at some mean annual probability of exceedance ("MAPE") and a set of design procedures and acceptance criteria. Cornell Tstmy, Post Tr. 7856 at 13-14. Both the State and PFS agree that the DBE must be fundamentally coupled with the design conservatisms and acceptance criteria within some paradigm of acceptable risk. Tr. (Arabasz) at 9121, 9127; Cornell Tstmy, Post Tr. 7856 at 14 (*stating* "important" that both must be considered); Bartlett Tstmy (Part E), Post Tr. 12776 at 6; Tr. (Bartlett at 12805). The design procedures and acceptance criteria include conservatisms that are intended to implement "performance goals (*e.g.*, target levels of the seismic failure probability for the [facility or structure]), which are defined in a manner reflecting the anticipated consequences of the failure." Cornell Tstmy, Post Tr. 7856 at 14.

496. Performance goals may be defined in terms of a permissible annual probability of unacceptable performance. Bartlett Tstmy (Part E), Post Tr. 12776 at 5. Performance goals are established to "assure safe and reliable performance" during an earthquake. DOE-STD-1020-94 at C-1 (PFS Exh. DDD). The required degree of the conservatism incorporated into the acceptance design criteria is referred to as risk goals or "risk reduction ratio." Bartlett Tstmy (Part E), Post Tr. 12776 at 5.

497. The desired level of seismic safety can be achieved by adjusting either the DBE or the level of conservatism of the design procedures and acceptance criteria, or both. Cornell Tstmy, Post Tr. 7856 at 14. Significantly, in response to Judge Lam's concern that there are "substantial uncertainties associated with any probabilistic assessment," Dr. Cornell testified that in computing the failure probability of SSCs, uncertainties must be factored into any estimates of safety margins. Tr. (Cornell) at 7919-7920.

DOE Risk Graded Seismic Design Methodology

498. DOE has formally adopted the seismic design and analysis methodology of balancing the DBE and design procedures. DOE-STD-1020-02 at iii-v (Staff Exh. II); Bartlett Tstmy (Part E), Post Tr. 12776 at 4-5; Arabasz Tstmy, Post Tr. 9098 at 9; Cornell Tstmy, Post Tr. 7856 at 15-16. DOE's procedures aide in illustrating a risk graded approach to the seismic design of nuclear facilities. DOE Standard 1020 is a reasonable framework to apply a risk graded approach to nuclear facilities. Tr. (Bartlett) at 12804. Importantly, DOE fundamentally couples the MAPE with performance goals. Tr. (McCann) at 8140.

499. In DOE Standard 1020, DOE prescribes its seismic design criteria for non-nuclear and nuclear facilities, including dry spent fuel storage facilities. Bartlett Tstmy (Part

E), Post Tr. 12776 at 8. Under the DOE 1020 framework, SSCs are classified in one of four performance categories. DOE-STD-1020-02 at C-2 (Staff Exh. QQ); Bartlett Tstmy (Part E), Post Tr. 12776 at 4-5; Cornell Tstmy, Post Tr. 7856 at 15. The PFS facility would be classified as a performance category 3 ("PC-3") facility. Tr. (Arabasz) at 9125-26, (Cornell) at 15, 30-31.

500. The next step under the DOE paradigm is to determine the appropriate target performance goal for each type of SSC. DOE-STD-1020-94 at C-1, -2 (PFS Exh. DDD). DOE mandates that seismically induced unacceptable performance should have an annual probability less than or equal to the established performance goal. Bartlett Tstmy (Part E), Post Tr. 12776 at 5. The probability of unacceptable performance is expressed in terms of a mean annual probability of exceedance or MAPE. Id. at 8.

501. DOE refers to the conservatism in the design procedures and acceptance criteria as the risk reduction ratio. Id. at 5; Cornell Tstmy, Post Tr. 7856 at 16. The risk reduction ratio is defined as $R_R = P_H/P_F$, where P_H is the mean annual seismic hazard exceedance probability and P_F is the permissible annual probability of unacceptable performance. Bartlett Tstmy (Part E), Post Tr. 12776 at 5. DOE Standard 1020-02 imposes a minimum risk reduction ratio of 4 for a performance category 3 ("PC3") facility. Id. at 9.

502. DOE controls the level of conservatism in SSCs through deterministic acceptance criteria to achieve specific risk reduction ratio or R_R levels. Bartlett Tstmy (Part E), Post Tr. 12776 at 10; Cornell Tstmy, Post Tr. 7856 at 18; DOE-STD-1020-94 at 1-5 (PFS Exh. DDD). As discussed in more detail at the beginning of this section, to determine if the established acceptance criteria have been met with an acceptable factor of safety, the

"capacity" of the SSC to resist an earthquake is calculated and must be greater than the demand or earthquake loading. A safety factor is a function of the capacity divided by the demand. Tr. (Bartlett) at 12823, 12834.

503. DOE acknowledges that specific acceptance criteria for overturning and sliding of foundations have not been developed. Bartlett Tstmy (Part E), Post Tr. 12776 at 12; Tr. (Bartlett) 12813; Cornell Tstmy, Post Tr. 7856 at 24 (*stating* "it is not entirely clear whether the R_R range conclusion . . . was intended to apply to foundations"). "[R]isk reduction factors are really deterministically done, and there are extra conservatisms and margins inherent in structural mechanical codes, which generally don't apply to foundation systems." Tr. (Bartlett) at 12812. DOE mandates that for some system components for overturning or sliding of foundations "there should be less than 10 percent probability of unacceptable performance at input ground motion defined by scale factor [SF] of 1.5SF times DE." Bartlett Tstmy (Part E), Post Tr. 12776 at 12 (*quoting* DOE-STD-1020-94 at 2-24).

504. Notably, the methodology and standards set forth in DOE Standard 1020 withstood the scrutiny of extensive technical peer review. Id. at 8. Importantly, none of the parties disagreed that abiding by DOE's seismic design methodology would provide a sufficient approach in this case.[73], [74] Tr. (Cornell) at 8017; (Arabasz) at 9127 (*agreeing* to the extent the entire methodology is adopted, not just selectively picking the DBE).

---

[73]Dr. Cornell references a DOE failure probability standard of 10^{-4} for performance category 3 facilities. Tr. (Cornell) at 8017-18.

[74]Curiously, although claiming no reliance on DOE Standard 1020 to justify its 2,000-year DBE (Tr. (McCann) at 8136), the Staff claims it had the "benefit of mature guidance as it had developed over in the DOE sector." Tr. (McCann) at 8138.

505. The Licensing Board finds that at this time NRC Staff's approach to evaluating the performance of ISFSI SSCs is ad hoc and has not evolved to the level of sophistication and technical rigor required by DOE. Tr. (Arabasz) at 9160-61; *see also* Tr. (McCann) at 8138. Although PFS is not subject to DOE standards, the Licensing Board finds that the methodology dictated in DOE-Standard-1020 is rationally developed and highly persuasive in this case.

506. To reasonably assure that activities associated with the PFS storage facility can be conducted without endangering the health and safety of the public, the Licensing Board finds that the DBE must be formally linked to a specific performance goal and risk reduction ratio. The Licensing Board further finds that the Staff has not established a performance goal (failure probability) for this facility or any previous ISFSIs.[75] Tr. (McCann) at 8140. As a result, the Staff has not coupled a performance goal and risk reduction ratio to the 2,000-year DBE for the PFS facility. In fact, the Staff admitted it did not explicitly address the DBE and performance of SCCs in concert. Id. at 8143.

Evidence of SSCs Probability of Failure or Risk Reduction Ratios.

Fragility Curves

507. A fragility curve describes the design margin and the variability of the design margin (Tr. (Cornell) at 8020) as a function of the amplitude of strong ground motion (Tr.

[75]NRC NUREG/CR 6728, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines* (October 2001), describes the desirability of setting a probability or mean annual frequency of earthquake-caused failure as a key issue for achieving risk-consistent design guidelines. Tr. (Arabasz) at 10048; *see also* Tr. 10146.

Bartlett Tstmy (Part E), Post Tr. 12776 at 11)[76]. The fragility curve in combination with the seismic hazard curve provides the probability of failure for the SSC for a range of ground motions. Id. at 9. Importantly, fragility curves would provide this Licensing Board better assurance in setting and evaluating the lower DBE sought by PFS because those curves allow as precise as possible an estimate of the actual seismic design margin and its variability (Tr. (Cornell) at 8020) for the range of ground motions (Bartlett Tstmy (Part E), Post Tr. 12776 at 9). PFS did not develop fragility curves for the proposed PFS ISFSI. Tr. (Cornell) at 8003.

508. Because of PFS's unprecedented and untested design, there is no existing data to demonstrate the seismic performance of its SSCs. Fragility curves are an acceptable method to predict the seismic performance or the annual probability of unacceptable performance for the SSCs. Bartlett Tstmy (Part E), Post Tr. 12776 at 6.

509. Although proffering differing importance to the development of fragility curves in this case, no party disagreed that, absent brittle behavior in the system, the Applicant could still demonstrate an acceptable probability of failure absent fragility curves if the SSCs are shown to meet the established performance goal and risk reduction factors at the specified DBE. *See e.g.*, Tr. (Bartlett) at 12852-53; Tr. (Cornell) at 8020. Thus, notwithstanding the comfort level that fragility curves would give us, this Licensing Board is reluctant to find that a properly justified DBE mandates fragility curves. The Licensing Board now turns to examine whether the Applicant has shown its SSCs reasonably meet performance goals and risk reduction factors.

---

[76]A fragility curve would be developed as part of a PRA. Tr. (Cornell) at 8020.

Probability of Failure

510. One can demonstrate it meets the performance goal by showing that the probability of failure is less than the specified performance goal or that the SCC is "estimated not likely to fail under a ground motion with an annual probability of exceedance that is less than the performance goal." Cornell Tstmy, Post Tr. 7856 at 38-39.

511. The State claims that the annual probability of exceedance cannot be determined because of unresolved uncertainty in the Applicant's analyses. Tr. (Bartlett) at 12871. *See* Contention Part D *supra*, wherein the State claims that the Applicant has not met its burden in demonstrating adequate seismic design margins exist for a 2,000-year DBE due to various omissions, unconservative assumptions, and errors in the PFS analyses. Any findings against PFS on its claimed conservatism in Part D are weighed here because in his testimony Dr. Cornell relies, in part, on the PFS Part D witnesses concerning the probability of failure and conservatism in the Applicant's design.

Storage Casks.

512. The Applicant has not conducted a probabilistic risk assessment to evaluate any design margins or the consequences of casks tipping over. Tr. (Cornell) at 7923. Additionally, there are no fragility curves for the HI-STORM 100 casks at the PFS site. Cornell Tstmy, Post Tr. 7856 at 35; Tr. at 8003. Dr. Cornell's opinion that the storage cask will achieve a performance goal of $1x10^{-4}$ is based on the cask vendor's unanalyzed prediction of what will occur from strong ground motions generated by a 10,000-year return period earthquake; the Staff's assessment that "no sliding impact between the casks" will occur under a 10,000-year ground motion; and Holtec's tipover analysis. Cornell Tstmy, Post Tr. 7856 at

39.

513. Although Dr. Cornell relies on Holtec's 10,000-year prediction of no cask tipover, he did not "believe" he reviewed the Holtec's *Beyond Design Basis Scoping Analyses*, Rev. 1 (April 19, 2002).[77] Tr. (Cornell) at 7986. Moreover, contrary to his opinion that uncertainties must be factored into estimates of safety margins, Dr. Cornell did not quantify the uncertainties in the cask vendor's nonlinear finite element cask stability analysis. Tr. (Cornell) at 7973. Dr. Cornell claims quantification of uncertainties was not necessary because the major source of uncertainty is nonlinear behavior and Holtec performed a nonlinear analysis. Id. The fact that Dr. Cornell opines that quantification of uncertainty in Holtec's nonlinear cask stability analysis is unnecessary because Holtec conducted a nonlinear analysis is too tenuous a connection to show that PFS has met its burden of showing conservatism in SSCs at PFS.

514. With respect to potential effects of cask tipover or drop, apparently oral discussions with the cask vendor were the sole basis to support Dr. Cornell's opinion because he could not recall reviewing Holtec's drop/tipover analysis entitled *PFS Site Specific HI-STORM Drop/Tipover Analyses*, HI-2012653, Rev. 2 (October 31, 2001). Tr. (Cornell) at 7975-76.

515. Holtec's conclusions that the canister would not be breached are dependent upon its assumption that the angular velocity of a tipping cask is zero. *See* Contention, Part D, Cask Stability. If the casks in fact tipped over during a seismic event, Dr. Cornell further

[77]Marked as Applicant Exh. 86A; Rev. 2, dated June 3, 2002 was admitted as Applicant Exh. 86C.

opined that "[t]he initial [angular] velocity [of the tipping cask] would probably clearly have to be something greater than zero or it would not be moving in that direction." Tr. (Cornell) at 7978; *see also* Tr. (Bartlett) at 12870-71; 12913-12917. The Board finds that Dr. Cornell's testimony is inconsistent with that of the Holtec witnesses.

516. As described in Contention D, Cask Stability, *supra*, the Board found, based on testimony of Staff witness Jack Guttmann, that the issue in the cask stability analysis is whether the Applicant has reasonably demonstrated that the HI-STORM 100 cask will not tip over when subject to the design basis earthquake. We further found that PFS did not make such a demonstration.

517. In Dr. Cornell's opinion the uncertainties in the soil structure interaction analysis ("SSI") for the PFS site would be comparable to uncertainties for an SSI analysis at a NPP. Id. at 8021. Somewhat contradictory to the foregoing is Dr. Cornell's admission that he is unaware of any NPP site which is supported by cement-treated soil and a layer of relatively soft soils such as at the PFS site. Tr. (Cornell) at 12968. The Board does not accept Dr. Cornell's testimony that the uncertainties in the soil structure interaction analysis at the PFS site are equivalent to those uncertainties at NPPs.

518. The slope of the hazard curve for the PFS site may also be impacted by nonlinear soil behavior. NUREG/CR-6728 recommends that nonlinear soil effects on the determination of the seismic scale factor be included in the development of hazard curve slope. Bartlett Tstmy (Part E), Post Tr. 12776 at 12. The NUREG/CR-6728 concept of accounting for nonlinear behavior is also applicable to any nonlinear behavior, such as a cask sliding on the pad. Id. PFS has not considered nonlinear effects, nor has it calculated the

seismic scale factor based on considerations of the slope of the hazard curve. Id.

519. Dr. Cornell opines that "given the decades of NRC's concern about seismic safety, and given the codes, standards and criteria they call for, one would expect *a priori* similar levels of conservatism in any SSC designed to their SRPs," a similar range of risk reduction ratios. Cornell Tstmy, Post Tr. 7856 at 35 (emphasis added). Without tangible evidence, the Board can give no weight to Dr. Cornell's unsupported supposition that NRC would employ "similar levels of conservatism" resulting in a similar range of risk reduction ratios.

520. Dr. Cornell's opinion that the storage casks will meet a performance goal of 1 x 10^{-4} relies on the cask vendor's computer cask stability analyses – for which there are no actual test data to verify the result, or any quantification of the uncertainties in the model – and on the cask vendor's representation to him of Holtec's cask tipover analysis. Further, the record is clear that Dr. Cornell did not review key Holtec analyses. The Board has already found in Contention Part D that neither the Applicant nor the Staff met the burden of demonstrating that the storage cask will not tipover under a 2,000-year DBE or that the nonlinear finite element computer analyses or the tipover/drop analyses do in fact demonstrate a specific risk reduction ratio. In light of the underlying basis of Dr. Cornell's opinion and our finding in Contention Part D, we reject Dr. Cornell's claim that the storage casks will meet a performance goal of 1 x 10^{-4}.

521. The Licensing Board finds that neither the Applicant nor the Staff have demonstrated a risk reduction ratio for the HI-STORM 100 casks for a 2,000-year DBE at the PFS facility.

Transfer Operations in the CTB

522. Yet another rationale for Dr. Cornell's claimed risk reduction factor of 5 is the time in which the canister is potentially exposed and SSCs are in use during transfer at the PFS site. Cornell Tstmy, Post Tr. 7856 at 26-27. Those transfer operations include transfer of the spent fuel canister from a transportation cask, to a transfer cask and then into a storage cask. For this proposition Dr. Cornell relies on the testimony of Mr. Lewis as to the timing of transfer operations in the CTB. Id. at 27. The timing of transfer operations are described in Contention D, Transfer Operations, *supra.*

523. Dr. Cornell also relies on the Mr. Ebbeson's testimony in the following respect: "the beyond-design-basis analyses and margins described in the testimony of Mr. Ebbeson confirm the existence of significant beyond-design-basis margins in the design of the CTB and the cranes and struts therein, which would enable them to survive earthquake ground motions much greater than those of the 2,000-year design basis earthquake." Id. at 26-27. Of particular note in Mr. Ebbeson's testimony is that the maximum load on the seismic struts (restrain for the cask during transfer operations) due to a 2,000 year DBE is 395 kips but the under the code acceptance criteria the capacity of the seismic struts is 400 kips.

524. The Board has already found that there are serious shortcomings in the Applicant's estimation of the transfer operations in the CTB *See* Contention D, Transfer Operations, *supra.* Furthermore, there are no license conditions or restrictions on the amount of time the seismic struts will be in use or the canister potentially exposed. In addition, the only engineering design calculations to support the PFS license are for a 2,000 year DBE. Given the foregoing, the Board is unwilling to accept such a cutting edge probabilistic

approach to seismic performance.

CTB and Storage Pad Foundations.

525. Dr. Cornell concludes, based on the testimony of Paul Trudeau and Bruce Ebbeson, that due to "differences such as those between calculated and design safety factors, realistic dynamic and the assumed static behavior, mean and the lower bound soil properties, dynamic and static soil properties, etc., that there is a significant margin with respect to the ground motions that might cause overturning or bearing failure of these foundations." Cornell Tstmy, Post Tr. 7856 at 27. Further, opines Dr. Cornell, the CTB foundation would have a risk reduction ratio of 5 or greater because Mr. Trudeau and Mr. Ebbeson estimate that the CTB foundation would be able to withstand 10,000-year ground motions. Id.

526. Dr. Cornell's opinion must be tempered with the Board's finding that there are no engineering calculations to support PFS's supposition that its facility can withstand a 10,000-year DBE. If such were the case, the Board sees no reason why PFS should be applying to the NRC for an exemption from the deterministic ground motions requirements. We note that Geomatrix computed the deterministic ground motions for the PFS site to be approximately 1.15 g, and those same ground motions are likely for a 10,000-year DBE. CITE. *See* Con-SER at 2-34, Staff Exh. C; Tr. (Trudeau) at 6342; Ebbeson Tstmy, Post Tr. 6357 at 9.

527. Dr. Cornell similarly concludes that based on the testimony of Paul Trudeau, the storage pads have margins of safety against overturning and soil bearing failure "at or approaching" mean return periods five times the 2,000-year DBE level. Cornell Tstmy, Post Tr. 7856 at 28.

528. Although Dr. Cornell's opinion relies, in part, on Paul Trudeau's testimony concerning the foundation stability of the storage pad and CTB for a 2,000-year DBE, he did not review Mr. Trudeau's foundation stability design calculations, including his methodology or all of his assumptions. Id. at 7989-91. Moreover, neither Mr. Trudeau nor any other witness has not performed any foundation stability calculations for a 10,000-year mean return period earthquake and has not shown that the foundations meet a factor of safety of 1.1 for that case. Tr. (Bartlett) at 12874; (Trudeau) at 6348. In attempting to make a point about no hazardous material release, Dr. Cornell, relying on other PFS witnesses, admitted there would be sliding of the storage pads for a 10,000-year mean return period earthquake. Cornell Tstmy, Post Tr. 7856 at 28.

529. Dr. Cornell opines again that because of "NRC's long concern over seismic safety margins there is *a priori* reason to expect" similar risk reduction ratios for the CTB foundation to those of NPPs. Cornell Tstmy, Post Tr. 7856 at 27 (*emphasis added*). Consistent with our finding earlier, the evidence does not support Dr. Cornell's confidence in NRC establishing similar risk reduction ratios for the foundations.

530. The Licensing Board is unconvinced that Dr. Cornell has shown that the CTB and storage pad foundations have a risk reduction ratio of 5 because, without reviewing the details, Dr. Cornell relies on the analyses of Mr. Trudeau and Mr. Ebbeson; there are no engineering calculations to support PFS's claim that the CTB and pad foundations can withstand a 10,000-year DBE; and our finding *supra*, that the Applicant has failed to meet its burden in demonstrating an adequate seismic design for foundations at a 2,000-year DBE.

Evidence that Risk Reduction Ratios for ISFSIs are "Similar" to those for

NPPs

531. The Applicant, through Dr. Cornell, proffers by analogy that the risk reduction ratio for ISFSIs is "similar" to that calculated for "typical SSCs" commonly found at NPPs. Cornell Tstmy, Post Tr. 7856 at 19-20. According to Dr. Cornell, NUREG/CR-6728 shows a quantitative finding that "typical" NPP SSCs have a risk reduction factor in the range of 5 to 20 or greater. Id. (*citing* NUREG/CR-6728 at Chapter 7). These NPP risk reduction ratios in NUREG/CR-6728 were estimated from PRAs, including fragility curves, performed at various NPPs. Id. at 20. Dr. Cornell concludes that the acceptance criteria, procedures, and guidelines in the NRC Standard Review Plans ("SRP") for NPPs have risk reduction ratios "as large as, or larger than" those established for PC4 facilities in DOE Standard 1020. Id. at 19. Notwithstanding his conclusion, Dr. Cornell admits that the "NRC [NPP] seismic SRPs are not explicitly keyed" to risk reduction ratios. Id. No other party supports the Applicant's "similarity argument."

532. It is important to note that NPP SRPs do not address the seismic performance requirements of unanchored casks supported by shallowly embedded pad foundations buttressed by cement-treated soil and subject to high levels of strong ground motion. Bartlett Tstmy (Part E), Post Tr. 12776 at 13. Specifically, free-standing storage casks are not typical of SSCs found at commercial NPPs. Cornell Tstmy, Post Tr. 7856 at 20. Reactor pressure vessel and primary coolant systems at NPPs are anchored and not allowed to freely slide, rotate, or uplift under seismic forces. Tr. (Cornell) at 7968-69. The methods used to analyze the sliding and tipping stability of free standing casks are not normally encountered in NPP SSC analyses. Id. 7970. Thus, risk reduction ratios encompassed in

SRPs for reactor pressure vessel and primary coolant systems at nuclear power plants cannot be inferred under PFS's "similarity argument" to free standing dry storage casks. Id. at 7969.

533. Although not aware of the "details," Dr. Cornell testified that the NPP PRAs discussed in NUREG/CR 6728 accounted for the sliding, overturning, bearing failures. Id. at 12952. Thus "infers" Dr. Cornell, NPP foundations have risk reduction ratios at least in the range of five to twenty. Id. Assuming *arguendo* that the NPP PRAs did demonstrate risk reduction ratios of between five and twenty for foundations, Dr. Cornell has failed to demonstrate that the NPP risk reduction ratios for foundations are applicable in this case. Dr. Cornell admitted that none of the NPPs evaluated were supported by cement-treated soil or soil cement. Id. at 7945-47; 12968. Moreover, Dr. Cornell could not identify a NPP site where foundations are supported by soil cement and relatively soft soils. Id.

534. Dr. Cornell claims that "NRC SRPs contain many conservatisms that result in risk reduction factors as large as, or larger than, those for PC4 category facilities designed to DOE-STD 1020-94." Cornell Tstmy Post Tr. 7856 at 19 (*emphasis added*). Dr. Cornell relies, in part, on NUREG/CR-6728 to support this statement. Id. NUREG/CR-6728 has quantified levels of risk reduction ratios in the range of 5 to 20 for certain NPP SSCs whereas in DOE Standard 1020 NPP SRPs have risk reduction ratios in the range of 10 to 20. The Board finds that the reverse is true – that DOE-STD-1020 has greater risk reduction factors than does NUREG/CR-6728. Furthermore, Dr. Cornell does not address under his "similarity argument" whether and how NUREG/CR-6728 calculations of some risk reduction ratios below that required by DOE Standard 1020 would result in risk reduction ratios of 5 for PC3 facilities. *See* DOE-STD-1020-02 at Table C-3 (Staff Exh. QQ).

535. Compared to the original deterministic standard, the 2,000-year DBE results in a substantial reduction in the seismic demand against which PFS has designed its facility. Bartlett Tstmy (Part E), Post Tr. 12776 at 6. Additionally, although Dr. Cornell maintains that because of the codes and standards imposed, the margin of safety is the same regardless of the established DBE, it is important to note a 2,000-year DBE reduces the safety level achieved (or increases the probability of failure) when compared to a deterministic DBE. Tr. (Cornell) at 7913-14.

536. Although a factor of safety may be the same for different DBEs, the amount of actual design margin is different. A factor of safety is a function of the capacity divided by the demand. Tr. (Bartlett) at 12834-35. Thus, if the factor of safety is kept constant and the demand is reduced from a 10,000-year DBE to a 2,000-year DBE, then the capacity is also is reduced. Id. at 12834-12837. Although the factor of safety is the same for both earthquakes, the actual capacity – the design margin – is larger for the 10,000-year earthquake compared to the 2,000-year earthquake. Id. at 12837. The Board finds that PFS's 2,000-year DBE design does not have the same design margin as a 10,000-year DBE design for a NPP.

537. Dr. Cornell's opinion that SCCs at the PFS facility have risk reduction ratios of "5 to 20 or greater" is inconsistent with his other testimony that the margins are 2 to 3 times the design basis capacity. Tr. (Cornell) at 7916-17.

538. The Licensing Board finds that the Applicant has not met its burden of demonstrating that its SCCs meet a supportable performance goal and risk reduction factors for a 2,000-year DBE. First, in authorizing a 2,000-year DBE, the Staff did not establish a prescribed performance goal or risk reduction factors. Further, Dr. Cornell claims that SSCs

at the PFS site will have risk reduction ratios of five or more. Cornell Tstmy, Post Tr. 7856 at 25-29. The Applicant implies, albeit not directly, that a risk reduction ratio of 5 provides an adequate safety margin because DOE mandates at least a risk reduction ratio of five for performance category 3 facilities, and NPPs have risk reduction ratios of five or greater. *See* id. at 18-19. Notwithstanding extensive testimony concerning the DOE Standard 1020, Dr. Cornell clearly states that he does not "rely upon the DOE Standard for either the appropriate DBE or the risk reduction factor appropriate for the PFSF." Cornell Tstmy, Post Tr. 7856 at 41; *see also* at 36-40.

Board Findings

539. The Board finds that neither PFS nor any party credibly established appropriate performance goals and risk reduction factors for the PFS facility. The Applicant cannot claim that PFS's design meets the performance goal and risk reduction factors in DOE Standard 1020 yet also claim it does not rely on DOE Standard 1020 so it does not have to follow its design philosophy and standards. The Applicant cannot have it both ways. Barring a supportable and defined performance goal and risk reduction factor specific to the PFS ISFSI, we conclude that the Applicant has not met an appropriate performance goal and risk reduction factor.

540. Next, notwithstanding the engineering design arguments posed by the parties, absent a regulatory framework which establishes performance goals and risk reduction ratios, the Board finds that conservatism in the PFS seismic design for a 2,000-year DBE cannot be measured.

541. When considering the seismic design analyses in Contention Part D *supra*, the

Licensing Board found that PFS had not met its burden that the SSCs are adequately designed to a 2,000-year DBE,

542. For the foregoing reasons, the Board finds that PFS has not shown that there is adequate conservatism to demonstrate appropriate risk reduction factors. Further, the Board finds that the Staff has not presented testimony on this issue. Thus, the Licensing Board finds there is insufficient reliable or probative evidence in the record to find that a 2,000-year DBE at the PFS site is reasonably conservative.

543. Further the Board finds that PFS relies on the risk reduction factors to demonstrate that it meets the requirements of 10 CFR § 72.7 for the grant of an exemption to allow a DBE of 2,000-years. The Board finds that PFS's showing does not measure up.

Compliance with Radiation Dose Limits

Issue: In the site-specific analysis that it performed for purposes of demonstrating that it should be granted an exemption from NRC standards for protection from earthquakes, did PFS show that unanchored HI-STORM 100 casks would "reasonably maintain confinement of radioactive material" under off-normal and credible accident conditions at the proposed PFS site, as required by 10 C.F.R. § 72.236? In particular, assuming that casks are tipped over during an earthquake at the PFS site, has PFS satisfied its burden of demonstrating that the radiation levels emitted from the casks will not exceed regulatory limits?

Regulations and Guidance

10 CFR § 72.104(a): "During normal operations and anticipated occurrences, the annual dose equivalent to any real individual who is located beyond the controlled area must

not exceed 0.25 mSv (25 mrem) to the whole body...."

10 CFR § 72.106(b): "Any individual located on or beyond the nearest boundary of the controlled area may not receive from any design basis accident . . . a total effective dose equivalent of 0.05 Sv (5 rem)...."

10 CFR § 72.236(b): "The storage cask and its systems important to safety must be evaluated, by appropriate tests or by other means acceptable to the Commission, to demonstrate that they will reasonably maintain confinement of radioactive material under normal, off-normal, and credible accident conditions."

NUREG-1536, *Standard Review Plan for Dry Cask Storage Systems* (January 1997).

NUREG-1567, *Standard Review Plan for Spent Fuel Dry Storage Facilities* (March 2000).

Findings of Fact

544. Here, the Licensing Board considers the competing views of the parties' witnesses regarding the likelihood that radiation dose limits would be exceeded in the event that casks tipped over during an earthquake. PFS submitted testimony by Dr. Krishna P. Singh, Dr. Alan Soler and Dr. Everett L. Redmond that radiation doses from the casks would not exceed regulatory limits. Dr. Redmond was the person responsible for the dose analyses presented by PFS. Tr. (Redmond) at 12080. Dr. Soler testified that his testimony relating to cask stability and tip over analysis basically related back to Contention Part D and both Drs. Singh and Soler testified that they were not qualified to conduct dose calculations. Tr. (Singh) 12080; (Soler) 12078.

545. NRC Staff witness Mr. Michael D. Waters, a health physicist, also testified that doses from tipped over casks would not exceed regulatory limits. Waters Tstmy, Post Tr.

12215 at 4.

546. Dr. Redmond testified that he used the Monte Carlo computer code for a site specific operational dose analysis under 10 CFR § 72.104(a) that he conducted in support of PFS's license application. Tr. (Redmond) at 12058, 12084. Dr. Redmond computed a 5.85 mrem per year dose to an individual at the owner controlled area boundary, based on a 2,000 hour per year period. Id. at 12084; Singh/Soler/Redmond Tstmy, Post Tr. 12044 at 8.

547. Dr. Redmond used 2,000 hours in computing the normal operational dose analysis based on his presumption that the nearest resident is two and a half miles away and the land beyond the owner controlled area boundary is unoccupied. Tr. (Redmond) at 12092. Dr. Redmond testified that has not visited the site, and did not know of present or future land use around the site. Id. at 12081-82.

548. Dr. Redmond did not conduct an analysis specific to PFS's exemption request and has not reviewed same. Id. at 12084-86. Instead, Dr. Redmond used the dose computation he did to satisfy PFS's normal operational dose calculation, based on a 2,000-year DBE, as a baseline and extrapolated from that computation for beyond design basis accident conditions. Id. at 12101.

549. In this part of the proceeding, PFS is relying on "conservatisms" built into PFS's design and the testimony of the Holtec cask vendors. Singh/Soler/Redmond Tstmy, Post Tr. 12044 at 5, 7, 15-17, 30; Tr. (Singh, Soler) at 12079-80; *see also* Contention Part D, *supra*. Dr. Redmond testified his starting premise was that the casks would not tip over and that any damage to the casks if they did tip over would be "localized" - although he could not quantify the effect. Tr. (Redmond) at 12068-69, 12093, 12097-98. Dr. Redmond further

testified that he had no experience in how the casks would be orientated if they did fall over. Id. at 12102.

550. The State presented testimony by Dr. Marvin Resnikoff, a qualified expert in the computation of radiation doses. Dr. Resnikoff calculated that the total dose from an array of tipped over casks, from direct gamma radiation, direct neutron radiation, and photons, would be 150 millirems per year. Id. at 12360. He noted that there are other dose contributors that he did not consider. Id. at 12371, 12380. Dr. Resnikoff chose to analyze Cobalt-60 and Cesium-137 because those are the primary contributors to gamma dose. Id. at 12638. He testified that there are other gamma emitters, and also the neutron producers are longer lived generally. Id. His calculations did not take into account the additional effects of cask heat-up. Id. at 12374.

551. Dr. Resnikoff acknowledged that he had done a rough calculation. Id. at 12639. There are a number of factors that he was unable to consider, such as the production of gamma by neutrons moving out of the cask, scattering, and radiation coming from other parts of the cask. Id. He testified that in his expert opinion, a calculation should be performed using the Monte Carlo method, especially with respect to the bottom of tipped over casks. Id. at 12639.

552. Dr. Resnikoff also identified a number of respects in which PFS had applied an incorrect standard or performed an inadequate technical analysis of the potential for exceeding the dose limits in 10 C.F.R. § 72.106. The key areas in which Dr. Resnikoff identified deficiencies were (a) incorrect assumption regarding the number of hours for the individual specified in 10 C.F.R. § 72.106; PFS's failure to specify a reasonable accident

duration period; and PFS's failure to perform a calculation regarding doses during a tip-over accident. Resnikoff Tstmy, Post Tr. 12349 at 4, 6, 7.

Applicable Standard

553. An initial question arises concerning the applicable standard. In his original prefiled testimony of April 1, 2002, which he later amended, Dr. Resnikoff applied the standard for normal operations that is found in 10 C.F.R. § 72.104(a). As counsel for the State explained at Tr.12451-52, this was appropriate because at the time when PFS filed its initial exemption request for a 1,000-year DBE, and later request for a 2,000-year DBE, PFS did not follow SECY 98-126, the guidance in effect at the time. Under SECY 98-126 an applicant must demonstrate compliance with 10 C.F.R. § 72.104(a) during a seismic event in order to use a PSHA methodology with a DBE of 1,000 years or, alternatively, the applicant must use a 10,000-year DBE and show compliance with 10 CFR § 72.106(b). SECY-98-126 (option 3). Only later, when SECY-98-126 was superseded, did section 72.106 arguably become relevant for PFS's request to use a PSHA with a DBE of 2,000 years.

554. The Board is faced with the question of whether the radiation dose limit applicable to PFS's analysis should be based on 72.104(a) or 72.106(b). Under current regulations, PFS must analyze accident dose limits from a deterministic earthquake (similar to a 10,000-year DBE) under 72.106(b). If, in fact, the record shows that PFS has provided supportable analysis for a 10,000-year mean return period event, then 72.106(b) is the applicable standard. If, however, PFS is relying on analyzing releases from a 2,000-year DBE to satisfy its exemption request, then by allowing PFS to use the 72.106(b) standard, the Board would be expanding the effect of PFS's request to be exempted from 10 CFR § 72.102

to a dilution of the standard in 10 CFR § 72.106(b).

Number of Hours of Radiation Exposure in a Year

555. PFS's position is that the cask will not tip over, *ergo* there is no need to perform a quantative dose calculation to determine whether PFS can meet the 5 rem limit in 10 CFR § 72.106(b). Tr. (Redmond) at 12093-95 ("we wouldn't do an analysis for a HI-STORM cask, obviously, because it's a hypothetical condition."). Instead PFS extrapolated from its operational dose calculation to demonstrate that it complies with 10 CFR § 72.106(b). Singh/Soler/Redmond Tstmy, Post Tr. 12044 at 8. PFS testified that it performed a qualitative analysis and concluded that doses during a cask tip-over accident would be on the same order as doses under normal operation, or about 5.85mrem/year. Id. In reaching this qualitative conclusion, PFS assumed that a person would be at the "fencepost" of the controlled area boundary 2,000 hours a year. Id. When questioned about the basis for the assumption of 2,000 hours, Dr. Redmond stated that the assumption was based on "the land usage." Tr. (Redmond) at 12092.

556. The NRC Staff assumed that the individual specified in 10 C.F.R. § 72.106 would be at the fence-post for 24 hours in a day. Tr. (Waters) at 12268.

557. PFS acknowledged that it has no way of excluding anyone from the northern part of the controlled area boundary because it does not own the property. Tr. (Donnell) at 12579-82. Moreover, it is difficult to predict what conditions will be in 20 years – or 40 years when PFS expects its license will terminate. Id.

558. Dr. Resnikoff assumed that a person was at the fence-post 24 hours per day, 8,760 hours per year. Resnikoff Tstmy, Post Tr. 12349 at 6. If one were to assume that an

individual was at the fence post for 24 hours a day, this would amount to 8,760 hours per year. This, by itself, would increase the radiation dose at the controlled area more than fourfold. Id.

559. PFS's assumption that an individual will be at the controlled area boundary for 2,000 hours in a year is not consistent with the language in 10 C.F.R. § 72.106. The regulation refers to "[a]ny individual located on or beyond the nearest boundary of the controlled area." This stands in contrast to § 72.104, which refers to "any real individual." We believe the difference in the language is intentional, and must be honored. While it may be appropriate to assume, for a "real" individual, that a person does not spend 24 hours a day at the controlled area boundary, that assumption is not appropriate for "any individual." The term "any individual" must be assumed to include individuals who are present at the fence post all year. In fact, in the CoC for the HI-STORM 100 System, the NRC Staff agreed with a comment by Dr. Resnikoff that 8,760 hours should be used for estimating the dose at the site boundary. *See* 65 Fed. Reg. 25,241, 25,245 (2000).[78] As long as it is possible that some individual will live at the controlled area boundary and spend his or her days there, PFS must base its calculation on radiation exposure of 24 hours per day, or 8,760 hours. This factor alone would increase PFS's radiation dose estimate four-fold. Resnikoff Tstmy, Post Tr. 12349 at 6.

Duration of Accident

[78]The specific response in the Federal Register states: "*Response*: The NRC agrees that 8,760 hours/year should be used and notes that Section 7.2.9 of the HI-STORM SAR explicitly states that: 'The individual at the site boundary is exposed for 8,760 hours....'"

560. PFS did not make an estimate of the duration of an accident. Tr. (Redmond) at 12093. Further, there is no evidence in the record that PFS has any contingency plan for uprighting casks if they tipped over. Tr. (Singh) at 12114; (Soler, Redmond) 12115; (Resnikoff) 12600. Nevertheless, PFS testified that it would be reasonable to assume that an accident lasts for 30 days. Tr. (Redmond) at 12093. PFS did not attempt to justify this assumption, but merely relied on NUREG-1536 or -1567.[79]

561. The NRC Staff also assumed that the accident event lasted 30 days. Tr. (Waters) at 12265. This assumption was based on NUREG-1567, the *Standard Review Plan for Spent Fuel Dry Storage Facilities*. Staff Exh. 53. While Staff witness Waters testified that he believed 30 days was reasonable, there is no evidence that his opinion was based on the existence of any contingency plan or actual knowledge of how long it would take to restore the site to pre-accident conditions. Tr. (Waters) at 12267. Instead, Mr. Waters relied on what he called "fundamental principles of radiological protection" – time, distance, and shielding. Id. at 12266. He testified that he would expect people near the fence post to have been moved away within 30 days of an accident. Id. at 12267.

562. Both Dr. Resnikoff and Dr. Singh agreed that convection cooling is inhibited in a tipped-over cask. Tr. (Singh) at 12180; (Resnikoff) at 12537. As Dr. Resnikoff noted, heatup of the concrete cask has an adverse effect on the effectiveness of neutron shielding. Tr. (Resnikoff) at 12406-07. Dr. Redmond testified that neutron doses are included in his normal operational dose computation but he has not conducted a neutron dose calculation

[79]Initially Dr. Redmond testified that he relied on NUREG-1536 but later thought the referance may be to NUREG-1567. Tr. (Redmond) at 12093; 12171-77.

under accident conditions. Tr. (Redmond) at 12100; *see also* Resnikoff Tstmy, Post Tr. 12349 at 10. Because PFS and the Staff assumed an accident event would last, at most, 30 days, there is the potential for an increase in neutron emissions if an accident persists for a longer period. Some guidance may be taken from the fact that the CoC for the Holtec 100 cask system requires that the ducts on a cask must be cleared within 33 hours. Staff Exh. FF at 11-5. 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 Depo. Tr. at 60-61, State's Exh. 137. Holtec calculations show that after 33 hours of 100% air inlet blockage, the concrete temperature will exceed the short-term limit of 350 degrees F specified in the CoC for the HI-STORM 100 cask. *See* HI-STORM 100 TSAR, p. 1.D-4, Table 1.D.1 (Rev 10), State's Exh. 139. During the hearing, however, Dr. Singh testified that it would take 80 to 100 hours to reach such a temperature. Tr. (Singh) at 12153. The record contains no quantative analysis for Dr. Singh's opinion.

563. It is unlikely that all the casks could be uprighted in a short period of time of time. Tr. (Resnikoff) at 12506. PFS has made no plans to have on hand a crane that could lift fallen casks, each weighing 175 tons. Resnikoff Tstmy, Post Tr. 12349 at 11. Nor has PFS described a contingency measure for getting a suitable crane to the site. While Dr. Singh testified that a "standard" crane with a yoke could lift a cask, he was speaking in purely generic terms. Tr. (Singh) at 12193. He also stated that he did not know what kind of a crane would be used at the PFS site. Id. at 12115. He also failed to point to any documentation that the crane PFS intends to use under normal conditions is adequately designed, yoke or

not, to stand 175-ton casks upright after an earthquake. As Dr. Resnikoff testified, an analogy can be made to the game of pickup sticks – if casks are impinging on each other, it could be difficult to pick a single cask up. Tr. (Resnikoff) at 12507. Moreover, it could be difficult to find a place to set a cask down. Id. at 12507-08.

564. Dr. Singh also speculated about other alternative measures for moving and uprighting casks, such as air pads, but his testimony did not testify regarding actual PFS contingency plans for the use of such measures. Tr. (Singh) at 12192.

565. PFS also testified that it could position steel plates around the periphery of the fallen casks, but once again PFS did not have any contingency plan. Tr. (Redmond) at 12126. Finally, PFS did not explain how installing steel plates around fallen casks would actually terminate the accident, when the casks had not been uprighted.

566. Illustrative of how the duration an "event" can last for many years is the situation at Palisades reactor. Concerned with quality assurance problems in a multiple purpose dry storage cask canister ("MPC") and how to remove the MPC from the cask, the situation at Palisades has lingered on for more than five years without resolution. Tr. (Resnikoff) at 12600-01, 12639-40.

567. It is important to note that 10 C.F.R. § 72.106 gives a total radiation dose limit, rather than a yearly or other time-dependent dose limit. We interpret this to mean that the dose limit may not be exceeded in the course of the entire accident. Thus, it becomes important to evaluate the length of an accident. PFS has cited to NUREG-1536, which constitutes the NRC Staff's guidance on the matter. Tr. (Redmond) at 12058. However, regulatory guides are not binding regulations. Louisiana Energy Services (Claiborne

Enrichment Center), LBP-91-41, 34 NRC 332, 354 (1991). Intervenors are not precluded from demonstrating that a prescribed method "is inadequate in the particular circumstances of the case." Id. *citing* Public Service Co. of New Hampshire (Seabrook Station, Units 1 and 2), ALAB-875, 26 NRC 251, 261 (1987); Gulf States Utilities Co. (River Bend Station, Units 1 and 2), ALAB-44, 6 NRC 760, 772-73 (1977). Here, the State has provided substantial evidence that the 30-day assumption, as applied to the PFS facility, is unreasonable. PFS has no contingency plan for righting the casks if they tip over. Tr. (Waters) at 12269. It could take a great deal of time to acquire an appropriate crane, get it into position, and right a large number of casks. Tr (Resnikoff) at 12506-08. The effort could be more prolonged if casks may have fallen on each other and/or have suffered damage during an earthquake. Id.

568. The Board finds there is simply no rational basis to presume that after a serious accident, PFS would be able to restore the site to normal conditions in only 30 days. Under such potentially severe post-earthquake conditions, it is not conceivable that many hundred casks can be set upright within 30 days.

569. Moreover, while it may be possible to mitigate radiation doses by installing steel plates around the periphery of the site as an interim measure, we would not consider that to constitute the termination of the accident. We think the principles expressed by Mr. Waters, of "time, distance, and shielding," are out of place in this context, and that the Staff's assumption of a 30-day accident is based on a fundamental misconception and misapplication of 10 C.F.R. § 72.106. The Staff essentially adopts the assumption that the "accident," as the term is used in section 72.106, ends when people are removed from the area or some temporary barriers have been erected. Tr. (Waters) at 12267-69; 12314-15. However, we

believe that the regulation can only be interpreted to mean that the accident has ended when the casks are set upright and restored to their previously designed condition in which the doses they emit are within the limits of 10 C.F.R. § 72.104(a). In reaching this conclusion, we rely on the language and the purpose of Section 72.106 itself.

570. The clear purpose of Section 72.106 is to ensure that the design of a proposed ISFSI is adequate to protect against excessive radiation doses in the event of an accident. The elements of the design consist of the casks and pads themselves, and the size and configuration of the controlled area. There is no reference in the standard to contingency measures, whether planned or ad hoc. They are not part of the design of the facility.

571. Moreover, it would violate the principle of defense-in-depth if contingency measures may be relied on as a substitute for an adequately designed ISFSI and controlled area. The physical design of a facility must be evaluated on its own merit against NRC standards for safe facility designs. Contingency measures constitute additional, independent steps for protecting the public in the event of an accident, not substitutes for an adequate design. Otherwise, the design requirements of 10 C.F.R. § 72.106 could be diluted merely by listing post-accident measures that could or would be taken to mitigate doses to the public.

572. In any event, it is fundamental to nuclear regulation that if such measures are to be relied on in a licensing decision, they must be planned, not ad hoc. Clearly, that is not the case here.

573. In summary, we believe that the Staff's interpretation of when an accident ends is fundamentally inconsistent with the language and purpose of 10 C.F.R. § 72.106, and violates fundamental principles of NRC safety regulation. An accident cannot be considered

to have ended until the casks have been restored to a condition in which their radioactive emissions are within the limits for normal operation, *i.e.*, the limits in 10 C.F.R. § 72.104(a). PFS has made no attempt to determine how long it would take to restore the proposed ISFSI to a normal condition. Given the immense size of the facility, and the potential complexity of restoring the casks to an upright state, PFS's failure to address this issue is fatal to its application. Further, even if the Board were to accept PFS's and the Staff's position that the duration of the event is only 30 days, neither PFS nor the Staff have conducted an analysis under normal operating condition to determine whether, post-accident, the casks would comply with the dose limits in 10 CFR § 72.104(a).

Assumptions re Configuration of Tipped-over Casks

574. In its tipover analysis, Holtec assumed that the casks would start at zero angular velocity, but the State refuted this assumption. HI-STORM 100 TSAR, § 3.A.6, State's Exh. 139. Dr. Bartlett testified that as you reach incipient tip over, then the cask will have some velocity going past the tip over point. Tr. (Bartlett) at 12913-15. In his pre-filed testimony, Dr. Resnikoff raised the concern that if the initial angular velocity of the casks during tip over were greater than zero, then there would be more cask flattening than contemplated by PFS. Resnikoff Tstmy, Post Tr. 12349 at 8.

575. In estimating radiation doses at the site boundary in a cask-tipover event, it is necessary to make some assumptions about how the casks will fall. The orientation of the casks, whether they have fallen onto each other, and whether they are stretched or flattened by the force of falling on each other, will have an effect on the dose that is calculated. Id. at 8-9.

576. Dr. Resnikoff performed an analysis assuming that the bottoms of a row of casks faces the fence post. He assumed this configuration because it was conservative. Tr. (Resnikoff) at 12428.

577. The NRC Staff performed calculations, assuming essentially the same configuration as assumed by Dr. Resnikoff. Tr. (Waters) at 12243. Mr. Waters assumed that 50 casks would be tipped over facing a northern direction. He considered that this would be the bounding case. Id. at 12257.

578. In his oral testimony, PFS witness Dr. Singh criticized Dr. Resnikoff for assuming that the bottom of the casks faced outwards toward the controlled area boundary. Tr. (Singh) at 12138. However, he did not assert that Dr. Resnikoff's assumption was nonconservative. In contrast, PFS made no attempt to model the alignment of the cask.

579. The Board believes that it is reasonable to require PFS to either prepare a defensible model of the configuration of tipped-over casks, or to make conservative assumptions about their configuration. Otherwise, we cannot find the reasonable assurance of safety that the regulations require.

Adequacy of Method Used by PFS to Calculate Doses

580. PFS's witness, Dr. Redmond, was responsible for the Applicant's dose analysis. Tr. (Redmond) at 12083. He is an expert in Monte Carlo analysis. Id. at 12082. He testified that Monte Carlo is "the most state-of-the-art code, or technique" for performing a radiation dose analysis. Id. Dr. Redmond performed a Monte Carlo analysis for the normal case, but did not do so for the accident case. Id. at 12086-87. PFS's evidence regarding the accident case was based on a qualitative analysis. Singh/Soler/Redmond Tstmy, Post Tr.

12044 at 7.

581. Dr. Redmond could have used the Monte Carlo method to calculate radiation doses in a tip over accident. Tr. (Redmond) at 12087. Yet, despite Dr. Redmond's expertise in the methodology, and despite its ready availability, PFS chose not to apply the Monte Carlo method. Id. Dr. Redmond testified that it would have taken him only a few days to change the Monte Carlo code to produce such a calculation. Id. at 12186.

582. Dr. Redmond testified that in the calculation for normal conditions, he did not calculate the radiation coming out of the bottom of the cask. Id. at 12121. The reason he gave for his failure to calculate the radiation dose from the bottom of the cask was that a cask tipover was a hypothetical situation, and PFS had decided not to look at such situations. Id. at 12124. While Dr. Redmond gave a qualitative opinion that the doses would not be significant, he had not made a calculation to back it up. Id. at 12125.

583. We find that PFS has not met its burden of proving that its exemption request is justified. If the casks should tip over in a severe earthquake, PFS has not shown that they will be able to contain radioactive emissions such that accident dose limits are not exceeded. Despite the accessibility of state-of-the-art Monte Carlo methodology for assessing radiation doses in a cask tip-over accident, PFS has made no attempt to model radiation doses in a cask tip over event. Moreover, it has made assumptions about the likely alignment of casks that are not supported by any model. Similarly, PFS has not calculated the amount of time before a cask must be uprighted in order to avoid a loss of integrity of concrete shielding.

584. The State has presented evidence of serious shortcomings in PFS's analytical method. While the State has not presented sufficient evidence to prove that doses will be

above regulatory limits during an accident, the State does not carry the burden of proof. Instead, it is the State's burden to go forward with evidence that challenges the sufficiency of PFS's application, to the extent that PFS must respond with proof that the State's concerns have been addressed. We find that the State has met its burden of going forward to which PFS has not adequately responded.

Annual or Lifetime Risk

585. Dr. Cornell has proposed an approach that is based strictly on annual risk without taking into account the life of the facility. Dr. Cornell's position is based, in part, on the fact that nationally there will be some risk wherever the fuel is stored. Arabasz Tstmy, Post Tr. 9098 at 16. While this argument may have some acceptance in setting a risk consistency regulatory standard, here we are asked to make a site specific decision on the acceptable risk of fuel storage at the PFS facility, which will have an expected operational life of 40 years. Moreover, Dr. Cornell admits that the cumulative fatality risk of an individual living next to the PFS facility for 40 years is greater than an individual living next to the facility for 20 years. Tr. (Cornell) at 8008. We are also asked to make this decision in the context of an exemption from a duly promulgated regulatory standard. Similar to the Uniform Building Code which considers the public interest by bringing in the exposure period to its decision-making process (Tr. at 9194-96), and the NRC's justification for its rulemaking plan, here we also consider the lifetime risk. Certainly, if the Board were asked to establish a design basis earthquake for the PFS facility, it would have to set the bottom marker at a minimum of 4,000 year mean return period event. See Tr. (Arabasz) 10152-53. This could be justified on the basis for a one percent probability of exceedance in the 40 year

lifetime of the PFS facility; it would be also be greater than the mean return period event for the reference probability for a nuclear power plant located at a site with a steep hazard curve (*i.e.*, 5,000 years).

Public Interest

586. There was no evidence presented that the Staff considered the public interest in agreeing to PFS's exemption request. Certainly it was obvious during the testimony of Drs. Stamatakos, McCann and Chen, that they did not consider the public interest. Tr. at 8249-55.

587. Even if the following testimony by Staff witness Dr. McCann is an effort to address the public interest, it misses the mark by taking costs considerations into account. Dr. McCann testified:

> 4,000 [MRP] gives you more safety, obviously. Is it more for, you know, too much money, thinking of the public's general interest? That's I think where you would argue that yeah, maybe 4,000 is not they way to go.

Tr. at 8278. The Board cannot take cost considerations into account as part of its health and safety analysis. Consequently, we see no basis to consider cost saving to the Applicant as part of the public interest. To do otherwise would allow cost considerations in through the back door.

588. PFS is requesting an exemption because the 1.15g (horizontal) and 1.17g (vertical) peak ground accelerations estimated using a worst case earthquake exceeding the design values in PFS's Safety Analysis Report. ConSER at 2-34. The Board first notes that the PFS facility is not the optimum design for the Skull Valley site. The Board also takes notice of the federal government's actions in its atomic weapons testing program during the

1950s and 1960s and the effects this had on the citizens of Utah. Given this legacy it is salient that the citizens of Utah should not be expected to bear risk from an untested and unconventionally designed nuclear storage facility. Unlike the INEEL ISFSI, where there may have been a public interest in storing fuel from the Three Mile Island incident, PFS will be storing spent nuclear fuel from commercial reactors. On balance, the public interest outweighs PFS's rationale for its exemption request, *i.e.*, that deterministic ground motion values exceed the design values in PFS's Safety Analysis Report.

E. Summary:

1. The PFS site has a fault dipping under the site and, located 5 miles from the site, the Stansbury fault is capable of generating a magnitude 7 earthquake.

2. There are significant differences between western coastal sites where they, in general, have steep hazard curves, are located near tectonic plate boundaries, and the risk reduction ratios are much lower than those in the Intermountain area.

3. Utah highways and some buildings under the Uniform Building Code have a design basis earthquake with a MRP of 2,500 years.

4. The benchmark design basis earthquake for the PFS site has a return period between 2,500 years and 10,000 years.

5. The Applicant has conducted an acceptable albeit not conservative PSHA.

6. The Staff has not put forward a logical and consistent justification for recommending the grant of a 2,000-year return period DBE for the PFS site.

7. The Applicant has not shown that there adequate conservatism to demonstrate appropriate risk reduction factors in support of a 2,000 year DBE.

8. If the casks tipover, the Applicant has not shown that they will be able to contain radioactive emissions.

8. Looking a lifetime risk for the expected 40 year design life of the PFS facilty, an absolute minimum DBE would be a 4,000 year mean return period event.

9. The Staff did not consider the public interest in its review of the PFS exemption request. The public interest in not being at risk from an untested and unconventionally designed facility outweighs PFS's reason for requesting an exemption - *i.e.*, that the deterministic design ground motions are too high for PFS's design.

F. Conclusions of Law:

Based on the evidence presented, a design basis earthquake for a 2,000-year return interval at the PFS site:

1. is not in the public interest;
2. is not founded on a proper technical basis;
3. may effect health and safety from the release of radiation; and
4. does not comply with 10 CFR § 72.7.

VI. OVERALL CONCLUSIONS OF LAW

Based on the foregoing, the Board concludes that PFS has not met the requirements of 10 CFR §§ 72.7, 72.90, 72.102(c), 72.102(d), 72.104(a), 72.106(b), 72.120(a), 72.122(b)(1), 72.122(b)(2), and 72.236(b). Therefore, the Board concludes that the exemption request should not be granted and PFS's licence application should be rejected.

DATED this 5th day of September, 2002.

Respectfully submitted,

Denise Chancellor, Assistant Attorney General
Fred G Nelson, Assistant Attorney General
Connie Nakahara, Special Assistant Attorney General
Diane Curran, Special Assistant Attorney General
Laura Lockhart, Assistant Attorney General
Attorneys for State of Utah
Utah Attorney General's Office
160 East 300 South, 5th Floor, P.O. Box 140873
Salt Lake City, Utah 84114-0873
Telephone: (801) 366-0286, Fax: (801) 366-0292

CERTIFICATE OF SERVICE

I hereby certify that a copy of STATE OF UTAH'S PROPOSED FINDINGS OF FACT AND CONCLUSIONS OF LAW ON UNIFIED CONTENTION UTAH L/QQ was served on the persons listed below by electronic mail (unless otherwise noted) with conforming copies by United States mail first class, this 5th day of September, 2002:

Rulemaking & Adjudication Staff
Secretary of the Commission
U. S. Nuclear Regulatory Commission
Washington D.C. 20555
E-mail: hearingdocket@nrc.gov
(*original and two copies*)

Michael C. Farrar, Chairman
Administrative Judge
Atomic Safety and Licensing Board
U. S. Nuclear Regulatory Commission
Washington, DC 20555-0001
E-Mail: mcf@nrc.gov

Dr. Jerry R. Kline
Administrative Judge
Atomic Safety and Licensing Board
U. S. Nuclear Regulatory Commission
Washington, DC 20555
E-Mail: jrk2@nrc.gov
E-Mail: kjerry@erols.com

Dr. Peter S. Lam
Administrative Judge
Atomic Safety and Licensing Board
U. S. Nuclear Regulatory Commission
Washington, DC 20555
E-Mail: psl@nrc.gov

Sherwin E. Turk, Esq.
Catherine L. Marco, Esq.
Office of the General Counsel
Mail Stop - 0-15 B18
U.S. Nuclear Regulatory Commission
Washington, DC 20555
E-Mail: set@nrc.gov
E-Mail: clm@nrc.gov
E-Mail: pfscase@nrc.gov

Jay E. Silberg, Esq.
Ernest L. Blake, Jr., Esq.
Paul A. Gaukler, Esq.
Shaw Pittman, LLP
2300 N Street, N. W.
Washington, DC 20037-8007
E-Mail: Jay_Silberg@shawpittman.com
E-Mail: ernest_blake@shawpittman.com
E-Mail: paul_gaukler@shawpittman.com

John Paul Kennedy, Sr., Esq.
David W. Tufts
Durham Jones & Pinegar
111 East Broadway, Suite 900
Salt Lake City, Utah 84111
E-Mail: dtufts@djplaw.com

Joro Walker, Esq.
Land and Water Fund of the Rockies
1473 South 1100 East, Suite F
Salt Lake City, Utah 84105
E-Mail: utah@lawfund.org
(*electronic copy only*)

Larry EchoHawk
Paul C. EchoHawk
Mark A. EchoHawk
EchoHawk Law Offices
151 North 4th Street, Suite A
P.O. Box 6119
Pocatello, Idaho 83205-6119
E-mail: paul@echohawk.com

Tim Vollmann
3301-R Coors Road N.W. # 302
Albuquerque, NM 87120
E-mail: tvollmann@hotmail.com

James M. Cutchin
Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001
E-Mail: jmc3@nrc.gov
(electronic copy only)

Office of the Commission Appellate Adjudication
Mail Stop: O14-G-15
U. S. Nuclear Regulatory Commission
Washington, DC 20555

Denise Chancellor
Assistant Attorney General
State of Utah