253. Dr. Ostadan and Dr. Bartlett also acknowledged repeatedly that their concern about the lack of margin in the PFSF design is limited to the foundations under the storage pads and the CTB, which is their area of expertise and interest. Tr. 7378-79, 10616, 10622, 10675-78, 10683-84 (Ostadan); Tr. 11209, 11334, 11904 (Bartlett).

254. Despite the State witnesses' misgivings, the testimony by the Applicant establishes that there are significant conservatisms in the analysis and design of the foundations of the CTB and the storage pads, such that the actual margins against the mechanisms for potential foundation failure are much larger than the State credits in its testimony. Testimony of Paul J. Trudeau on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 6135) [hereinafter "Trudeau Section D Dir."] at A14 - A18; Trudeau Soils Reb. at A2 – A3; Ebbeson Dir. at A16; Rebuttal Testimony of Bruce E. Ebbeson on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 10790) [hereinafter "Ebbeson Reb."] at A3; Tr. 6143-50, 6170-71, 6272-76, 6316-20, 11733-35, 11965-68 (Trudeau); Tr. 7990-91, 12952-56 (Cornell).

255. There is also ample uncontested testimony in the record that the designs of the CTB, the casks and the storage pads incorporate such wide margins of safety that even if a failure of the foundations took place there would be no adverse safety consequences. Testimony of Bruce E. Ebbeson on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 6357) [hereinafter "Ebbeson Dir."] at A8 - A21; Testimony of Krishna P. Singh and Alan I. Soler on Unified Contention Utah L/QQ (inserted into the record after Tr. 5750) [hereinafter "Singh/Soler Section D Dir."] at A17- A23; Testimony of C. Allin Cornell (inserted into the record after Tr. 7856) [hereinafter "Cornell Dir."] at A38 – A52;

Tr. 5858-63 (Singh); Tr. 6360-61, 6446-53, 6461 (Ebbeson); Tr. 6972-77 (Guttman); Tr. 7989-90, 7992-93, 8025-27 (Cornell).

256. Given that Applicant has demonstrated the existence of significant margins in the designs of the CTB, the storage casks and pads, and their respective foundations, the State needs to make a showing that the specific deficiencies it postulates would have a significant adverse impact on safety. No such showing has been made; in fact, the State witnesses acknowledged that they conducted no calculations or any other analyses to substantiate and quantify their concerns. See, e.g., Tr. 7326-27 (Bartlett, Ostadan). Therefore, the alleged deficiencies raised by the State, even if confirmed to exist, have not been shown to have any safety significance.

5. Summary of Pad Stability Analyses and State's Concerns

257. Applicant's consultant, Stone & Webster, prepared two seismic stability analyses, Calculation Nos. 05996.02-G(B)-04, Rev. 9, *Stability Analyses of Cask Storage Pads* (July 26, 2001) ("Cask Storage Pad Stability Calc. Rev. 9") ("G(B)-04"), and 05996.02-G(B)-13, Rev. 6, *Stability Analyses of Canister Transfer Building* (July 26, 2001) ("CTB Stability Calc. Rev. 6") ("G(B)-13"). PFS Exhibits UU and VV; Trudeau Section D Dir. at A6. In these seismic stability analyses, PFS sought to evaluate three potential "failure modes" for the structures: sliding stability, overturning stability, and bearing capacity stability. Sliding "failure" occurs if the structure moves horizontally, parallel to the ground. Overturning "failure" occurs if the structure rotates as a rigid body about a horizontal axis. Bearing capacity "failure" takes place if the soils beneath the structure become overloaded in the vertical direction, leading to settlement or rotation of the structure's foundation. Trudeau Section D Dir. at A7.

258. The intent of the seismic stability analyses is to establish what margin of safety or "factor of safety" ("FS") is provided by the design of the structure's foundations against each of these failure modes. It is typical in the industry to use FS = 1.1 as the desired safety factor against each of the three above-mentioned failure modes for load combinations that include seismic loads from the design basis earthquake. Trudeau Section D Dir. at A8.

259. Failure to meet the factor of safety of 1.1 against one of the postulated failure modes does not mean that the failure mode in question will occur. It is only when the results of the analysis predict a factor of safety of less than 1.0 that the failure mode in question is possible. Trudeau Section D Dir. at A9. Even then, the conservatisms incorporated into the design of the structures and into the stability analyses mean that, even if the calculated factor of safety is less than 1, the structures may not experience the failure mode in question during the seismic event. Trudeau Section D Dir. at A9; Tr. 6456-60, 10801 (Ebbeson). (As discussed below, the factors of safety for the stability of the critical structures at the PFSF are much greater than the 1.0 threshold.)

260. In addition, because of the cyclic nature of the seismic loading, each of the peak accelerations that impart dynamic loads from the earthquake exists only briefly – typically less than 0.005 seconds – and then reverse direction. Therefore, even if the forces due to the peak acceleration of the earthquake exceeded the resisting forces, a fraction of a second later the accelerations would decrease, and the corresponding inertial forces would decrease as well, such that the structure would not experience significant displacements. Trudeau Section D Dir. at A9.

261. For each type of failure mode, the stability analyses include a "base case" that reflects the design intent with respect to the soils and foundations, and which con-

siders combinations of horizontal and vertical seismic loadings. Trudeau Section D Dir. at A11; Tr. 6164 (Trudeau). In addition, the stability analyses also include hypothetical, "what if" analyses, in which other behavioral modes are explored for various combinations of earthquake loadings. Trudeau Section D Dir. at A11.

262. In the discussion that follows, we will review several concerns that have been raised by the State witnesses with respect to the stability analyses included in calculations G(B)-04 and G(B)-13. It is important to note at the outset that, as will be seen below, the main potential consequence of the concerns raised by the State is the possibility that the structure in question (storage pad or CTB, as the case may be) will fail to satisfy the 1.1 factor of safety against sliding in the event of a design basis earthquake.

263. In raising these concerns, the State witnesses did not express an opinion on the safety consequences of the sliding of the pads or the CTB. Tr. 10693 (Ostadan); Tr. 11904 (Bartlett). Their area of expertise, and the scope of their concerns, is limited to foundation stability and, in particular, to the possibility that the pads or CTB may slide in the event of a design basis earthquake. Tr. 10616, 10675-76, 10682-83 (Ostadan); Tr. 11904 (Bartlett).

264. There is no dispute that in the event the CTB experiences sliding, there will be no adverse safety consequences because the building is not connected to any safety-related components, such as electrical or piping lines, that may be damaged if sliding occurs. Tr. 7323-25 (Bartlett, Ostadan).

265. There will also be no adverse safety consequences due to the sliding of the pads because there are no safety-related components connected to them. Tr. 6151-52 (Trudeau); NRC Staff Testimony of Daniel J. Pomerening and Goodluck I. Ofoegbu Concerning Unified Contention Utah L/QQ, Part D (Seismic Design

And Foundation Stability) (inserted into the record after Tr. 6496) [hereinafter "Pomerening/Ofoegbu Dir."] at A11(a). In fact, as Dr. Ostadan and other witnesses testified, if pad sliding occurs, such sliding reduces significantly the seismic loading to which the cask is subjected. Tr. 7348-49, 7354, 10408 (Ostadan); Tr. 6596-97 (Ofoegbu); Tr. 6633-34 (Pomerening); Tr. 6155-56 (Trudeau); Tr. 10653, 10663-64 (Soler);. Pomerening/Ofoegbu Dir. at A11(a). Therefore, there are no adverse safety consequences due to the sliding of the pads; to the contrary, such sliding is beneficial to the stability of the safety-related component of concern; i.e., the storage casks. Tr. 6155-57, 6278-79 (Trudeau).

6. Specific State Claims re Seismic Analysis of the Storage Pads, Casks, and Their Foundation Soils[27]

a. Pad Flexibility

266. Pad flexibility is raised as a concern by the State in Sections D.1.b and D.1.c(ii) of Contention Utah L/QQ. See PFS Exh. 237 at 4. The State alleges that PFS's dynamic analyses for the pads are deficient because they fail to take into account the flexibility of the pads under dynamic loadings. The State asserts that the Applicant's calculations incorrectly assume that the pads will behave rigidly during the design basis earthquake, leading to significant underestimation of the dynamic loading atop the pads, especially in the vertical direction, and overestimation of foundation damping.

[27] The direct testimony filed by the State witnesses Dr. Steven Bartlett and Dr. Farhang Ostadan does not address the claims raised by the State in Section D of Contention Utah L/QQ in the same order as they appear in the stipulated text of the contention, and raises issues not identified explicitly in the contention. For simplicity, the discussion here follows generally the order in which the claims are presented in the direct testimony of Drs. Bartlett and Ostadan, although a cross-reference is provided in each instance to the section (if any) of Contention Utah L/QQ where the claim is raised.

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

268. Dr. Wen S. Tseng, President of International Civil Engineering Consultants, Inc. ("ICEC") testified that ICEC performed a detailed calculation for the design of the reinforced concrete pad on which the storage casks will be placed. Joint Testimony of Robert Youngs and Wen Tseng on Unified Contention Utah L/QQ (inserted into the record after Tr. 5529) [hereinafter "Youngs/Tseng Dir."] at A11; PFS Exh 85. As part of this calculation, ICEC computed the maximum displacements of the pad in the vertical direction at various nodes in the pad assuming two, four and eight casks are placed on the pad and using the lower range, best estimate and upper range estimates of the soil properties and determined that the largest such displacements were on the order of ⅜ of an inch. Youngs/Tseng Dir. at A70. These displacements included rigid displacements, that is, vertical motions of the entire pad as a rigid body. Id. at A71. When the rigid displacements are removed, the maximum deviation of local displacements from rigid body motion for the pad is of the order of approximately ⅛ of an inch. Id. at A72; Tr. 10733-39, 10754-55 (Tseng). Dr. Tseng testified that such a small local displacement would produce only secondary effects on the global dynamic response of the pad/cask system. Youngs/Tseng Dir. at A73; Rebuttal Testimony of Wen

S. Tseng on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 10727) [hereinafter "Tseng Reb."] at A1; Tr. 5662 (Tseng). Testimony by Holtec witnesses confirmed that small local displacements would not affect the stability of the casks. Singh/Soler Dir. at A78.

269. At the hearing, Dr. Ostadan asserted that what is significant is not so much the amplitude of the non-rigid displacements of a pad but the relative motion of various points on a pad with respect to each other, so that if there were a rippling effect of the pad, this would tend to decrease the radiation damping available. Tr. 7464-65, 7469-71 (Ostadan). However, a plot of vertical displacements on the pad as a function of location on the pad shows that the displacement along the pad is virtually zero for most of the length of the pad and there is one single, gradual, small vertical displacement of the pad at the point of application of the seismic loading, which slowly decreases as one moves away from the point of application of the seismic force. PFS Exh. 227; Tseng Reb. at A3–A5; Tr. 10733, 10737-39, 10755-60 (Tseng). These results show the absence of "ripples" of the type of concern to Dr. Ostadan, and demonstrate the rigid behavior of the pad under dynamic seismic loadings. Id.

270. PFS performed an evaluation of the effects of pad flexibility on the properties of the foundation, based on the methodology described in a recognized technical paper (Iguchi and Luco (1981)) and demonstrated that the effect of flexibility on the foundation damping properties of the pad is insignificant in the frequency range of importance to the cask response. PFS Exh. MM; Youngs/Tseng Dir. at A65-67; Tseng Reb. at A2; Tr. 5683-85, 10751-52 (Tseng). This result is confirmed by the computer analyses conducted by Sandia Laboratories for the NRC Staff, which incorporated pad flexibility and yielded very small cask displacements un-

der seismic loadings. Tr. 6789 (Luk). Holtec also performed, for another facility, parametric studies that compared the stability of the casks assuming a rigid versus a flexible pad and determined that the differences in the two cases were negligible. Singh/Soler Dir. at A60-61. We therefore conclude that the effects of pad flexibility on the dynamic behavior of the casks in a seismic event are negligible.

271. The State also contends that the flexibility of the pads should have been taken into account in the dynamic stability analysis of the pads. This contention is further addressed below.

b. Frequency dependence of spring and damping values

272. In Section D.1.e of the Unified Contention, the State claims that "Applicant's calculations for cask sliding do not address the frequency dependency of the spring and damping values used to model the foundation soils." PFS Exh. 237 at 4. This concern is voiced by the State witnesses in the following manner: "To be able to predict the motion of the pad and cask movement, it is important to select the appropriate soil spring and damping values. The Holtec analysis did not properly consider the frequency dependency of these parameters with respect to important frequencies of the vibration. Holtec has provided no check to compare the parameters used by other available rigorous solutions to ensure the foundation parameters are reasonably accurate. Soil springs and damping change significantly with frequency of vibration." Bartlett/Ostadan Section D Dir. at A33.

273. At the hearing, Dr. Ostadan acknowledged that the Holtec analyses have, by necessity, to be non-linear, time-dependent analyses and thus Holtec could not compute the spring and damping as a function of frequency. However, he believed that Holtec could have investigated the frequency dependency of the spring and damping and then use a peak value of damping that corresponded to the funda-

mental frequency of the pad. Tr. 7575-76 (Ostadan). Failure to do this, according to Dr. Ostadan, resulted in potentially selecting an inappropriate value of soil damping, since damping changes with frequency. Tr. 7581 (Ostadan). Dr. Ostadan, however, was unaware of the frequency to which the value of damping used by Holtec corresponded, and he did not know the extent of the error, if any, in Holtec's assumed value of soil damping. Tr. 7584-87 (Ostadan).

274. As explained by PFS witness Dr. Tseng, Dr. Ostadan's suggested method requires knowing the system frequency beforehand; however, due to the non-linear response of the casks caused by sliding and tipping, the predominant frequency of the cask/pad system's response to the seismic input is not unique, but, rather, shifts as the casks move on the pad. Therefore, the iterative solution suggested by Dr. Ostadan may not converge because, as the cask experiences motion, the predominant frequency of the system changes. Tseng Reb. at A8; Tr. 10735-36, 10752-54, 10772-76 (Tseng). Moreover, it is not appropriate to look at the frequency of the pad's response using the ICEC design calculation (as Dr. Ostadan suggests should be done), since that response is only applicable to the pad/soil system and to be accurate it would have to include the casks as well. Tr. 10773-74 (Tseng).

275. Applicant's witnesses testified that the predominant frequency of the cask/pad system's response to earthquake motions is in the range 1 – 5 Hz. Youngs/Tseng Dir. at A44-45; Singh/Soler Section D Dir. at A82). Applicant witnesses testified that while there may well be some higher order frequency contributions, their effects on the cask response will be secondary since the cask's response to the earthquake (i.e., amplitude of excursion vs. time) is primarily at or below 5 Hz. Singh/Soler Section D Dir. at A82. Thus, if the soil's spring-mass-damper model

used as the design basis input were replaced by a model involving multiple masses, springs, and dampers to incorporate effects of higher order frequency "bumps" in the spectra (if indeed, any such bumps were identified), the response of the casks would not be significantly altered. Id.

276. In its analyses, Holtec selected the soil mass, spring and damping parameters using formulae published in a well-recognized technical treatise, Newmark, N. M., and Rosenblueth, E., *Fundamentals of Earthquake Engineering.* The combination of soil mass and spring parameters produces approximate frequency-dependent foundation impedance functions that cover the frequency range important to the cask response. Use of this method, coupled with the use of three sets of soil properties (best estimate, lower bound, and upper bound) ensures that a sufficiently large range of frequencies of the cask/pad/soil system is considered. Tseng Reb. at A8.

277. In one of the analyses that Holtec performed using VisualNastran, a minimum value of damping of 1% was used. State Exh. 179; Tr. 7592-96 (Soler). Despite the reduction of the damping to such a small value, the predicted displacements of the cask during a seismic event were similar to those obtained with higher values of damping, indicating that variations in damping have relatively little impact on the behavior of the casks during a seismic event and that there are sufficient margins in the design to maintain the sliding and tipping of the casks within acceptable levels. State Exh. 179; Tr. 5768-69 (Soler); PFS Exh. OO, Case 2; Tseng Reb. at A8. Holtec also ran an analysis in which the frequency of the soil springs was tuned to the predominant frequency of the casks response to earthquake motions, and again no significant changes in the results were observed. Singh/Soler Dir. at A114-A121; Tr. 6057-6060.

278. The existence of wide margins against cask sliding and tipping has been confirmed by the analyses performed by the Sandia National Laboratories for the NRC, which did not use soil springs and dampers, but represented the soil by a detailed finite element model. NRC Staff Testimony of Vincent K. Luk and Jack Guttman Concerning Unified Contention Utah L/QQ (Geotechnical Issues) (inserted into the record after Tr. 6760) [hereinafter "Luk/Guttman Section D Dir."] at A6, A13, A16; Tr. 6835-36 (Guttman); Tr. 6979-81 (Luk); Tseng Reb. at A8; Tr. 10736 (Tseng). Thus, even assuming that the frequency dependence of the soil spring and damping was insufficiently accounted for in the Holtec analyses, such underestimation would be accommodated by the large cask-to-cask spacing at the PFSF and the large margins provided by the design against overturning and cask-to-cask impact. Singh/Soler Section D Dir. at A85.

c. Long-Term Pad Settlement

279. In their direct testimony,[28] State witnesses Bartlett and Ostadan raise a concern that "the long-term settlement of the pads has not been considered in design of the pads." Bartlett/Ostadan Section D Dir. at A29. At the hearing, this concern was described as involving not the design of the pads themselves, but the potential impact of pad settlement on the sliding of the casks on the pads due to the deformation caused by such settlement. Tr. 7382-83, 7386, 7393-94 (Ostadan). Dr. Ostadan indicated that he expects that, over the long range, the middle of the pad will settle more and the edges will settle less, deforming the pad into a concave shape. This deformation may reduce the area of contact between those casks placed on

[28] This concern does not appear to have been set forth in the State's enumeration of claims in Contention L/QQ.

the center of the pad and the surface of the pad, and may influence the rate of cask sliding during a seismic event. Tr. 7393-95 (Ostadan).

280. Pad settlement is due to three mechanisms: immediate settlement similar to elastic settlement as the pad is loaded; consolidation settlement; and long-term, creep-type settlement. State Exh. 168; Tr. 7495-96 (Bartlett). Immediate settlement is over in a matter of days; consolidation settlement occurs over a term of months, or at most a few years; creep settlement takes place over the full design life of the facility. Tr. 7495-96, 7644-46 (Bartlett).

281. The estimated total long-term settlement of the pads was computed in Stone and Webster Calculations 05996.02-G(B)-03, Rev. 3, *Estimated Static Settlement of Storage Pads,* and 05996.02-G(B)-21, Rev. 0, *Supplement to Estimated Static Settlement of Cask Storage Pads* (May 21, 2001). As explained in those calculations, the settlement of the pad is predicted based on conservative assumptions that result in an upper-bound estimate of approximately 1.75 in. for the total long-term settlement of the pads. Rebuttal Testimony of Paul J. Trudeau on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 11275) [hereinafter "Trudeau Section D Reb."] at A5); State Exh. 168.

282. Mr. Trudeau testified that, based on the conservatisms incorporated in the pad static settlement analyses, the actual long-term static settlement of the pads that can be reasonably expected to occur would be much less than the 1.75 inches that is predicted in the Stone and Webster calculations – only one fourth to one third of this estimated value, or approximately ½ inch. Trudeau Section D Reb. at A5.

283. Dr. Bartlett indicated that he did not contest the 1.75 inch total settlement reflected in the most recent PFS calculation. Tr. 11347-48 (Bartlett). Dr. Ostadan, on the other hand, testified that there would be an additional impact on the settle-

ment experienced by any one pad from the loading of other pads. Tr. 7765-73 (Ostadan). Such an impact, however, would appear to be at best a second order effect considering the distance between pads and the fact that Dr. Ostadan assumes that the pad deflection will be greatest at the center of the pads and least at the edges, where any impact of the settling of adjacent pads would be experienced. Id.

284. Dr. Bartlett criticized Mr. Trudeau's estimate of 0.5 inch total pad settlement as perhaps "sharpen[ing] a pencil too finely." State of Utah Partial Surrebuttal Testimony of Dr. Steven F. Bartlett to Rebuttal Testimony of Paul J. Trudeau on Unified Contention Utah L/QQ (inserted into the record after Tr. 11306) [hereinafter "Bartlett Section D SuReb."] at A1; Tr. 13147 (Bartlett). However, this opinion was based on his perception that settlement predictions cannot be made with such accuracy, and not on any independent settlement estimates he performed. Bartlett Section D SuReb. at A1; Tr. 11347-48 (Bartlett).

285. We note that, according to State Exh. 168, the 1.75 inches of maximum long term pad settlement is computed assuming that the entire upper layer of subsoil has the same compressibility characteristics as those of the Upper Bonneville Lake deposits, which as discussed above in connection with Section C are the weakest and most compressible soils at the PFSF site. Therefore, we conclude that, as indicated in State Exh. 168, the 1.75 inch estimate "conservatively overestimates the expected settlements." In light of this clear overestimation, we regard the 0.5 inch estimate provided by Mr. Trudeau, from which the known conservatisms have been removed, as reasonable. Such settlement levels would raise no significant stability concerns. Tr. 11125 (Mitchell).

286. The Applicant and the Staff witnesses also testified that, because of the great stiffness contrast between the concrete pad and the underlying clayey soils, the long-term settlement of the pads at the PFSF will be essentially uniform across the pad, thus its effect on the dynamic response of the pads and the casks supported on the pads should be negligible. Trudeau Section D Reb. at A5-A6; Tr. 6675-78 (Ofoegbu).

287. Dr. Bartlett disputed the assessment that the long-term settlements of the pad would be essentially uniform. Bartlett Section D Surreb. at A2. However, as Dr. Bartlett himself explained, determining the distribution of the estimated maximum settlement of a foundation is difficult and involves choosing between assuming it occurs at the center of the pads to maximize "the dishing effect," assuming is all occurs on one side of the foundation so as to produce some tilting, and distributing the total settlement over the minimum footing width, to emphasize differential settlement with adjacent structures. Tr. 11349-50 (Bartlett). However, in reality the assumption of uniform pad settlement is the only one supported by physical considerations, as pointed out by Mr. Trudeau and Dr. Ofoegbu.

288. The main consequence posited by Dr. Ostadan of the long term settlement of the pads would be altering the pattern of cask sliding on the pad by giving rise to a "dishing" effect in the middle of the pad that would make it somewhat more difficult for a cask to slide at some points and easier to slide at others. Tr. 7501-02 (Ostadan). However, assuming that there was a 0.5 inch differential settlement in the center of a pad relative to the pad's edges, the average slope measured along the short end of the pad would be only 0.159 degrees. Rebuttal Testimony of Alan I. Soler on Section D of Unified Contention Utah L/QQ (inserted into the re-

cord after Tr. 10557) [hereinafter "Soler Section D Reb."] at A8. Such a slight slope would have no significant impact on the motion of the casks. Id.

289. The State witnesses sought to distinguish this effect from the expected local variations in the coefficient of friction between the cask and the pad. Tr. 7502-06 (Ostadan, Bartlett). However, as discussed above, the cask stability analyses performed by Holtec utilized a variety of friction coefficients, including random variations in such coefficients, and in no case was a substantial amount of cask displacement observed. Therefore, it does not appear likely that the long term settlement phenomenon will induce cask motions that differ significantly from those obtained in the Holtec analyses. Soler Section D Reb. at A8.

290. Another potential consequence of long term settlement of the pads postulated by the State was a "slight inclination" or tilting of the pads. Tr. 7500, 11323 (Bartlett). However, the maximum angle of tilting of the pad resulting from such settlement would be on the order of only 0.64 degrees. Tr. 7761-63, 11349-50 (Bartlett). That level of tilting could result in effectively changing slightly the coefficient of friction between the cask and the pad. Tr. 7504 (Bartlett). Pad tilting is accounted for in the Holtec analysis and shown to have only secondary effects on the stability of the casks. Tr. 6012-14 (Soler, Singh).

291. Another potential concern raised by the State with respect to long-term settlement of the pads is that it may lead to the cracking of the soil cement layer adjacent to the pads. See, e.g., Bartlett/Mitchell Dir. at A26; Tr. 11321-24 (Bartlett). However, Dr. Mitchell testified that if the maximum differential settlement between the center of the pad and the soil cement were one half inch, this "would alleviate [his] concern a great deal." Tr. 11125 (Mitchell).

292. Ultimately, the issue with long term pad settlement is to what extent having a pad that exhibits some deformation as a result of long-term settlement will change the dynamic behavior of the casks in the event of an earthquake. Dr. Ostadan expressed the view that, in the ranking of the State's seismic concerns, long-term pad settlement would lie "somewhere between moderate to significant." Tr. 7730 (Ostadan). Dr. Ostadan emphatically stated on several occasions that he knew of no nuclear facility for which two inches or more of long term settlement was allowed, and that settlements of that magnitude were considered unacceptable by structural engineers. Tr. 7382, 7501, 7729, 7749-7750, 10396-97 (Ostadan). However, testimony presented by Applicant at the hearing established that several nuclear power plants have operated with estimated long-term static settlements of the foundations of safety-related structures in excess of 2 inches. PFS Exh. 232; Trudeau Section D Reb. at A8; Tr. 11283-85 (Trudeau); Tr. 11327 (Bartlett).

293. Indeed, no criterion for allowable static settlement is set in the NRC regulations and guidance documents; the regulatory materials provide only generic guidance regarding how static settlements should be taken into account. For example, Section 2.5.6.4, *Stability of Subsurface Materials,* in NUREG 1567, *Standard Review Plan for Spent Fuel Dry Storage Facilities*, states: "Ensure that the static analyses address settlement and lateral pressure and are accompanied by representative laboratory data." Trudeau Section D Reb. at A7. Outside the nuclear arena, the geotechnical standards set by the U.S. Army Corps of Engineers allow as much as a foot of settlement for reinforced concrete foundations supporting smoke stacks, silos, and towers. Tr. 7744-47 (Ostadan). Therefore, Dr. Ostadan's position with respect to the significance of long term pad settlements of the order of a few inches is not supported by the record.

294. Dr. Bartlett testified that, even though it was difficult to give a precise number, the change in the amount of cask sliding due to long term pad settlement would be no more than 50 to 100%. Tr. 7512 (Bartlett). Given that the maximum cask displacements estimated by PFS (and those estimated by Dr. Luk) for the design basis earthquake are only a few inches, an increase of even 50% or 100% in the sliding rate would have no adverse safety consequences.

295. Witnesses for both Applicant and the Staff testified that the anticipated long term settlement of the pads does not pose a concern in terms of the dynamic stability of the foundations and constitutes, at most, a maintenance issue. Trudeau Section D Reb. at A6; Pomerening/Ofoegbu Dir. at A11(a); State Exh. 168; Tr. 6009-6010, 6013-14 (Singh). Based on the evidence on the record, we agree.

d. Pad-to-Pad Interaction

296. Perhaps the most complex of the claims asserted by the State in Section D is that of "pad-to-pad interaction." This claim represents one of the two "overriding concerns" of the State with respect to the PFS pad stability calculation G(B)-04. Bartlett/Ostadan Section D Dir. at A35; Tr. 7599-7600 (Bartlett). The concern is described in deceptively simple terms in Section D.1.g of Contention L/QQ as follows: "The Applicant has failed to analyze for the potential of pad-to-pad interaction in its sliding analyses for pads spaced approximately five feet apart in the longitudinal direction." See PFS Exh. 237 at 4.

297. While the reference in the contention is to pad sliding, the claim was expanded in the State's direct testimony as follows: "Further, sliding failure of the pads is not a requisite condition to produce pad-to-pad interactions. Significant gapping and pounding (*i.e.*, inertial interaction) can occur without initiating sliding failure. . . . The primary concern with pad-to-pad interactions pertains to the potential transfer

of cask and pad inertial loads from one set of pads and casks to adjacent pads and casks. . . . The consequences of this transfer have been completely neglected in the sliding and stability calculations for the casks and the pads." Bartlett/Ostadan Section D Dir. at A36; see also id. at A31.

298. The State's claim in its prefiled testimony appears to relate to the effect of interaction between contiguous pads on each other, the intervening soil cement layer, and on the motion of the casks, and the initial testimony at the hearing by Dr. Bartlett and Dr. Ostadan reflects this concern:

> Well, the soil cement in the case of the pads isn't required in the design to resist sliding. So I think the more concern is the transfer of this unexpected force to the two pads, and what that does to the casks.

Tr. 7615 (Bartlett). See also, e.g., Tr. 7517-20 (Ostadan); Tr. 7605-06 (Bartlett, Ostadan).

299. Later on at the hearing, however, Drs. Bartlett and Ostadan disclaimed any direct concern about potential effect of pad-to-pad interaction on the motion of the casks, but expressed instead a concern about the effect of such interaction on the soils beneath the pad:

> Q. Well, here is my concern. I thought that all along in this proceeding our concern has been with the stability of the casks, which are the material that contains the radioactive matter. And I thought that when pad-to-pad interaction was traced by you some time ago, always the concern was pad A is going to produce a force on pad B and that could change the loading that the casks could see, could make them less stable. Why do I care how much force goes down to the soil? My interest will be how much force goes up onto the cask. Can you illuminate that?
>
> A. Yes. I think the record will speak for itself. One of the concerns has always been the effect of pad-to-pad inter-

action on the sliding analysis, and basically that means how much one pad is pushing the other. And this has not been picked up or analyzed, and now we are seeing an evidence of it.

To answer the second part of your comment as to what is the impact on the stability of the cask, I have always stayed away from that subject as not my area of expertise. I believe I have expressed a concern that if the impact or the gapping due to movement may create additional source of energy on the pad, that could also increase the motion of the cask. I think perhaps that concern has been expressed. The main concern, again, being an expert on the foundation here, has been really on the stability of the foundation.

Q. On the stability of the foundation, you mean the stability of the soil underneath the pad?

A. Yes, the pad and the soil.

Q. So your concerns are really having to do as to the extent to which the soil underneath the pad is going to remain stable; but this doesn't address directly the loads on the casks, does it?

A. Very much so. I think we've got to look at the load pad and how it comes down to the soil. Are they going to accumulate and then go down -- you see, the fact that you transfer a load from one pad to the other doesn't mean that this load disappears forever, because it goes down somewhere. Because it keeps the constant regime. The load just doesn't go away because you have a neighbor. This load has to get down to the clay a certain stage. So I think that hasn't been carefully studied on the part of PFS. It has now only shown that there is this interaction. I think there's some careful thinking needs done and a study to see what is the critical thesis and critical condition.

> Q. One more time. You just said it I think very well. The concern here is that this load is not going to disappear, it may go from pad A to pad B, but ultimately it's going to go down to the soil, isn't it?
>
> A. Right.
>
> Q. So why do we care with respect to the casks above which way the loading goes?
>
> A. Well, if you create a condition that you may have a concentration of this load for various reasons on a specific pad, you're going to also impact the motion of the pad.
>
> Q. Oh. Then what you're saying is that the effect, the potential effect that you are positing will be one in which there could be some circumstance in which a pad for some reason could accumulate these loadings from various directions. Is that what you're saying?
>
> A. Various directions and various neighboring pads, yes.

Tr. 10682-85 (Ostadan).

300. Because of the apparent shift in emphasis of the State's claim with respect to pad-to-pad interaction, it is necessary to address each of the concerns about pad-to-pad interaction separately. The issues raised by this claim can be categorized as follows:

1. Potential interaction between adjacent pads due to seismically-induced strain in the native soil beneath the cement-treated soil and the pads – with or without pad sliding.

2. Effect of interaction between pad and five-foot layer of soil cement separating the pads.

3. Effects on the underlying soils of loadings introduced through pad-to-pad interaction.

4. Significance of these effects with respect to cask stability.

i) Potential interaction between adjacent pads due to seismically-induced strain in the soil whether or not leading to pad sliding.

301. At the hearing, the State witnesses expressed the view that there can be seismically-induced interaction between adjacent pads even if the pads do not slide relative to the underlying soil. Two mechanisms were cited as potentially leading to such interaction: the weakness, deformability and potential lack in uniformity of the soils beneath the pads, and differences in the number of casks loaded in adjacent pads. Tr. 7521-26 (Bartlett, Ostadan). Both of these mechanisms were cited as leading to out-of-phase motion of adjacent pads and to potential dynamic loadings of one pad on another across the five-foot soil cement "plug" that separates them. Tr. 7521 (Ostadan).

302. Testimony by Applicant and Staff witnesses showed that the soils beneath the pad foundations are essentially uniform across the pad emplacement area and have sufficient strength to withstand the design basis earthquake loadings without experiencing significant deformation (i.e., strain). Mr. Trudeau testified that the effective shear strain in the clayey soil underlying the soil cement was only .13%. Trudeau Section D Dir. at A32; Tr. 6208-09 (Trudeau). While this strain was computed for the free-field, no significant variations in soil strain level would be anticipated if the presence of the pads and the casks was taken into account. Tr. 6210-12 (Trudeau).

303. With respect to soil strength, Dr. Bartlett referred to the Upper Lake Bonneville clays as "fairly soft" and "somewhat as a jello." Tr. 11309 (Bartlett). On further examination, however, he acknowledged that the clays have a strength in excess of 2,000 pounds per square inch and are only "soft" when compared with an adjacent soil cement layer. Tr. 11335 (Bartlett). A better description of the strength of these clays was provided by Mr. Trudeau, who stated that the Upper Lake Bonneville clays at the PFSF site are partially saturated, stiff and competent. Tr. 6278 (Trudeau).

304. Also, as discussed with respect to the Section C claims, there is remarkable uniformity of properties in the Upper Lake Bonneville clay soils across the pad emplacement area. See, e.g., Trudeau Section D Reb. at A11; Tr. 11726 (Trudeau); Tr. 11816-18 (Ofoegbu).

305. In addition, Holtec conducted an analyses in which it modeled two adjacent pads, five feet apart, one pad fully loaded with eight casks, the other having only a single cask, and included a representation of the soil cement between the pads. Soler Reb. at A2; Tr. 10560 (Soler). Holtec performed two simulations for this model: one in which the soil cement between the pads is assumed to retain its integrity and therefore be able to transmit both tension and compression forces; and another simulation in which the soil cement is assumed to be cracked and thus able to transmit only compression forces. Soler Reb. at A2; Tr. 10560-63 (Soler).

306. The configuration in these cases was set so that the potential for pad-to-pad forces was maximized. No forces were allowed to be absorbed by the soil cement; no forces were allowed to be transmitted downwards to the cement-treated soil and to the soils beneath; no damping was included in the model; a maximum value of Young's modulus for the soil cement was assumed; and no

credit was taken for the potential crushing of the soil cement by the forces going from one pad to the other. Tr. 10657, 10720-24 (Soler). Notwithstanding these very conservative assumptions intended to maximize pad-to-pad interactive forces, the maximum estimated force in the soil beneath the pads was less than the minimum required to initiate pad sliding. PFS Exh 225; Tr. 10723 (Soler). Also, while both cases predicted some interactions between the pads or between the pads and the soil cement, the forces resulting from those interactions, when added to the seismic loadings, resulted in total cask motions of the same order – inches – as had been obtained in prior simulations that had not expressly accounted for pad-to-pad interaction forces. Soler Reb. at A6; Tr. 10697-700 (Ostadan).

307. Dr. Ostadan made it clear that his concern was not with the effect of pad-to-pad interaction forces on the structural integrity of the pads or the direct effect of these forces on the casks, but only with their potential effect on the foundations. See, e.g., Tr. 10697-10700 (Ostadan). Since the results of the Holtec simulation indicate that pad-to-pad interaction forces have essentially no impact on the stability of either the pads or the storage casks, pad-to-pad interaction forces have no practical significance.

308. It is telling that Dr. Bartlett could not cite any reported instances in which the effects of foundation-to-foundation interaction without sliding were observed in an earthquake. Tr. 11310-11 (Bartlett). This confirms that the concerns raised by the State witness, which are not backed by any analysis or other objective evidence are, at most, of academic interest.

309. State witnesses testified that their concern over pad-to-pad interaction would be magnified if the pads actually were to slide. Tr. 7520 (Ostadan). However, the testimony of PFS witnesses is that the design of the cement-treated soil will pro-

vide a large margin against the potential sliding of the pads. See, e.g., Trudeau Section D Dir. at A18, A32; Trudeau Section D Reb. at A9; PFS Exh. UU. Thus, the contingency of pads actually sliding is very unlikely to materialize and pad-to-pad interaction between sliding pads is not a realistic concern.

310. The State interpreted the pad-to-pad interaction forces resulting from the two Holtec analyses referenced above as being potentially additive to those included in the PFS sliding stability calculation and resulting in making the forces acting on the pad exceed the available resisting forces and potentially induce pad sliding. Tr. 10618-21 (Ostadan). This interpretation is erroneous for two reasons. First, and most significant, the Holtec model is all-inclusive, since it accounts both for the seismic forces acting directly on the pads and for the effects of pad-to-pad interaction. Tr. 10618-20 (Soler). In addition, it would be improper to add the maximum seismic forces acting on the pad and the maximum pad-to-pad interaction forces, since they could act at different points in time and, depending on the direction of the pad motion, could be subtractive rather than additive. Id.

311. Dr. Ostadan also theorized that there could be configurations in which interaction loads from various pads could accumulate on a single pad and result in potential sliding of the pad, but indicated that without additional analysis he could not specifically postulate any. Tr. 10685-91 (Ostadan). We decline to give credit to such speculative and unsubstantiated testimony. At any rate, as we discussed above, sliding of the pads is beneficial to the stability of the casks and has no adverse safety consequences.

ii) Effect of interaction between pad and five-foot layer of soil cement separating the pads.

312. In Section D.1.c.(i) of Contention Utah L/QQ the State raises the concern that "[t]he Applicant has failed to provide a realistic evaluation of the foundation pad motion with cement-treated soil under and around the pads in relation to motion of the casks sliding on the pads" because of failure to account for "unsymmetrical loading that the soil-cement would impart on the pads once the pads undergo sliding motion." See PFS Exh. 237 at 4. In their direct testimony, Dr. Ostadan and Dr. Bartlett expressed concern about the effects of the potential impact between a pad and the adjacent soil cement "plug" in the event there is a crack or gap between the two surfaces. The concern was stated as follows: "Moreover, we have significant concerns about the separation or gapping of soil-cement from the pads during the cycling of earthquake forces. This gapping will most likely occur along preexisting shrinkage or settlement cracks or will be introduced as tensile cracks in the soil cement resulting from the bending and torsional forces introduced by the design basis earthquake. This separation and lack of tensile strength will not allow the pads and soil cement to act as an integral unit, thereby introducing out-of-phase motion and additional dynamic forces that will act alternately on the pads and on the soil-cement during earthquake cycling." Bartlett/Ostadan Section D Dir. at A36.

313. PFS testified that, by virtue of the interface strengths between the concrete pad and the underlying cement-treated soil and between the cement-treated soil and the underlying silty clay/clayey silt, the pads will be bonded to the underlying clayey soils; therefore, because pads will not slide, there will not be interaction between the pad and the soil cement frame. Trudeau Section D Reb. at A9; see also Youngs/Tseng Dir. at A80. The pads are sufficiently close in the north-south

direction that the pads and 5-ft wide soil cement plug between them will move in concert with the underlying soils when they deform due to the earthquake loading; thus, there will be no pad-to-pad interaction. Id.

314. In addition, should there be a sliding of the pads leading to a collision with the soil cement frame across a postulated gap between the two surfaces, the soil cement will tend to crush under the imparted loading because there is a significant difference between the compressive strength and modulus of elasticity of the storage pad (3000 psi and 3,120,000 psi), and the compressive strength and the dynamic modulus of the soil/cement (250 psi and 228,000 psi). Pomerening/Ofoegbu Dir. at A17. The crushing of the soil cement will limit the magnitude of the force that can be transmitted from one pad to another. Id. at A25. Because of the low magnitude of force that can be transmitted through the soil-cement layer between the storage pads, the influence on the structural integrity of the storage pads and the stability of the casks of a collision between the pad and the soil cement plug will be minor. Id.

315. In their direct testimony, PFS witnesses Dr. Singh and Dr. Soler provided an answer analogous to the Staff's, indicating that if one postulated the existence of a gap between a pad and the adjacent soil cement plug and further postulated that the pads did slide under the design basis seismic event, the closure of the soil cement to pad gap would lead to horizontal impacts not included in the current analysis; however, the impact would result in an additional energy absorption by the soil cement, resulting in minimal changes to the forces on the pad and casks. Singh/Soler Section D Dir. at A74.

316. In order to test the validity of this hypothesis, Holtec performed an analysis in which it examined the potential effect of a gap between a pad and the adjacent

soil cement layer. The analysis evaluated the impact forces that would be imparted on the pad as a result of its collision with the soil cement across the gap and the effect of those forces on the stability of the casks on the pad. For this analysis, a single pad fully loaded with eight casks was allowed to slide on the underlying soil and collide with a fixed, rigid soil cement frame surrounding the entire pad with a clearance gap of approximately 0.6 in. to all edges of the moving pad. Soler Reb. at A2; Tr. 10564-67 (Soler). The results of the Holtec analysis for this case indicate that, while there will be impacts between the pad and the surrounding soil cement, the forces produced by those impacts tend to offset the forces that would be imparted by the gradual application of compression of the pad against the soil cement, so that the net result is a reduction in the overall forces acting on the pad and the casks and a reduction by a factor of two in the displacement of the casks. Id. In short, the collision between the pad and the soil cement frame has no discernible adverse impact and, indeed, has a beneficial effect, on the stability of the casks. See PFS Exh. 225.

iii) Effects on the underlying soils of loadings introduced through pad-to-pad interaction.

317. At the hearing, Dr. Ostadan posited the existence of a new "load path" from the pads to the underlying soil due to the pad-to-pad forces predicted by Holtec. Tr. 10673-74 (Ostadan). The State witness expressed the concern that having additional lateral forces transmitted from one pad to another could have a potential effect on the stability of the soil. Tr. 10673 (Ostadan). The theory of this concern appears to be that there could be an accumulation of loads on a particular location and that, as those loads are transmitted down to the soil, they may exceed the soil's loading capacity. Tr. 10682-85 (Ostadan). This concern is wholly speculative, since the State witnesses offered no evidence that such load accumulation

will take place, its magnitude, or its potential effect on the soil given that the soil has a substantial strength, even at its weakest points. See Section C above. Moreover, to the extent that such loading results in sliding of the pads, such sliding will be beneficial, as the analyses by Holtec consistently show both without and with soil cement around the pads.

iv) Significance of issue with respect to cask stability.

318. In sum, the pad-to-pad interaction analyses conducted by Holtec predict some interactions between pads or between the pads and the surrounding soil cement, resulting in some loadings being applied to the pads. However, the forces imparted as a result of the interactions do not result in significant motions of the casks on the pads. The maximum peak-to-peak cask displacement observed in any of these cases is six inches, and the maximum cask excursion from its starting location is 3.8 inches. Soler Reb. at A6.

319. The reason for the limited effect of pad-to-pad interactions is that such interactions do not impart forces of sufficient magnitude on the pads to affect the stability of the casks on the pads. Id. at A7. The speculations by the State witnesses on how those interaction forces may alter the "load path" and cause sliding of individual pads or soil failures do not alter the fact that, as shown by the testimony pad-to-pad interaction concerns are inconsequential. Again, if the interaction results in sliding of the pads, this effect is beneficial in terms of enhancing the stability of the casks.

e. Calculation of Dynamic Forces for Pad Stability[29]

320. Dr. Ostadan also identified as an "overriding concern" with the PFS stability analysis for the storage pads contained in calculation G(B)-04 that the inertial force acting on a pad was calculated by multiplying the peak ground acceleration times the combined masses of the casks and the pad. Bartlett/Ostadan Section D Dir. at A37. Dr. Ostadan testified that use of peak ground acceleration to calculate the inertial forces was incorrect and that PFS should have instead used the response acceleration values generated by Holtec in its cask response calculation HI-2012640. Id.; Tr. 7261 (Ostadan).

321. The question raised by this concern is what the correct value of response acceleration for the pad would be and how much of an error would be introduced by using, as PFS did, the peak ground acceleration (that is, the free-field ground acceleration) as a proxy for the response acceleration of the pad. The short answer is that the horizontal response acceleration computed based on Holtec analysis would be .79g instead of the .711g used by PFS in its analyses. Trudeau Section D Reb. at A1, A4; Tr. 11278-79 (Trudeau). Use of the .79g acceleration instead of the peak ground acceleration employed by PFS would merely result in a slight decrease in the "base case" factor of safety against sliding of the pads from 1.27 to 1.22, which still provides a margin against of 22% against the potential onset of sliding. Trudeau Section D Reb. at A4.

322. PFS provided substantiation that the horizontal pad response acceleration differs little from the peak ground acceleration by showing that the radiation damping applicable to the soil/pad/cask system is so high (50% for the "best estimate" soil

[29] This claim is also not expressly set forth in the text of Contention L/QQ.

properties case) that the effects of soil-structure interaction in terms of amplifying the accelerations imparted on the pad are limited. Therefore, the response acceleration of the pad is essentially equivalent to the free field ground acceleration. Trudeau Section D Dir. at A28; PFS Exh 231; Tr. 11280 (Trudeau).

323. Dr. Ostadan indicated that he had not seen a calculation that demonstrated the existence of the 50% value of radiation damping estimated by PFS and expressed concern that such a damping level might be unrealistic. Tr. 7623 (Ostadan). He agreed, however, that if such a level of damping could be established, his concern about the difference between peak ground acceleration and the response acceleration of the pads would diminish. Tr. 7624 (Ostadan). PFS subsequently produced a calculation that substantiated the radiation damping values it used and which was not challenged by the State. See PFS Exh. 231; Tr. 11279-81, 11289 (Trudeau).

324. PFS provided still another confirmation of the appropriateness of its use of peak ground acceleration in its cask stability analysis by comparing the factor of safety against sliding of the pads it computed for its base case, 1.27, against the factor of safety that would be obtained using the time history of forces developed by Holtec in its soil-structure interaction ("SSI") analysis of the pad and casks. The use of this time history of forces at the base of the pad and casks yielded a factor of safety against sliding of 1.25, demonstrating that there is only a very slight reduction in the minimum factor of safety against sliding when these loads are used instead of computing the inertial forces of the pad and cement-treated soil based on the peak horizontal ground accelerations. Trudeau Section D Dir. at A28 – A29.

325. The State challenged the PFS calculation of the inertial force acting on the pad on two grounds. First, it sought to contrast the horizontal pad acceleration used by PFS (.711g) with the response of at the mat of the CTB from the soil-structure interaction analysis performed by PFS for that building, which is 1.047g. The State argued that the response acceleration of the pad should reflect a similar increase from the free-field value. Tr. 7626-27 (Ostadan). However, Mr. Trudeau noted that the CTB is a much taller structure than the pads, hence the soil-structure interaction effects should be more pronounced for that building than for the pads. Tr. 6192 (Trudeau).

326. Another argument raised by Dr. Ostadan to challenge the use of .711g as the horizontal acceleration of the pads was to refer to several figures in the Sandia National Laboratory report (Staff Exh. P) as "clearly show[ing] that the pad response accelerations are several times larger than the peak ground acceleration used by Stone and Webster in its stability analysis." Bartlett/Ostadan Section D Dir. at A37; Tr. 7627-30 (Bartlett, Ostadan). Dr. Ostadan, however, acknowledged that at the time he provided the testimony in A37 he had not reviewed the Sandia report in any detail. Tr. 7781, 7786, 7793, 7798 (Ostadan), so he was not aware that the figures from the report on which he relied were obtained by omitting the stiffness proportional damping and were only for a single node, and thus could not be relied upon to be a correct representation of the pad accelerations. Tr. 7788 (Bartlett, Ostadan); Staff Exh. HH; Tr. 7794-98, 7801-02, 7806 (Ostadan). On redirect, Dr. Ostadan reiterated his view that the Sandia report can be read to suggest that high pad response accelerations exist. Tr. 10342-44 (Ostadan). However, Dr. Ostadan could not reconcile his assertion that the Sandia report's high accelerations should be given credit with the inconsistent fact that the same report pre-

dicts very little displacement of the cask under such accelerations. Tr. 10427-28 (Ostadan). In light of this and the rest of the evidence on this point, we find that the State's argument based on the accelerations depicted in the Sandia report is clearly erroneous and is due no weight.

327. To summarize, nothing in the record indicates that PFS's use of the peak ground acceleration to compute the dynamic forces for pad stability is erroneous. In addition, we note that the peak acceleration, whatever its value, will be applied only at a single point in time in the entire time history; therefore, any errors in the computation of that acceleration will be absorbed by the fact that the average factor of safety against sliding is approximately 10 throughout the duration of the earthquake. Trudeau Section D Dir. at A28; PFS Exh. WW; Tr. 6272-73, 6297-99 (Trudeau).

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

f. Factors of Safety in Sliding Stability Calculation

329. As discussed above, the State witnesses raised at various times the concern that PFS has failed to demonstrate in its stability analyses that a factor of safety of 1.1 has been achieved against the sliding, bearing capacity failure and overturning of the storage pads. Drs. Bartlett and Ostadan raise this concern specifically with respect to the "simplified Newmark sliding block analysis" presented in calculation G(B)-04. Bartlett/Ostadan Section D Dir. at A38.

330. The testimony by PFS and Staff witnesses indicated that the Newmark sliding block analysis is included in the pad stability calculation for a hypothetical case in which it is conservatively assumed that the shear strength available to resist sliding at the interface between the cement-treated soil and the in situ clayey soils is based only on the frictional portion of the clay strength, completely ignoring the cohesive strength of the clay. Trudeau Section D Dir. at A30; Tr. 6327-29 (Trudeau). This hypothetical, highly conservative scenario predicted pad sliding on the order of a few inches. Id.; Tr. 6151-54 (Trudeau). The Newmark sliding block analysis was independently confirmed by a more rigorous analyses performed by Holtec, which predicted sliding on the order of several inches for this scenario. Tr. 6152-54, 6328-29 (Trudeau).

331. The State raised a number of concerns with the methodology used by PFS in its Newmark sliding block analyses. Bartlett/Ostadan Section D Dir. at A38. The testimony at the hearing, however, was to the effect that the Newmark sliding block analysis performed by PFS is conservative and, if anything, tends to overestimate the sliding displacements. Tr. 6597-6602 (Ofoegbu). In any event, Dr. Bartlett described the concerns over the PFS Newmark analysis as a "secondary issue." Tr. 7650 (Bartlett).

332. We note that the hypothetical case in which cohesionless soils are assumed to exist beneath the storage pads is not the design base case. PFS's design basis for the pads relies on the shear strength available at the interfaces between the cask storage pad and the underlying cement-treated soil and between the cement-treated soil and the underlying clayey soils. The design basis of the pads provides a conservatively calculated factor of safety against sliding that exceeds 1.1; therefore, the pads do not slide. Trudeau Section D Dir. at A30.

333. Finally, it was established at the hearing that the there are no safety-related connections in the nature of buried piping or electrical systems between the cask storage pads and the yard area or any other buildings; thus, sliding on the order of a few inches by the pads would be of no consequence, were it to occur. Pomerening/Ofoegbu Dir. at A11(a); Tr. 6151-52, 6156 (Trudeau).

g. *Non-Vertically Propagating Waves*

334. In Sections D.1.a and D.1.d of Contention L/QQ, the State asserts that the Applicant's calculations unconservatively assume that only vertically propagating in-phase waves will strike the pads, casks and foundations, and fail to account for horizontal variation of ground motion that will cause additional rocking and torsional motion in the casks, pads and foundations. PFS Exh. 237 at 3-6; Bartlett/Ostadan Section D Dir. at A42. A similar claim, raised in Section D.2.d of the contention with respect to the Canister Transfer Building, is discussed separately below.

335. PFS performed an analysis in which it computed the angle of incidence on the storage pads of earthquake waves originating from the primary sources of earthquake hazards to the PFSF, the Stansbury and East faults. The analysis utilized the physical laws governing the propagation path of seismic waves from a point source deep under the surface of the earth to a point on the surface. The propagation path obeys Snell's law at all boundaries between soil layers, such that the ratio of the sine of the angle of incidence (measured from the normal to the layered boundary) to the layer velocity is constant along the ray path ($\sin(i_i)/V_i$=constant). Youngs/Tseng Dir. at A42.

336. The analysis performed by PFS imposed Snell's Law along the travel path of the seismic waves and solved iteratively for the ray angle at the source that resulted in

a ray path that reached the surface at the site. Youngs/Tseng Dir. at A42. The analysis determined that the angle of incidence of the waves at the PFSF site would be very close to vertical, typically less than 10 degrees. Youngs/Tseng Dir. at A40; PFS Exh. LL. Thus, the proximity of the site to the major active faults does not result in high angles of incidence from vertical for earthquake waves impinging the site, and the assumption of vertically propagating waves is reasonable. Youngs/Tseng Dir. at A40.

337. PFS then calculated the difference in arrival times at opposite edges of a pad for waves having angles of incidence on the order of 10 degrees or less. The storage pads have a width of 30 ft. in the east-west direction, which is also the fault normal direction. Calculating the difference in the arrival times of earthquake waves at the east and west edges of a pad for the small angles of incidence determined by the analysis, PFS obtained differences in arrival times on the order of 0.001 to 0.002 seconds. Youngs/Tseng Dir. at A51. These time differences would only affect motions in the very high frequencies of 50 to 100 Hz, which are far above the dominant frequency range of peak cask response of 1 to 5 Hz calculated by PFS. Youngs/Tseng Dir. at A44, A51. Therefore, the rocking and torsional motions of the storage pads caused by the small angles of incidence from vertical of the seismic waves arriving at the PFSF site would be insignificant. Youngs/Tseng Dir. at A39; Pomerening/Ofoegbu Dir. at A13(a).

338. The same result would be expected from purely physical considerations. Given the small departure of the angle of incidence from vertical and the relatively small size of the pads (30 by 67 ft in plan dimensions), one would expect only very minor effects on the pad's response. The results of the PFS evaluation confirm that the small departure in the angle of incidence from vertical causes negligible ef-

fects on the response motion of the storage pads. Youngs/Tseng Dir. at A48; Pomerening/Ofoegbu Dir. at A13(a).

339. The NRC Staff independently computed the maximum bending in the storage pads due to the arrival of non-vertically propagating waves and determined that the maximum bending will occur at approximately 4 Hz and will be on the order of 0.68 inches. The maximum rocking of the storage pad will occur at 2.2 Hz, and will produce displacements of 1.16 inches. The amount of rotation will be less than 0.1 degrees. Pomerening/Ofoegbu Dir. at A13(a). Based on these reports, the Staff concluded – as did the Applicant – that the stability of the cask will not be affected by non-vertically out-of-phase seismic waves that may occur at the site. Id.

340. Seismic waves are of two principal types, body waves and surface waves. Body waves are seismic waves that travel through the body of the earth, and surface waves are waves that travel at the surface of the earth or along boundaries between layers of different velocities. Within about 50 kilometers of a large earthquake rupture, the principal source of strong ground shaking is from body waves. Surface waves have only a small contribution to the strong ground shaking in this distance range. Rebuttal Testimony of Robert R. Youngs on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 10479) [hereinafter "Youngs Reb."] at A1. For that reason, the effect of surface waves can be disregarded in the analysis of non-vertically propagating waves. Tr. 10503, 10505-06 (Youngs).

341. There are two general kinds of body waves, compression or "P" waves, and shear or "S" waves. Compression waves represent push-pull motion along the direction of wave travel and are equivalent to sound waves. They travel the highest veloci-

ties and thus are the first waves to arrive at an observation site. Shear waves represent side-to-side motion transverse to the direction of wave travel and arrive at the observation site after the P waves. Shear waves are typically the strongest source of ground shaking near to a fault. Youngs Reb. at A1. Shear waves are classified by the plane in which the particles move into "S_h" waves with particle motion in the horizontal plane and "S_v" waves with particle motion in the vertical plane. Youngs Reb. at A3.

342. Geomatrix evaluated the effects of non-vertically propagating waves on a pad's response using published work of Luco (1976) and Wong and Luco (1978). Youngs/Tseng Dir. at A53. When seismic waves strike a structure at an angle of incidence (from vertical) greater than 0, they can induce additional components of motion beyond horizontal and vertical translation (side-to-side and up-and-down motions). Inclined S_h waves tend to induce torsional motions (rotation about a vertical axis) and inclined P and S_v waves tend to introduce rocking motions (rotation about a horizontal axis). The amount of this additional motion depends on the angle of incidence and the dimensions of the structure. Youngs/Tseng Dir. at A54.

343. Studies by Luco (1976) and Wong and Luco (1978) provide evaluations of the amount of this additional motion as a function of two dimensionless parameters. The first is the normalized frequency of the foundation and represents the ratio of the foundation dimension to the wave velocity in the underlying material. The second is the ratio of the wave velocity in the underlying material to the apparent wave-passage velocity and is equivalent to the sine of the angle of incidence. Youngs/Tseng Dir. at A54.

344. Luco's 1976 work studied the effects of obliquely incident S_h waves on the torsional response of foundations. For the frequency range of 1 to 5 Hz, Geomatrix estimated the maximum angles of incidence to be 11° for 1-Hz waves and 3° for 5-Hz waves. Based on the results published in Luco's 1976 paper, Geomatrix concluded that these angles of incidence would induce a very small amount of additional torsional response of the pads, on the order of 1 to 3 percent of the amplitude of the direct horizontal translational motion. Youngs/Tseng Dir. at A54.

345. The work published in Wong and Luco's 1978 paper addresses the rocking motion induced by inclined S_V and P-waves. Based on this work, Geomatrix concluded that for the frequency range of 1 to 5 Hz, the angles of incidence of 3° to 11° would induce rocking motion on the order of 5 percent of the direct vertical motion amplitude. Youngs/Tseng Dir. at A54.

346. These analyses show that the additional rocking and torsional motion of the pad caused by inclined incident waves at the PFSF would be small compared to the motion caused by the vertically propagating waves. The calculations performed by Holtec show that there are very large margins in the range of cask movements calculated for the design basis earthquake. Any small additional motion induces by inclined waves would be insignificant and would be absorbed by these margins. Youngs/Tseng Dir. at A55.

347. Dr. Ostadan did not disagree with Dr. Young's conclusions that the departure from vertical of the angle of incidence of seismic waves arriving at the PFSF site is small, and that the difference in arrival times of the wave from one end of a pad to another is also small. Tr. 10515-16 (Ostadan). He testified, however, that if one focuses on the potential effect of a row of ten pads interacting with another adjacent row, the phase difference in the seismic loadings caused by the differ-

ence in arrival times at one row versus another could cause some interaction between the two pads. Tr. 10512-13, 10518-21 (Ostadan).

348. Dr. Ostadan's concern relates only to the hypothetical case discussed above in which the pads are assumed to slide because of the existence of cohesionless soils. This is not the design base case, for which there is no sliding of any pads. Trudeau Section D Dir. at A30. Also, as Dr. Ostadan acknowledged, the separation between two contiguous rows of pads is only a few feet, whereas the source is several miles away, under the earth's surface. Tr. 10523-24 (Ostadan). Thus, the difference in phase due to different arrival times of the seismic excitation to contiguous rows of pads will be small and the interaction between the two rows of pads is likely to be so small as to be insignificant.

349. Dr. Ostadan also disagreed with other aspects of the Applicant's analysis of the effect of non-vertically propagating seismic waves. He indicated that the PFS calculation only considered "S_h" waves, and failed to account for the contribution of other types of waves, particularly "P waves" and "S_v" waves. Tr. 7692-94 (Ostadan). He also asserted that the PFS calculation only considered energy releases initiating several kilometers down below ground, without considering the effect of waves initiating from shallower depths. Id.

350. The relative speed and amplitude of the S and P wave types can be seen in the design time histories for PFSF, which were developed from real earthquake recordings. The records contain a few seconds of low-amplitude motion at the beginning, representing the arrival of primarily P waves. After this, the strongest shaking occurs as the S or shear waves arrive. Because shear waves are the principal source of strong shaking, it is appropriate to focus the analysis on direct shear waves. Youngs Reb. at A1.

351. In response to the State's concerns, PFS performed a further analysis of the incidence of seismic waves at the PFSF for direct compression (P) waves and obtained small angles of incidence similar to those obtained for the direct shear waves. Youngs Reb. at A2; Tr. 10487 (Youngs). Therefore, the results of the original analysis would be unaffected by the inclusion of the contribution of P waves.

352. Dr. Youngs also testified that, contrary to Dr. Ostadan's assertion, the analyses he performed by PFS included the effects of S_v waves. Youngs Reb. at A3; Tr. 10485 (Youngs). He explained that when shear waves strike a boundary, some of the energy is reflected back down in what is called a reflected wave, and most of the rest is refracted and transmitted as a continuation of the direct wave. (Tr. 10484-86 (Youngs). (Upon striking a boundary, some energy may also be converted into surface waves that travel along the boundary between the two layers. As discussed above, the energy content for surface waves is small compared to body waves in the near field of the earthquake rupture as is the situation at the PFSF.) Youngs Reb. at A4.

353. When S_v shear waves strike a boundary, in addition to the reflected and refracted shear waves, some of the energy is converted into compression waves (both as reflected and refracted P waves). This process of reflection and conversion of wave types and layer boundaries is partly responsible for energy loss and scattering that occurs along the direct ray path. However, both the direct (refracted) wave ray paths and the reflected wave ray paths obey Snell's Law at the boundary, and the angles of arrival for those waves can be computed in the same manner as for S_h waves, and PFS did so in its calculation. Youngs Reb. at A4. Therefore, this concern by the State has no factual basis.

354. Dr. Youngs acknowledged the accuracy of Dr Ostadan's observation that PFS did not calculate the seismic waves originating from points at very shallow depths on the fault planes, but explained that empirical and numerical modeling of earthquake ground motions shows that little of the energy released during a seismic rupture occurs at very shallow depths. Youngs Reb. at A5. In his original analyses, Dr. Youngs calculated the angle of incidence of seismic waves emanating from depths of 5, 10, and 15 km. In response to Dr. Ostadan's concerns, he repeated the calculation of the direct shear wave for a ray path from a point at a depth of 2 kilometers on the Stansbury fault, 7.6 kilometers east of the site. For this case, the angles of incidence were 14.1 and 3.7 degrees for 1 Hz and 5 Hz waves, respectively (compared to values of 11.3 and 3.0 calculated for a point on the fault at a depth of five kilometers). For these angles of incidence and the pad dimensions, the work of Luco (1976) and Wong and Luco (1978) again shows small differences in ground motions from those represented by vertically propagating waves. Youngs Reb. at A5.

355. Based on the above discussion, the conclusion follows that the effect of earthquake motions on structures and components at the PFSF may be represented by the use of vertically propagating earthquake waves, and the effect of non-vertically propagating waves alleged by the State is insignificant. Youngs/Tseng Dir. at A58.

h. Cold Bonding

356. Section D.1.f of Contention L/QQ asserts that PFS "has failed to consider the potential for cold bonding between the cask and the pad and its effects on sliding in its calculations." PFS Exh. 237 at 4. As State witness Dr. Ostadan explained in his direct testimony, cold bonding occurs when two bodies (cask and pad) with

such a large load (the cask) are in contact. Some local deformation and redistribution of stresses may occur over many years at the points of contact, which would create a bond in the form of a welding, which increases the resistance to sliding of the cask on the pad. Bartlett/Ostadan Section D Dir. at A46.

357. The Applicant testified that the average pressure at the interface between the pad and the cask is approximately 26 psi. Singh/Soler Section D Dir. at A88. Even assuming that the entire weight of the cask was supported only over a 12" wide annulus around the periphery, the static contact pressure would rise only to 40 psi. Id. This pressure is well below the allowable bearing stress of 1785 psi in concrete with a compressive strength of 3000 psi. Pomerening/Ofoegbu Dir. at A23. Indeed, this level of pressure is comparable to a 200 lb. man standing on the ball of one foot. Singh/Soler Section D Dir. at A88. Such pressure is clearly insufficient to create a bonding between the steel bottom of the cask and the concrete surface of the pad. Id.

358. In order for cold bonding to occur, the pressure applied by the steel cask on the concrete would somehow have to increase significantly from the amount quoted above. Tr. 5895-96 (Singh). However, Applicant's witness Dr. Singh testified he could not visualize how this could occur because concrete would crush with the increasingly large pressure before it became bonded with the steel cask. Tr. 6116-17 (Singh). Thus, occurrence of cold bonding between concrete and steel is highly improbable.

359. A calculation performed by the Staff established that the initial strain in the concrete caused by the presence of the cask is 8.33 micro-inches/inch, for a total deformation of 300 micro-inches. Pomerening/Ofoegbu Dir. at A23. Long term creep accounts for an additional deformation of 672 micro-inches. Combining the

initial and creep deformations gives a total deformation of 972 micro-inches. Id. This is an insignificant amount of deformation, which will not result in cold-bonding of the cask and storage pad and will not have any influence on the overall stability of the casks under seismic load conditions. Id.; Tr. 6505-06 (Pomerening).

360. At the hearing, the State witnesses attempted to introduce a new theory that would allegedly explain cold bonding as the result of stresses imposed by individual grains of sand on contact with each other. Tr. 7708-13 (Bartlett, Ostadan). Such a theory was inconsistent with the explanation of the phenomenon previously advanced by the State in this proceeding and it would be unfair to give it any credence given the timing and manner in which it was raised. Moreover, even if bonding develops in sand through the "interlocking" of sand particles, no basis was presented for assuming that the same phenomenon would occur between two dissimilar materials (concrete and steel), particularly since the latter does not have the same granular structure and other physical and mechanical properties as concrete. Finally, we note that the expertise of Drs. Bartlett and Ostadan is in foundations and soil-structure interaction, not metallurgy, so there is no reason to give weight to their opinions on this subject.

361. The State witnesses testified that the existence of cold bonding would operate to impede the initial sliding motion of the cask under seismic loadings. Tr. 7720-21 (Ostadan). However, the seismic forces would readily break the bond and the cask would then slide on the pad in accordance with whatever coefficient of friction existed between the cask and the pad. Tr. 7722-23 (Ostadan). Therefore, assuming the cold bonding phenomenon actually took place, its effect would be very limited both in duration and effect and, as Applicant's witnesses testified,

would be subsumed in the variable coefficients of friction assumed in the Holtec analyses. Any small perturbations in the cask response due to irregular sliding would be within the range of results encompassed by the design basis simulations. Singh/Soler Section D Dir. at A90-91.

i. Need for Multiple Sets of Time Histories

362. Section D.1.h of Contention L/QQ alleges as a deficiency the fact that the PFS cask stability calculations use only one set of seismic time histories. The State claims that non-linear analyses are sensitive to the phasing of input motion and more than one set of time histories should be used, and that "fault fling" (i.e., large velocity pulses in the time history) and its variation and effects are not adequately bounded by one set of time histories. PFS Exh. 237 at 4-5. The ame concerns are expressed in the direct testimony of State witness Dr. Ostadan. Bartlett/Ostadan Section D Dir. at A45.

363. Time histories represent the variation of ground acceleration with time during an earthquake. They are used to represent the motions to which the site structures would be subject during the design earthquake. Youngs/Tseng Dir. at A22.

364. NRC guidance (Section 3.7.1 of NUREG-0800 and Section 5 of NUREG-1567) allows the designer a choice between two alternative methods for developing design time histories. One approach is to use multiple sets of time histories that in the aggregate envelop the design response spectra, although any individual time history may fall well below the design spectrum at some frequencies. The second approach is to develop a single set of time histories that envelops the design response spectra and a target power spectral density function. Time histories developed using the second approach are often called spectrum-compatible time histories. Youngs/Tseng Dir. at A24.

365. PFS elected to use the second approach, that is to utilize a single set of time histories, and its consultant Geomatrix developed a set of time histories (consisting of three independent, time histories, representing two horizontal and one vertical components of ground motion), in accordance with the methodology specified in the NRC guidance documents. Youngs/Tseng Dir. at A23. The three components of motion were then modified until their resulting response spectra enveloped the design response spectra following the criteria specified in NUREG-0800. Youngs/Tseng Dir. at A25.

366. The methodology used by PFS for developing the time histories for the stability analyses of the casks is appropriate and consistent with NRC Staff guidance. Pomerening/Ofoegbu Dir. at A27; Tr. 6507-08 (Pomerening). The response spectrum envelops the design response spectrum and encompasses the power spectral density of the design spectrum over the requisite frequency range. Id.

367. With respect to the "fault fling" issue, fault fling is a term generically used to describe enhanced ground motions that have been observed in a number of earthquake recordings obtained very near to the causative fault rupture. As an earthquake ruptures towards a site, the rupture moves at a speed that is near to that of the seismic waves radiating from the fault plane. Consequently, the seismic waves build up into a coherent, strong velocity pulse that arrives in the early portion of the strong shaking. The amplitude of low-frequency ground motions depend on the orientation of the observation point relative to the fault: motions in the direction perpendicular to the fault (fault-normal) are, on average, greater than those in the direction parallel to the fault rupture (fault-parallel). Youngs/Tseng Dir. at A92.

368. The Staff opined that it may not be necessary to consider fault fling effects for the PFSF, because the faults that may be the source of the design basis earthquake are normal faults and fault fling is a phenomenon that needs to be considered only for strike-slip faults. Pomerening/Ofoegbu Dir. at A27. In any case, the time histories developed by Geomatrix accounted for fault fling by enhancing all three components of the design response spectra for the coherent forward motion ("forward directivity") of the rupture. The east-west horizontal spectrum was then increased for fault-normal effects and the north-south component was reduced for fault parallel effects. A starting time history was also selected that exhibited a velocity pulse in the early portion of strong shaking. The recordings were then scaled upward until their response spectra enveloped the design response spectra. Through this methodology, fault fling was conservatively incorporated into the input seismic motions. Youngs/Tseng Dir. at A93.

369. The State does not challenge the appropriateness of the methodology used by Geomatrix for developing the time histories for Holtec's non-linear analyses. Rather, State witness Dr. Ostadan opined at the hearing that the industry practice, as reflected in the ASCE 4-98 standard, is to require the use of multiple time histories for non-linear analyses. Bartlett/Ostadan Section D Dir. at A45; Tr. 7674-77 (Ostadan). However, Dr. Ostadan's opinion is based of his reading implicitly into Section 3.7.1 of the Standard Review Plan the requirement, which is not stated in the SRP, that multiple sets of time histories must be used in connection with non-linear analyses. Tr. 7677-78, 7810-15 (Ostadan).

370. There are several problems with Dr. Ostadan's position. First, it does not derive support from the text of Section 3.7.1 of the SRP (Staff Exh. DD). In fact, as Dr. Ostadan acknowledged, there is nothing in that section that specifically states that

multiple time histories shall be used in cases involving non-linear analyses. Tr. 10266-69 (Ostadan). Second, Dr. Ostadan's reading of the SRP is inconsistent with that of the Staff, who has unique expertise in the interpretation of regulatory guidance. See Staff Exh. C at 5-13 – 5-14; Tr. 6507 (Pomerening). Third, Dr. Ostadan's position is inconsistent with the practice of the Staff in numerous proceedings in which Holtec has been involved: Drs. Soler and Singh testified that Holtec used a single set of time histories in its non-linear analyses in more than 40 licensing proceedings and the use of a single set was uniformly approved by the NRC Staff. Singh/Soler Section D Dir. at A102. Finally, both the Applicant and the Staff presented persuasive testimony to the effect that the use of a single time history set constructed according to the SRP 3.7.1 guidelines ensures that the time history will generate a set of appropriate, enveloping response spectra, and that the use of the single time history procedure is more likely to ensure that maximum amplitudes and proper frequency content are captured and utilized in the seismic design of the PFSF. Singh/Soler Section D Dir. at A104; Pomerening/Ofoegbu Dir. at A27.

371. In any event, the testimony by the Applicant and the Staff shows that the there is a large margin of safety against cask tip-over and/or cask-to-cask impact. While use of more or different time histories will give different response levels, the margins of safety that exist based on the current design basis results lead us to conclude that there is no merit to the State's claimed need for additional time histories, even were one to assume that multiple sets of time histories should have been used. Singh/Soler Section D Dir. at A109; Pomerening/Ofoegbu Dir. at A27; Tr. 6506-07 (Pomerening).

7. Findings of Fact on Section D.2 of Contention L/QQ re Stability Analysis of CTB

a. *Canister Transfer Building Design and Seismic Stability Analyses*

372. The State raises several concerns regarding the analyses performed by PFS of the dynamic stability of the Canister Transfer Building. PFS Exh. 237, Section D.2; Bartlett/Ostadan Section D Dir. at A40. Before discussing these concerns in detail, it is important to provide some context for the discussion. The CTB is a massive building, conservatively design to industry codes and standards that provide wide margins of safety. Ebbeson Dir. at A6 – A15. In particular, a number of conservatisms are incorporated into the design of the CTB foundations. Id. at A16. Because of these conservatisms and its physical configuration (short, squat, bottom heavy), there is no concern about potential overturning of the CTB under beyond-design basis earthquake loadings. Id.; Trudeau Section D Dir. at A38; PFS Exh. VV; Pomerening/Ofoegbu Dir. at A31(a); Tr. 6378 (Ebbeson). Nor is there any concern about bearing capacity failure of the building, since the margin of safety provided in the design is 5.5. Trudeau Section D Dir. at A39; PFS Exh. VV; Pomerening/Ofoegbu Dir. at A31(a), Tr. 6378 (Ebbeson).

373. Thus, the only failure mechanism that is being raised as potentially occurring with respect to the CTB is sliding. See Tr. 7655-56 (Bartlett); Tr. 7663, 7674 (Ostadan). Moreover, it is undisputed that such sliding, if occurring, would have no safety consequences, since there are no safety-related structures connected to the building that could be adversely affected by the sliding. Tr. 7323-25 (Bartlett, Ostadan); Trudeau Section D Dir. at A37; Ebbeson Dir. at A25; Ebbeson Reb. at A3. Therefore, the significance of the concerns raised by the State with respect to

the dynamic analyses of the CTB must be carefully assessed, even if the concerns are determined to be valid.

b. Failure of Soil Cement Buttress in Seismic Event

374. The first concern expressed by the State regarding the dynamic stability of the CTB is set forth in Section 2.c of Contention L/QQ. See PFS Exh. 237 at 5(8). The contention states that PFS has not supported its assumption that the soil cement buttress surrounding the building will provide the adequate passive resistance to sliding, because PFS has failed to demonstrate that the proposed soil cement buttress will not crack during a seismic event. Bartlett/Ostadan Section D Dir. at A40; Tr. 7651-55 (Bartlett, Ostadan). The cracking of concern to the State would take the form of continuous, non-vertical cracks. Tr. 7652-55 (Bartlett).

375. As discussed in Section C above, the cracks that can be anticipated to be formed in the soil cement surrounding the CTB are all thin, vertical, random cracks that do not affect the ability of the soil cement to provide the passive resistance to sliding relied upon in the design. Trudeau Section D Dir. at 33.

376. No new, sub-vertical cracks will be formed in the soil cement around the CTB as a result of seismically-induced bending stresses on the soil cement because those stresses will alternately open and close the tops and bottoms of any shrinkage cracks that may have occurred in the soil cement in the area. Trudeau Section D Dir. at A34.

377. The passive resistance of the soil cement will not be diminished by the presence of cracks. The effect of cracks opening as seismic waves pass through the soil-cement layer will, at most, cause the building to displace a small distance to close each crack, and then the full passive resistance of the soil cement to sliding is re-

stored. Trudeau Section D Dir. at A36. Such motions do not constitute sliding, but horizontal displacements of the soil column under the building as it strains, elastically, to reach that passive resistance required to resist sliding. Tr. 6267-68 (Trudeau).

378. The State also asserts that the CTB dynamic stability analysis performed by PFS does not address the dynamic interaction between the CTB mat foundation and the soil cement surrounding it. Bartlett/Ostadan Section D Dir. at A40. The concern is that, due to the difference in stiffness between the reinforced concrete CTB foundation mat and the surrounding soil cement and the potential out-of-phase motion of the two structures, the mat could impart loadings on the soil cement that could cause it to crack. Tr. 7654-55 (Bartlett).

379. The soil cement is strong enough to resist the horizontal loads to be applied by the CTB foundation mat and stiff enough to minimize the movement of the canister transfer building base mat against it. Tr. 6266 (Trudeau). In addition, the accelerations of the structure and the soil cement are expected to be similar in the vicinity of the structure. Thus, the loadings applied to the soil cement by the CTB mat will not be as substantial as suggested by the State. Tr. 6265 (Trudeau). Therefore, soil cement cracking is unlikely to develop through this mechanism.

c. Potential Reduction in Damping

380. Another State claim, set forth in Section D.2.b of Contention L/QQ, is that the soil cement buttress will trap some of the energy that would be dissipated through damping, thus increasing the loads to which the building will be subjected. PFS Exh. 237, at 5(8); Bartlett/Ostadan Section D Dir. at A40; Tr. 7656-60 (Ostadan). Dr. Ostadan cites a technical paper that reports that when two building foundations are in proximity, the presence of one foundation will trap the energy that

would otherwise be released to the soil by the other foundation. Tr. 7658-60 (Ostadan). However, the soil cement around the CTB is not a rigid structure like a building foundation. That soil cement will have a Young's modulus much lower than concrete, has no reinforcing steel, has no stiffening walls, and may exhibit cracking at a number of locations. Thus, the soil cement is totally unlike a building foundation and will not trap energy in the manner described by Dr. Ostadan. Ebbeson Reb. at A1; Tr. 6430 (Ebbeson); Tr. 10792-93 (Ebbeson). The soil cement acts more like regular soil and its presence may tend to increase, not diminish, radiation damping. Tr. 10794-95 (Ebbeson).

381. In addition, the main interface between the CTB and the subgrade occurs at the base of the foundation mat. Energy radiates downward and outward into the soil at this interface. The presence of a soil-cement cap around the CTB has no effect on this energy-dissipation mechanism, because it is directed downward and not in the horizontal direction. Ebbeson Dir. at A31; Tr. 6429-30 (Ebbeson).

382. Even if one were to assume that the presence of soil cement around the CTB is equivalent to having another building in the CTB's proximity, under the applicable industry code there is no need to consider structure-to-structure interaction in the dynamic analyses. Ebbeson Reb. at A1; Ebbeson Dir. at A30-31; PFS Exh. XX.

383. As a matter of fact, every nuclear power plant site has a number of buildings adjacent to each other, yet each building is typically analyzed without taking into account the potential dynamic effects of other buildings. Ebbeson Reb. at A1.

d. Mat Rigidity

384. Section 2.a of Contention L/QQ questions the assumption in the CTB dynamic stability analyses that the building's foundation mat is rigid. PFS Exh. 237 at 5(8). State witness Dr. Ostadan suggests that PFS "should have all necessary data from the structural analysis and design of the mat to make a determination on the validity of the assumption for rigidity of the mat." Bartlett/Ostadan Section D Dir. at A40; Tr. 7665 (Ostadan).

385. PFS has performed such an analysis. See PFS Exh. YY. The calculation shows that, for the loading combination with the full peak vertical earthquake acceleration acting downward and 40% of the peak accelerations acting on the two horizontal directions, the maximum variation of vertical displacement along the centerline of the building in the N-S direction is 0.163 inches over the length of 279.5 ft., which represents a less than 0.005% deflection. The maximum variation of vertical displacement in the E-W direction is .333 inches over the length of 240 ft., or about 0.01% deflection. Ebbeson Reb. at A4. Such small displacements over an area of 67,200 square feet (240 feet times 280 feet) show that the CTB basemat acts like a rigid body under earthquake loadings. Id.; Ebbeson Dir. at A24.

386. Dr. Ostadan maintains that the small displacements predicted in PFS Exh. YY can still be significant because the important consideration is not the amplitude of the displacements but how many times it occurs over the length of the structure. Tr. 7668 (Ostadan). However, it is clear from a review of PFS Exh. YY that the displacements do not take place over short distances, as Dr. Ostadan postulates, but rather over a distance of about 65 feet, and there is only one such occurrence, at the southern end of the mat; the northern end of the mat is quite rigid. Ebbeson

Reb. at A5. Thus, applying Dr. Ostadan's suggested approach of focusing on the number and distribution of displacements across the pad, the conclusion is reached that the CTB basemat is rigid. Id.

387. Treating the CTB mat as rigid is also supported by Section 3.3.1.6 of industry code ASCE 4-86, which states: "The effect of mat flexibility for mat foundations and the effect of wall flexibility for embedded walls need not be considered in the SSI analysis." See PFS Exh. XX; Tr. 6409 (Ebbeson).

388. Assuming the mat to be rigid is appropriate in view of the physical configuration of the mat (five-foot thick reinforced concrete, stiffened by shear walls connected to it), which provides the mat with significant resistance to deformation in the vertical and the horizontal directions. Ebbeson Dir. at A24; Ebbeson Reb. at A5; Tr. 6440 (Ebbeson). The assumption of mat rigidity is also consistent with the practice in the nuclear industry, which is to treat foundations for safety-related structures similar in design to the CTB at nuclear power plants as rigid. Ebbeson Reb. at A5.

e. Non-vertically Propagating Seismic Waves

389. The State's contention on the effect of non-vertically propagating waves applies both to the CTB and to the pads and has been addressed above with respect to the pads. The analysis presented there is applicable to the CTB as well, leading to the same conclusion that the effects of non-vertically propagating waves on the seismic loadings imparted on the CTB foundations are negligible. See, e.g., Pomerening/ Dir. at A33; Ebbeson Dir. at A33; Tr. 6498-6500 (Pomerening).

390. In addition, it is undisputed that to the extent there are any potential effects from non-vertically propagating waves on the stability of the CTB, PFS has addressed

such effects by incorporating into the design of the building a mass eccentricity factor of 5% to address the effects of inclined and incoherent waves. Ebbeson Dir. at A33; Tr. 6440-42 (Ebbeson). This approach is recommended in the Commentary to Section 3.3.1.2(a) of ASCE 4-86 industry code. Ebbeson Dir. at A33; PFS Exh. XX at 66; Tr. 7689 (Ostadan). By implementing this recommendation in the detailed design of the CTB, PFS avoids any need to account in the seismic analyses of the building for non-vertical propagation of seismic waves. Ebbeson Dir. at A33; Tr. 6441 (Ebbeson); Tr. 7690, 10387-88 (Ostadan).

f. Conclusions on CTB Dynamic Stability Claims

391. As noted above, all the concerns raised by the State with respect to the CTB dynamic stability analysis, if substantiated, would have the same effect: reducing the margin of safety against sliding, potentially leading to some sliding of the structure in the event of an earthquake. It is by no means clear, however, that sliding of the building would occur even under the scenarios postulated by the State. There are substantial conservatisms included in the CTB sliding stability calculation, which provide additional margins of safety against sliding. Ebbeson Reb. at A3; Tr. 10796 (Ebbeson). Because of these conservatisms, it is unlikely that the building would actually experience sliding even if the calculated factor of safety were to drop somewhat below 1.0. Ebbeson Reb. at A3; Tr. 6376-77, 6428, 6458-60, 6465-67, 10797-10801 (Ebbeson).

392. Also, as the Staff witnesses testified, it is not necessary to meet a factor of safety of 1.1 against sliding in order to satisfy the regulatory requirements of Part 72. Part 72 requires that the structures, systems and components important to safety be shown to perform their safety functions when subjected to seismic loadings. The sliding analyses performed by PFS indicate that this condition will be met,

whether or not the factor of safety recommendations in the Standard Review Plan for nuclear power plants are satisfied. Tr. 6594-96, 6739-41 (Ofoegbu). Therefore, the claims raised by the State in Section D of Contention L/QQ with respect to the dynamic stability of the CTB have no licensing significance.

C. Section E of Contention Utah L/QQ

1. Introduction and Background

393. Section E of Contention Utah L/QQ challenges the Staff's granting of an exemption from NRC regulatory requirements so as to allow PFS to design the PFSF based on a probabilistic seismic hazard analysis and a 2,000-year return period earthquake. The contention reads (PFS Exh. 237):

<u>*Section E Seismic Exemption*</u>

Relative to the PFS seismic analysis supporting its application and the PFS April 9, 1999 request for an exemption from the requirements of 10 C.F.R. § 72.102(f) to allow PFS to employ a probabilistic rather than a deterministic seismic hazards analysis, PFS should be required either to use a probabilistic methodology with a 10,000-year return period or comply with the existing deterministic analysis requirement of section 72.102(f), or, alternatively, use a return period significantly greater than 2,000 years, in that:

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

2. *PFS has failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits.*

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

4. *In supporting the grant of the exemption based on 2,000-year return period, the NRC Staff relies upon the United States Department of Energy (DOE) standard, DOE-STD-1020-94, and specifically the category-3 facility SSC performance standard that has such a return period, notwithstanding the fact the NRC Staff categorically did not adopt the four-tiered DOE category scheme as part of the Part 72 rulemaking plan.*

5. *In supporting the grant of the exemption based on the 2,000-year return period, the NRC Staff relies upon the 1998 exemption granted to DOE for the Idaho National Engineering and Environmental Laboratory (INEEL) ISFSI for the Three Mile Island, Unit 2 (TMI-2) facility fuel, which was discussed in SECY-98-071 (Apr. 8, 1998), even though that grant was based on circumstances not present with the PFS ISFSI, including (a) existing INEEL design standards for a higher risk facility at the ISFSI host site; and (b) the use of a peak design basis horizontal acceleration of 0.36 g that was higher than the 2,000-year return period value of 0.30 g.*

6. *Because (a) design levels for new Utah building construction and highway bridges are more stringent; and (b) the PFS return period is based on the twenty-year initial licensing period rather than the proposed thirty- to forty-year operating period, the 2,000-year return period for the PFS facility does not ensure an adequate level of conservatism.*

394. Applicable regulations in 10 C.F.R. § 72.102(f) and 10 C.F.R. § 72.102(b), provide for the assessment of design basis seismic ground motions for ISFSIs at sites west of the Rocky Mountains based on the deterministic procedures and criteria formerly used for nuclear power plant seismic design (Appendix A, 10 C.F.R. Part 100). In 1996 the Commission changed the seismic design requirements for new nuclear power plants by issuing regulations and guidance documents that

provide for use of Probabilistic Seismic Hazard Analysis ("PSHA") methodology. 10 C.F.R. §100.23; Regulatory Guide 1.165. The Commission is considering a similar rule change to employ the use of PSHA methodology for the seismic design of ISFSIs. See 67 Fed. Reg. 47745 (July 22, 2002).

395. SECY-98-126 (June 4, 1998), referenced in the State's contention, was the initial rulemaking plan for implementing the change from deterministic methods to PSHA methods for the seismic design of ISFSIs. That SECY document discussed three different rulemaking options for the Commission for incorporating PSHA methods into 10 C.F.R. Part 72. The "preferred" approach set forth in SECY-98-126 proposed a 1,000-year mean return period design basis earthquake for "Category 1" structures, system and components important to safety ("SSCs") (those whose failure would not result in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a)) and a 10,000-year mean return period design basis earthquake for Category 2 SCCs (those whose failure would result in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a)).

396. This initial rulemaking plan, however, was essentially superseded by SECY-01-0178, dated September 26, 2001 in which the NRC Staff recommended to the Commission that the rulemaking plan be modified to add another option, which it identified as the "preferred" one, in lieu of the two-tiered approach identified as the preferred option in in SECY-98-126. This new "preferred" option features the use of a 2,000-year mean return period earthquake as the design basis for all ISFSI SSCs. In a Staff Requirements Memorandum dated November 19, 2001, the Commission approved the modification to the rulemaking plan proposed by SECY-01-0178, further instructing the NRC Staff that the proposed rule should

solicit comments on a range of "exceedance levels" from $5.0x10^{-4}$ through $1.0x10^{-4}$ to which the failure probability of SSCs should be set.

397. On July 22, 2002, the NRC issued a proposed rule to make the Part 72 regulations compatible with the 1996 revision to Part 100 that addressed uncertainties in seismic hazard analysis. "Geological and Seismological Characteristics for Siting and Design of Dry Cask Independent Spent Fuel Storage Installations and Monitored Retrievable Storage Installations," 67 Fed. Reg. 47745 (July 22, 2002). The proposed rule would require a new specific license applicant for a dry cask storage facility located in either the western U.S. or in areas of known seismic activity in the eastern U.S., and not co-located with a NPP, to address uncertainties in seismic hazard analysis by using appropriate analyses, such as a PSHA or suitable sensitivity analyses, for determining the DBE. The new proposed regulation, 10 C.F.R. § 72.103, would eliminate the current requirement to comply with deterministic methodology of Appendix A to Part 100. As part of the proposed rule, the Commission indicated it is considering using a mean annual probability of exceedance value in the range of 5.0E-04 to 1.0E-4 for ISFSI applications. Draft Regulatory Guide DG-3021, "Site Evaluations and Determination of Design Earthquake Ground Motion for Seismic Design of Independent Spent Fuel Storage Installations and Monitored Retrievable Storage Installations," has been developed to provide guidelines that are acceptable to the NRC staff for determining the DE for an ISFSI. The Draft Regulatory Guide currently recommends a mean annual probability of exceedance value of 5.0E-04 as an appropriate risk-informed value for the design of a dry cask storage ISFSI.

398. On April 2, 1999 PFS filed an exemption request to use PSHA methods for determining the seismic design of the PFSF using a 1,000-year mean return period

earthquake as the PSHA design basis. PFS Exh 247. On August 24, 1999, PFS amended its request for an exemption to seek the use of a 2,000-year mean return period earthquake as the design basis for the PFSF. PFS Exh. 248. In its Safety Evaluation Report of October 2000 the NRC Staff approved PFS's request to use PSHA methodology for the seismic design of the PFSF based on a 2,000-year mean return period design basis earthquake. The final statement of the Staff's reasons for granting the exemption is set forth in the Consolidated SER issued in March 2002. See Staff Exhibit C at 2-50 to 2-51.

399. The State filed its contention challenging the exemption request November 9, 2000. On January 31, 2001 the Board determined that contention would largely be admissible under the Commission's standards for the admission of contentions, but referred the rulings regarding admissibility of the contention to the Commission and certified to the Commission as well the question whether the State challenges should be cognizable in this adjudicatory licensing proceeding.

400. In its decision of June 14, 2001, the Commission affirmed the Board's findings concerning the admissibility of the proffered contentions and held that the State's challenge to the exemption should be heard as part of this licensing proceeding. With respect the State's challenge to the Staff's rationale for granting the exemption, the Commission reasoned, as had the Board, that "although the contentions attacking the Staff's reasons for granting the exemption were not artfully pleaded, the substance of Utah's complaints was that the 2000-year return period has not been shown to be adequately protective." Therefore, "the contentions should not be dismissed simply because they referred to the Staff's reasoning." The Commission went on to say that, although PFS has the "burden to show that the exemption is 'authorized by law, will not endanger life or property or the common

defense or security and [is] otherwise in the public interest,'". PFS had here "essentially adopted the Staff's reasoning when it agreed to use the 2000-year return period the Staff recommended." Therefore, the Commission concluded that it was "appropriate under these circumstances to consider the Staff's bases for granting the exemption." CLI-01-12, 53 NRC at 473.

401. In its testimony and evidence before this Board, the Applicant has fully set forth the reasons why use of the 2,000-year mean return period design basis earthquake will not endanger life or property or the common defense or security, and is otherwise in the public interest. As set forth below, the Applicant's justifications provide full legal and technical bases for granting the exemption, wholly independently of the Staff's rationale, which also provides sufficient technical and legal basis for the granting of the exemption.

402. Our findings with respect to the remainder of Section E of the Unified Contention are organized as follows. First, we will discuss the appropriateness of using PSHA methods for the seismic design basis for the PFSF. Second, we will discuss whether using a 2,000 year design period earthquake in accordance with the applicable design requirements will adequately protect public health and safety. Third, we will address the State's claims concerning radiation dose consequences and the conclusions that can be drawn from them.

2. Appropriateness of Using Probabilistic Seismic Hazard Analysis Methodology for the PFSF Seismic Design

403. The parties are in agreement that use of PSHA methods is appropriate for the seismic design of the PFSF, and should be used instead of the deterministic methods currently provided for by Part 72 of the regulations. Cornell Dir. at A11-A18; PFS Exh EEE at 44-45; Tr. 9116-19 (Arabasz).

404. Deterministic methodology as applied to nuclear power plants under Appendix A of Part 100 typically leads to a small set of representative earthquakes (magnitudes and locations) that could affect a site and a corresponding set of ground motion response spectra. From these, the dominant event pair (magnitude and location) is identified, together with its representative response spectra at the site, which becomes the design basis ground motion. Cornell Dir. at A13.

405. PSHA methods differ from deterministic methods in that a PSHA takes into account the entire range of potential seismic events (magnitudes and locations) that could affect a site and resulting site ground motions, and their corresponding frequencies of occurrence and associated uncertainties. The result is a curve of estimated annual probability of exceedance versus level of ground motion. This curve can be used to identify the design ground motion corresponding to a specified mean annual probability of exceedance. Cornell Dir. at A14. In this manner, probability and risk factors are incorporated into the selection of a design basis earthquake.

406. PSHA methodology is commonly used for determining the design basis ground motions for the seismic design of building and structures, and today is the prevalent methodology in the seismic design of structures and facilities. Current regulations and guidelines based on probabilistic seismic hazard principles include those governing the design of buildings under both the Uniform Building Code ("UBC") and the International Building Code, offshore structures under API RP2A, and Department of Energy ("DOE") facilities under DOE-STD-1020. Cornell Dir. at A15.

407. The PSHA methodology has become widely accepted and used because of the advantages of using a probabilistic approach to establish design ground motions.

These advantages are: (1) the probabilistic approach captures more fully the current scientific understanding of earthquake forecasting than the deterministic method; (2) the probabilistic approach is capable of reflecting the uncertainties in professional knowledge of key elements of the seismic hazard; and (3) the probabilistic approach can be used to set design criteria that are consistent among different regions and among different failure consequences, thus allowing a rational and a equitable allocation of safety resources. Cornell Dir. at A16.

408. The Commission has recognized the advantages of the probabilistic approach and has replaced Appendix A, 10 C.F.R. Part 100 with regulations and guidance documents that provide for use of PSHA methodology for the seismic design of new nuclear power plants. 10 C.F.R. §100.23; Regulatory Guide 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," March 1997 (Staff Exh. UU). The Commission has also used probabilistic seismic procedures in areas such as re-evaluation of existing nuclear power plants and seismic standards for high-level waste geological repository design. Cornell Dir. at A17. This move towards probabilistic methodologies is consistent with the Commission's general policy of risk-informed regulations and decision making. See, e.g., Regulatory Guide 1.174, "An Approach for Using Probablistic Risk Assessment in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis," July 1998; Commission Direction Setting Issue 12, "Risk-Informed, Performance-Based Regulation". In accordance with this use of probabilistic procedures, the Commission has recently undertaken a proposed rulemaking to modify the current provisions of 10 C.F.R. § 72.102 to employ probabilistic procedures for the seismic design of ISFSIs. See

"Proposed Rule: Geological and Seismological Characteristics for Siting and Design of Dry Cask ISFSIs and MRSs," 67 Fed. Reg. 47745 (July 22, 2002).

409. Thus, PFS's proposed use of PSHA methods to characterize the seismic hazard at the site and to set the seismic design basis of the PFSF is fully consistent with NRC policy and practices, as well as with the current state of the art in engineering practice. We accordingly conclude that the use of PSHA methods for determining the design basis ground motion for the PFSF, as requested in PFS's exemption request, is warranted.

3. Appropriateness of Using a 2,000-Year Return Period Earthquake for the Seismic Design of the PFSF

410. We next turn to consider the appropriateness of using a 2,000-year return period earthquake for the seismic design of the PFSF, on which there is dispute among the parties. There are two main areas of dispute, one between PFS and the State and a second between the NRC Staff and the State. Those are discussed separately below in subsections b and c. Subsection a discusses general principles of risk informed seismic design. Subsection d discusses the specific issues raised by the State in the various subparts of Section E of the contention (other than subpart 2 concerning radiation does consequences) drawing primarily on our earlier discussion.

a. General Risk-Based Principles for Judging the Adequacy of a 2,000-Year Return Period Earthquake for the PFSF

411. The Applicant's witness, Dr. Cornell, articulated PFS's position on the appropriateness of using a 2,000-year return period earthquake for the seismic design of the PFSF based on accepted principles of risk-informed seismic design. Dr. Cornell has extensive experience in seismic risk analysis and the development of ap-

propriate seismic codes and standards. He has been involved in seismic PRAs and seismic margin studies for dozens of nuclear projects and is among the foremost experts in seismic risk assessment for nuclear facilities. Given Dr. Cornell's recognized expertise and the other parties' general agreement with the risk principles enunciated by Dr. Cornell in his testimony, we will first set forth those general risk-based principles, which we adopt.

412. The first general principle of risk-informed seismic design is that there should be a risk-graded approach to seismic safety that allows facilities and structures with lesser consequences of failure to have larger mean annual probabilities of failure than those allowed for facilities for which the consequences of failure would be more severe. In other words, under a risk-graded approach to seismic safety, the less severe the anticipated consequences of failure, the larger the probability of failure that can be tolerated. Examples of seismic standards that explicitly incorporate a risk-graded approach are the draft International Standards Organization guidelines for offshore structures, Federal Emergency Management Agency ("FEMA") guidelines for building assessment, and DOE Standard 1020. Cornell Dir. at A20-A22; Tr. 8014-18 (Cornell).

413. Such a risk-graded approach was implemented in the Staff's approval of the PFS exemption request. The Staff concluded that, because an ISFSI like the PFSF poses less radiological risk than a nuclear power plant, an ISFSI can be subjected to less stringent licensing requirements for seismic safety than those for an operating nuclear power plant. [Staff Exh. C at 2-50, 2-51] This conclusion is in accordance with the Commission's acknowledgement that the potential consequences of failure of ISFSIs are much less severe than those for nuclear power plants, and

therefore, the licensing standards for ISFSIs need not be as strict as those for operating nuclear power plants. See Cornell Dir. at A23.

414. The State's expert witness, Dr. Arabasz, agreed that it is appropriate to use a risk-graded approach for the seismic analysis and design of facilities and structures. PFS Exh. EEE at 59-60; Arabasz Dir. at A11; Tr. 9122 (Arabasz). Likewise, they agreed with Dr. Cornell and the Staff that it is appropriate to allow a higher probability of seismic failure for ISFSIs, such as the PFSF, than for nuclear power plants, since ISFSIs inherently pose less risk than an operating nuclear power plant. Tr. 9122-24 (Arabasz); Tr. 12831-32 (Bartlett) Thus, the parties are in full agreement that it is appropriate to use a risk-graded approach to seismic safety for licensing the PFSF and that under such a risk-graded approach the PFSF can be subject to less strict seismic safety requirements than those for an operating nuclear power plant.

415. The second general principle of risk-informed seismic design articulated by Dr. Cornell is that the adequacy of a design basis earthquake ("DBE") to provide the desired level of seismic safety is judged based on two considerations or factors, often referred to as the "two-handed approach." The first factor is the mean annual probability of exceedance ("MAPE") of the DBE. The second factor is the level of conservatism incorporated into the criteria and procedures for the design of the facility. Cornell Dir. at A20. Following DOE 1020 parlance, this second factor was referred to by PFS and the State as the risk reduction factor, R_R. See, e.g., id. at A27; State of Utah Testimony of Dr. Steven Bartlett on Unified Contention Utah L/QQ, Part E (Lack of Design Conservatism)(Introduced at Tr. 11822) (revised June 5, 2002) ("Bartlett Section E Dir.") at A11; Tr. 9131-36 (Arabasz); Tr. 12804-05 (Bartlett).

416. Underlying this second general principle is the fact that the design procedures and the acceptance criteria (e.g., applicable codes and standards) for seismic design usually include conservatisms that reduce the risk of failure. These conservatisms are not explicitly identified, but are embedded in the design procedures and in the provisions of the various codes and standards pursuant to which seismic design is accomplished. Because of the conservatisms incorporated in seismic design procedures and acceptance criteria, the probability of failure of a seismically-designed facility is virtually always less than the MAPE of the governing DBE. In other words, virtually all facilities designed against a given DBE have a mean return period to failure that is longer than the mean return period of the earthquake for which they are designed. In practical terms, this means that seismically-designed systems, structures and components are able to withstand a more severe, i.e., more infrequent, earthquake than that used as the DBE. Cornell Dir. at A25-A26.

417. This second principle is of great import here, for its means that the actual probability of failure of a seismically-designed facility, such as the PFSF, is a function of both the MAPE of the DBE and the level of conservatism incorporated in the design procedures and the acceptance criteria for seismic design of the facility. This function can be expressed by the simple algorithm MAPE/ R_R. Cornell Dir. at A20, A25-A26.

418. The MAPE is the inverse of the DBE. Cornell Dir. at A19; Tr. at 9145-46 (Arabasz). For example, the MAPE of the PFSF 2,000 year DBE is 5×10^{-4}. Id. Therefore, assuming that the seismic design procedures and acceptance criteria for the PFSF achieved a R_R on the order of 5, the annual probability of seismic

failure for the PFSF would be 1 x 10^{-4}, or 1 in 10,000. See, e.g., Cornell Dir. at A44 & A48; Tr. 9134, 9180-81, 10154 (Arabasz); Tr. 7925-26 (Cornell).-

419. Therefore, the actual level of seismic safety achieved by the seismic design of a facility, such as the PFSF, cannot be determined by simply looking at its DBE. Equally important, the comparative level of seismic safety of two facilities cannot be evaluated solely on the basis of their relative DBEs, unless they are also designed to the same procedures and criteria. Rather, both factors – the MAPE of the DBE as well as the level of conservatism in the design procedures and acceptance criteria – must be considered when comparing the seismic safety of two facilities or structures. Cornell Dir. at A25-A26.

420. For example, the annual probability of seismic failure for a facility or structure with a 2,500-year return period earthquake as its DBE (with a corresponding MAPE of 4 x 10^{-4}) but designed to seismic codes and standards providing a R_R of only 2 would be 2 x 10^{-4}, or 1 in 5,000. Therefore, even though the DBE of such a facility would be an earthquake of higher intensity than that for the PFSF, its annual probability of failure would be twice that for the PFSF (assuming a R_R of 5 for the PFSF seismic design) because the underlying seismic codes and standards for such a facility would embody significantly less conservatisms than those for the PFSF. See, e.g., Cornell Dir. at A91-93; Tr. 12961-63 (Cornell).

421. The State and PFS agree that DOE-STD-1020-94, "Natural Phenomena Hazards Design and Evaluation Criteria for Dept. of Energy Facilities," Jan. 2002 (PFS Exh. DDD), is a good example of the application of a risk-graded approach toward seismic design. This standard establishes a set of "performance categories" for seismically designed SSCs with increasing consequences of failure, and thus decreasing probabilities of failure, as their performance goals. DOE-1020-94 es-

tablished performance goals (reflecting increasingly severe consequences of failure) of 10^{-3} for PC-1 structures (designed to protect occupant safety) $5x10^{-4}$ for PC-2 category structures (essential facilities and buildings, such as hospitals, that should continue functioning after an earthquake with minimal interruption), and 1×10^{-4} and 1×10^{-5} for PC-3 and PC-4 category structures (which correspond to ISFSIs and NPPs respectively). The MAPE for the design basis ground motions under DOE-1020-94 were set as $2x10^{-3}$, 10^{-3}, $5x10^{-4}$, and 10^{-4} for PC-1, PC-2, PC-3 and PC-4 structures respectively.

422. To bridge the gap between the performance goals and the DBE MAPEs, DOE 1020 standards call for design procedures and acceptance criteria that vary among the categories, ranging from those "corresponding closely to model building codes" for PC1 and PC2, to those for PC4 which "approach the provisions for commercial nuclear power plants" PFS Exh DDD (DOE-STD-1020-94, p. 2-2, C-4 to C-5). The quantitative effect of applying the conservatisms built into these various design procedures and acceptance criteria is to reduce the risk reflected in the MAPE of the design basis ground motions so that it meets the corresponding performance goals.

423. The experts for both the Applicant (Dr. Cornell) and the State (Drs. Arabasz and Bartlett) "emphatically" agreed on the appropriateness of applying this two-factor, or two-handed, approach to evaluating the seismic safety of the PFSF. Tr. 9120-21, 9187-89, 9199, 10048, 10150-51 (Arabasz); Tr. 12804-05, 12859-60, 12878 (Bartlett); Tr. 8012-13 (Cornell). The NRC Staff also agreed in principle with the fact that conservatisms in the PFSF seismic design would reduce the probability of seismic failure of the PFSF to be less than the MAPE for the 2,000-year DBE, but the Staff's approach in evaluating those conservatisms, which is challenged by

the State, differed from that of PFS and the State. See Stamatakos/Chen/McCann Dir. at A25, A31; Tr. 12716-17 (Stamakatos). We turn next therefore to the different views of the parties about the application of these principles.

b. PFS-State of Utah Disputes on Adequacy of 2,000 Year DBE

i) Position of PFS and State on Adequacy of 2,000 Year DBE

424. Dr. Cornell's opinion on the adequacy of a 2,000 year DBE for the PFSF is based on two conclusions. The first conclusion is that the risk reduction factors (R_R) applicable to the SSCs important to hazardous material containment for the PFSF are 5 to 20, or greater. These R_R levels, coupled with the 2000-year ($5x10^{-4}$ MAPE) DBE imply that the PFSF SSCs will achieve a performance goal of $1x10^{-4}$ or better. Dr. Cornell's second conclusion is that $1x10^{-4}$ is an appropriate performance goal for the PFSF based on the risk-graded principles for seismic safety discussed above. Cornell Dir. at A54.

425. Dr. Cornell's conclusion that the risk reduction factors (R_R) applicable to the SSCs important to hazardous material containment for the PFSF are 5 to 20 or greater is based on his familiarity with the conservatisms embodied in nuclear codes and standards and evidence of actual conservatisms in the PFSF seismic design. Specifically, Dr. Cornell's conclusion is based on (1) his general knowledge and experience regarding risk reduction factors as applied to many different types of structures designed to a wide variety of codes and standards; (2) his general knowledge and experience of risk reduction factors applicable to nuclear power plants designed in accordance with the applicable design codes and standards as specified by the NRC NPP SRP (NUREG-0800); (3) his independent review of the SRPs applicable to ISFSIs and spent fuel storage casks (NUREGs 1567 and

1536) and confirmation that the codes and standards applicable to nuclear power plants are generally applicable to ISFSIs, such as the PFSF; (4) confirmation by those responsible for the design of the structures and components at the PFSF that such structures and components are generally designed to the same codes and standards applicable to nuclear power plants; (5) analytical and qualitative demonstration by those responsible for the design of the PFSF of significant beyond-design-basis margins for structures and components important to safety; (6) the limited fraction of time that certain SSCs are in use; (7) a showing by Holtec through analysis that casks at the PFSF will not tip over at the 10,000-year earthquake and (8) analyses by Holtec showing that a postulated cask tipover will not result in breach of a cask and release of radioactivity. Cornell Dir. at A45.

426. Dr. Cornell concluded that $1x10^{-4}$ is an appropriate performance goal for the PFSF is based on several considerations. First, the use of a probability of seismic failure or performance goal for the PFSF of $1x10^{-4}$ is consistent with the risk-graded probabilistic approach that the Commission has adopted. Second, a performance goal of $1x10^{-4}$ is consistent with DOE policy as represented by DOE-STD-1020, which provides a performance goal of $1x10^{-4}$ for ISFSIs, for facilities comparable to the PFSF. Third, a performance goal of $1x10^{-4}$ provides a lower probability of failure than the performance goals associated with even critical structures, such as bridges and hospitals. Cornell Dir. at A55; Tr. 12961-63 (Cornell).

427. The State's witnesses agreed with Dr. Cornell that $1x10^{-4}$ is an appropriate performance goal for the PFSF. PFS Exh. EEE at 80-81; Tr. 10154-55 (Arabasz); Tr. 12798-99 (Bartlett). Further, the State's witnesses agreed that if the risk reduction factors (R_R) applicable to the SSCs important to hazardous material containment

for the PFSF are 5 to 20 or more as concluded by Dr. Corenell, then the performance goal of $1x10^{-4}$ would be met. Tr. 9134, 9180-81, 10154 (Arabasz). However, Dr. Bartlett raised issues concerning the risk reduction factors available in the design of the SSCs important to hazardous material containment for the PFSF, which we discuss next.[30]

ii) Appropriate Risk Reduction Factors for Typical SSCs Designed to NRC SRPs

428. As stated, Dr. Cornell's conclusion that the risk reduction factors (R_R) applicable to the SSCs important to radioactive material containment for the PFSF are 5 to 20 or greater is based on his familiarity with the conservatisms embodied in nuclear codes and standards and evidence of actual conservatisms in the PFSF seismic design. The State acknowledges that "Dr. Cornell is a recognized expert in [the] area of evaluating conservatisms that exist in codes and standards." Tr. 10159-62 (Arabasz).

429. It is well established that the NRC guidelines on design acceptance criteria and procedures for nuclear power plants set forth in the Standard Review Plan (NUREG-0800) (Staff Exhs. CC-EE, and 64) contain many conservatisms that result in significant risk reduction factors for typical nuclear power plant components. These conservatisms are introduced through prescribed analysis methods, specification of material strengths, limits on inelastic behavior, etc. However, unlike DOE-1020, the conservatism levels in the NRC acceptance criteria guidelines are not keyed to specific risk reduction factors. Nonetheless, the risk reduc-

[30] Dr. Arabasz did not take issue with the risk reduction factors of 5 to 20 or greater that Dr. Cornell concluded exist for PFSF SSCs and indeed agrees, as set forth in the findings above, that "Dr. Cornell is a recognized expert in [the] area of evaluating conservatisms that exist in codes and standards." Tr. 10159-62 (Arabasz); see also id. at 9180.

tion factors achieved through the use of NRC guidelines for typical nuclear power plant SSCs have been found to be equal to, or higher than, the risk reduction factor of 10 for PC4 category facilities designed to DOE-STD-1020. Cornell Dir. at A30-31; PFS Exh. DD (DOE-STD-1020-94, p. 2-2, C-4 to C-5) ("[c]riteria for PC4 approach the provisions for commercial nuclear power plants").

430. The significant risk reduction factor (of 5 to 20, or more) for typical nuclear power plant SSCs was established by seismic risk analyses performed at many NPPs. Virtually all the current U.S. NPPs were designed based on Appendix A "deterministic" design basis ground motion, prior to the adoption of PSHA methodologies, and on SRP guidelines that were intentionally more conservative than, for example, corresponding building design standards. Subsequent PSHAs for these NPPs established that the Appendix A design basis ground motions had a mean return period of approximately 10,000 years. Further, numerous seismic probabilistic risk analyses ("PRAs") and seismic margin studies were also subsequently performed for SSCs at existing NPPs which established the beyond-design-basis robustness for SSCs designed to the NPP SRP. The results of these PRAs and margin studies provide the data upon which the general range of risk reduction factor values of 5 to 20 or more for typical NPP SSCs designed to the NRC's SRPs is based. These conservatisms in the design of NPP SSCs enable NPPs to achieve a performance goal of about 1×10^{-5}. Rebuttal Testimony of C. Allin Cornell to the Testimony of State Witness Dr. Walter Arabasz on Section E of Unified Contention Utah L/QQ, June 27, 2002 (Introduced at Tr. 12951) ("Cornell Reb.") at A3, following Tr. 12952-53 (Cornell); Cornell Dir. at A31-A32, A40 and Attachment A.

431. The NRC's SRPs for ISFSIs, NUREG-1567,[31] and for dry cask storage systems, NUREG-1536[32] generally provide for use of the same codes and standards employed for NPPs under NUREG-0800. By virtue of this commonality of design procedures and acceptance criteria, similar levels of conservatisms can be expected for SSCs designed to the SRPs for ISFSIs and dry storage systems as for NPP SSCs designed to NUREG-0800. Cornell Dir. at A34-A37. Additionally, those responsible for the PFSF design testified that in designing the PFSF they generally used the same design criteria and procedures applicable to nuclear power plants and applied the standards and codes applicable for nuclear components. Singh/Soler Dir. at A19 & A20; Ebbeson Dir at A7, A14; Trudeau D Dir. at A8 & A9; Young/Tseng Dir. at A30-A34. Because SSCs at the PFSF are designed following the same codes and standards as those for nuclear power plants, the conclusion that the risk reduction factors for typical systems, structures, and components designed to the NPP SRP are in the range of 5 to 20 (or greater) would apply to such structures systems and components at the PFSF. Cornell Dir. at A39.

432. Dr. Bartlett suggestsed however, that the SRPs for ISFSIs and dry storage systems "may already incorporate less design conservatism" than NUREG-0800 for NPPs, from which he argued that it would be improper to use a risk reduction factor for typical SSCs of 5 to 20 (or greater) based on their design to the SRPs for ISFSIs and dry storage systems. Bartlett Section E Dir. at A27. However, this statement was merely an expression of "concern," and not one of reasoned expert opinion.

[31] U.S. Nuclear Regulatory Commission, NUREG-1567, *Standard Review Plan for Spent Fuel Dry Storage Facilities,* March 2000). (Staff Exh. 53)

[32] U.S. Nuclear Regulatory Commission, NUREG-1536, *Standard Review Plan for Dry Cask Storage Facilities,* January 1997). (Staff Exh. 58)

Tr. 12824 (Bartlett). Unlike Dr. Cornell, who has reviewed and compared the ISFSI and dry storage SRPs against NUREG-0800 and has determined that their levels of conservatism are comparable, Dr. Bartlett has not evaluated the SRPs for ISFSIs and dry storage systems against NUREG-0800. Therefore, he could not opine on the relative conservatisms of the ISFSIs and dry storage systems SRPs compared to those in NUREG-0800. Tr. 12824-25, 12919-20, 12939-40 (Bartlett). Moreover, as stated above, the actual design of the PFSF SSCs did follow the same codes and standards as those used for nuclear power plant design. Therefore it is appropriate to use a R_R, in the range of 5 to 20 (or greater) for typical SSCs at the PFSF.

iii) The CTB Building and the Cranes and Seismic Struts therein are Typical SSCs

433. The CTB (including the building itself and the cranes and seismic struts inside the building) are typical of NPP SSCs for which the risk reduction factor has been shown to be a factor of 5 to 20 or more by the many seismic PRAs and seismic margins studies and evaluations that have been undertaken for NPPs. Cornell Dir. at A40, A48; Cornell Reb. at A3. This is sufficient to conclude that the CTB and the cranes and seismic struts inside the CTB have a risk reduction factor of five or more. Cornell Dir. at A48. The State did not take issue with the appropriateness of using a risk reduction of 5 or more for the CTB and the cranes and struts therein Tr. 9132 (Arabasz); Tr. 12786, 12814 (Bartlett).

434. In addition, the testimony of Mr. Ebbeson describes the existence of significant beyond-design-basis margins in the design of the CTB and the cranes and struts therein. Ebbeson Dir. at A20; see also Tr. 7989 (Cornell). Further, the CTB cranes and seismic struts are in use at most approximately 20% of the time and

thus a canister would be exposed to potential risk of damage due to their failure only for that fraction of the time. Lewis Dir. at A11. For such intermittent-use components, the annual likelihood of failure during a safety-important operation is further reduced 5 times, thereby effectively increasing the R_R factor for these components by a factor of 5. Cornell Dir. at A49. The testimony of Messrs. Ebbeson and Lewis provides additional direct support for the use of a risk reduction factor of five or more for the CTB and the cranes and struts therein.

iv) Appropriate Risk Reduction Factor for Foundations

435. The State did take issue with applying a risk reduction factor of 5 to 20 or more for typical NPP SSCs to the foundations for the CTB and the storage pads for potential foundation failure mechanisms i.e., sliding, loss of bearing capacity and overturning. Bartlett Section E Dir. at A--; Tr. 12785-86 Bartlett (opinions rendered in Section E testimony "limited to conservatisms for foundations" and in "the foundation design"); Id. at 12825 (Bartlett) ("no basis to disagree with Dr. Cornell['s]" conclusion that "the levels of conservatisms are the same with respect to SRPs for nuclear power plants and those for ISFSIs" other than "foundation design" issues); see also Tr. 12819-12824, (Bartlett).

436. Dr. Bartlett made two arguments to support his position that a risk reduction factor of 5 to 20 or more for typical NPP SSCs is inapplicable to the storage pad and CTB foundations. First, Dr. Bartlett asserted that the seismic PRAs and margins studies on which the 5 to 20 risk reduction factor for typical NPP SSCs is based would not have included potential soil failure mechanisms for NPP foundations. Tr. 12812-17 (Bartlett). However, Dr. Bartlett acknowledged on cross-examination that he did not know in fact whether these seismic PRAs and margins studies did or did not include potential failure due to foundation sliding, overturn-

ing and loss of bearing capacity.[33] Id. at 12817. On the other hand, Dr. Cornell testified, based on his extensive knowledge of this area, that the seismic PRAs and seismic margins studies for NPPs did in fact consider NPP foundation failure modes – such as overturning, loss of bearing capacity and sliding – and that these failure modes were not identified "as being critical failure conditions." Cornell Dir. at A41; Tr. 12952-53 (Cornell). Accordingly, it is appropriate to conclude that similar levels of conservatism have been provided by NUREG-0800 for NPP foundation as for other typical NPP SSCs and that a risk reduction factor of 5 to 20 or more is equally applicable to these foundation failure modes. Id.

437. Second, Dr. Bartlett claimed that applying the SRP factor of safety of 1.1 to a smaller earthquake level (as allowed under the PFS exemption) than that of the equivalent safe shutdown earthquake ("SSE") for NPPs reduces the absolute margin terms provided for by the 1.1 factor of safety. Tr. 12835-40 (Bartlett). However, Dr. Bartlett acknowledged that the *proportional* margins would be the same. Id. at 12840. Moreover, the actual margins provided for by the PFSF foundation design are much greater than the 10% suggested by the SRP factor of safety due to numerous conservatism in the PFSF design. Cornell Dir. at A50-51; Trudeau D Dir. at A13-A19; Trudeau Soils Reb. at A2-A3; Ebbeson Dir at A8, A9.

438. Specifically, for example, the factor of safety that PFS calculated for the storage pads against sliding was obtained by applying the following conservatisms:

[33] Of the potential foundation failure mechanisms, the one of "greatest concern" to Dr. Bartlett and the State is the potential sliding of the storage pads. Tr. 12845 (Bartlett). Dr. Bartlett would not expect "overturning of a pad foundation even for a 10,000-year return period" earthquake, and has testified that PFS's "bearing capacity analysis" for the pads for the 2,000-year return period "seems to be adequately conservative." Tr. 12845-46 (Bartlett). Similarly, Dr. Bartlett has no concerns with respect to "catastrophic potential failures of the foundations" for the CTB other than potential sliding of the building. Tr. 12849 (Bartlett). Thus, even for a 10,000 year earthquake event, the primary concern of the State is with respect to potential sliding of the foundations for the storage pads and the CTB.

- The calculated factor of safety of the pads against sliding of 1.27 in the east-west direction and 1.36 in the north-south direction did not take into account the passive resistance provided by the soil cement around the pads. Taking credit for this conservatism would increase the factor of safety from 1.27 to 3.3 in the east-west direction and from 1.36 to 2.35 in the north-south direction without taking other conservatisms into account. Trudeau D Dir. at A18-A19.[34]

- In addition, the calculation for sliding is based upon the static shear strength of the underlying clay silt soils. Trudeau Soils Reb. at A3. It is undisputed that the underlying clayey silt soils will exhibit greater strength under the dynamic loadings experienced under an earthquake of at least 30% and potentially up to 100%. Tr. 11967-68 (Trudeau); Tr. 12858, 12976-77 (Bartlett); Trudeau Dir. on Section D at A15-A16; Bartlett Soils Reb. at R3. Assuming a 50% increase in strength would increase the factor of safety for the east-west base case from 1.27 to 1.9, again without taking other conservatisms into account. Trudeau Soils Reb. at A3.

- PFS computed the minimum 1.27 and 1.36 factors of safety using the lower-bound, worst-case static shear strength for the entire pad storage area. Tr. 11960-62, 11966 (Trudeau); PFS Exh. 238. Further, this lower-bound strength was obtained from the weakest layer of soil underlying the pads whereas the pads will be resting in most cases on the soils above this layer which are much stronger than the weakest layer for which the lower bound shear strength was determined. Trudeau Soils Reb. at A3.

- Any measurement of the strength of soils will disturb the soils and result in soil strength values that are less than the actual strength that the soils will exhibit in place. Therefore, when the measured value of strength is used in the factor of safety computations, there is a "built-in" conservatism because the actual strength of the soil in place will be higher. Trudeau Soils Reb. at A3.

- The minimum factor of safety is applicable only when the earthquake reaches its peak magnitude. At all other times there is considerably more margin available. Trudeau Soils Reb. at A2-A3.

[34] The calculation with the passive resistance of the soil cement was based upon a minimum compressive strength of 250 psi. Trudeau D Dir. at A14. In fact, the compressive strength of the soil cement is likely to be greater providing more passive resistance than that calculated. Id.

- Further, due to the cyclic nature of the seismic loading each of the peak accelerations that impart dynamic loads from the earthquake exist for only one very brief moment of time – typically less than 0.005 seconds – and then the seismic loading reverses direction, which minimizes any sliding displacement that would occur. Trudeau D Dir. at A9.

439. Thus, PFS's calculation of the minimum factors of safety against pad sliding are "exceptionally conservative." Removing the various conservatisms in the calculation would result in a much greater factor of safety against pad sliding (of at least 5 for the east-west base case). Tr. 11968 (Trudeau); Trudeau Soils Reb. at A3; Trudeau D Dir. at A14-A24. Moreover, if pad sliding does occur, it reduces significantly the seismic loading to which the casks are subjected and therefore reduces the potential for radiological release. Singh/Soler Dir. at A70.

440. There is similarly a large margin against pad failure due to the loss of soil bearing capacity. The minimum factor of safety of 1.17 against bearing capacity failure for the storage pads was computed using the extremely conservative assumption that 100% of the earthquake loads act in both horizontal directions at the same time. Trudeau D Dir. at A22; Trudeau Soils Reb. at A3. If the load combinations allowed by ASCE 4-86 were used instead, the factor of safety against loss of bearing capacity would be increased to 2.1. Trudeau D Dir. at A16; Trudeau Soils Reb. at A3; <u>see also</u> Bartlett Soils Reb. at R3 (states ASCE 4-98 would increase safety factor).

441. Another major conservatism in the computation of the factor of safety against loss of bearing capacity is the use of the lower bound static shear strength of the weakest layer of soil underlying the pads. Standard practice for computing bearing capacity is to average the contributions of all soil layers over a depth equal to the shortest dimension of the foundation, in this case the 30 feet width of the pads. Approximately 2/3 of this depth below the pads would have soils or cement-

treated soils that would be much stronger than the weakest layer of soil from which the lower bound static strength was measured. Using the average strength of the cement-treated soil and soil for the 30 ft. below the pad and the soil's dynamic strength rather than its static strength would have significantly increased the factor of safety against loss of bearing capacity failure. Trudeau Soils Reb. at A3. Also, as noted with respect to pad sliding, the laboratory measured strength of the soils would be less than their in situ strength and the maximum earthquake magnitude to which the pads would be subject would be cyclic and of very short duration. Trudeau Soils Reb. at A2-A3; Trudeau D Dir. at A9.

442. Taking into account just two of the above many conservatisms (use of the load combinations allowed by ASCE 4-86 and the dynamic strength of the clayey soils) would increase the factor of safety for the pads against loss of bearing capacity to 3.63, which would provide a factor of safety of 1.0 against loss of bearing capacity for vertical and horizontal earthquake accelerations of 1.24g and 1.27g respectively, essentially the same as the 10,000 year earthquake accelerations for the PFSF site. Trudeau D Dir. at A9. Thus, as acknowledged by Dr. Bartlett, the bearing capacity analysis performed by PFS for the 2,000-year return period earthquake is "adequately conservative." Tr. 12846 (Bartlett). It provides ample margin to conclude that a risk reduction factor of more than 5 applies with respect to the pads' capability to withstand a loss of bearing capacity. Cornell Dir. at A51.

443. The factor of safety against pad overturning is 5.6, without taking into account any conservatism, Trudeau D Dir. at A23, and Dr. Bartlett acknowledged that he would not expect "overturning of a pad foundation even for a 10,000-year return period." Tr. 12846-47 (Bartlett). Thus, the margins against pad overturning are

also sufficient to conclude that a risk reduction factor of more than 5 applies with respect to pad overturning. Cornell Dir. at A51.

444. There are also numerous conservatisms included in the design of the foundations of the CTB such that, as acknowledged by Dr. Bartlett, "catastrophic" failure of the CTB due to overturning or loss of bearing capacity would not occur for a beyond-design basis earthquake event. See Tr. 12849 (Bartlett). For example, removing some of the conservatisms in the analysis results in a factor of safety against loss of bearing capacity of the CTB on the order of 10, and the 2,000-year return period earthquake accelerations would have to increase by a factor of more than four to reduce this factor of safety to 1.0. Trudeau D Dir. at A16, pages 7-8. Similarly, the CTB would not overturn during a 10,000-year earthquake event. Ebbeson Dir. at A16. Therefore, the risk reduction factors applicable to these foundation failure modes would be of 5 or more. Cornell Dir. at A50.[35]

445. Dr. Bartlett suggested at the hearing that one could not conclude that a foundation failure would not occur for a 10,000-year return period earthquake based on the margins for the 2,000-year return period DBE without performing the equivalent, formal design calculation for the 10,000 year event. Tr. 12841-42, 12874-75 (Bartlett). However, in determining the available margins associated with a DBE, such as the 2,000-year return period DBE for the PFSF, the purpose is to strip away the conservatisms and determine at what point failure would realistically occur. Therefore, it would be wholly inappropriate to require performance of a

[35] The margins against sliding of the CTB are not as large. Trudeau D Dir. at A16, page 8. But as already discussed, no negative safety consequences would result from sliding of the CTB. Tr. 7323-24 (Bartlett/Ostadan); Ebbeson Dir. at A18, A25; Cornell Dir. at A50.

10,000 year analysis using the same conservative SRP design assumptions as used for the design. Tr. 12954-56 (Cornell).

446. It is also not necessary to do a formal 10,000-year return period earthquake evaluation to show a lack of SSC failure in the event of a 10,000 year earthquake. One can determine, as reflected by the discussion above, that sufficient conservatisms exist in the design of the SSCs and their foundations to meet the increase in loadings due to the higher ground accelerations for the 10,000-year event. Indeed, if anything, the demands placed on foundations would be proportionally less for higher earthquake levels, due to the higher damping that would be associated with the higher strain levels in the soil for the 10,000-year event so that such an approach would be both appropriate and conservative. Id; see also Ebbeson Dir. at A18.

447. Therefore, risk reduction factors of five or more are appropriate for foundation failures associated with overturning, loss of bearing capacity and sliding of the storage pads. Moreover, foundation failure of the pads would not by itself constitute ultimate failure of the PFSF resulting in radioactive release, but would be part of a chain of events that one would need to analyze to determine whether the ultimate performance goal had been met. Tr. 12802-03 (Bartlett). In this respect, the record shows that the foundation failure mechanism of the pads of most concern to the State, sliding of the storage pads, would in fact reduce the loads transferred to the storage cask on the pad and reduce the likelihood of cask tipover. Singh/Soler Dir. at A70; see also Tr. 10377 (Bartlett). Similarly, the risk reduction factors for turnover and loss of bearing capacity of the CTB would be five or more, and any potential sliding of the CTB that might occur for a 10,000-year event would result in no adverse health of safety impact.

v) Appropriate Risk Reduction Factor for the Casks

448. The HI-STORM 100 cask system is designed to the SRP for dry storage systems, NUREG-1536, including SRP-dictated áccident conditions, such as hypothetical drop and tip-over events. Singh/Soler Dir. at A43. The cask and canister are not, however, "typical" NPP SSCs for which R_R factors of 5 to 20 or more have been demonstrated. Therefore, some further analysis is necessary to provide confidence that the desired performance goal for the HI-STORM 100 cask system has been achieved. Both Holtec and Sandia have performed beyond design basis analyses of the HI-STORM 100 cask system which demonstrate that the casks will not tip over during a beyond-design basis 10,000-year return period earthquake and that significant margins still remain against tipover even at the 10,000 year earthquake event. These analyses demonstrate that the effective R_R of the HI-STORM 100 cask system is in excess of 5 because the casks can survive both the 2,000 year DBE and the beyond-design-basis 10,000 year earthquake. Accordingly, the design of the HI-STORM 100 provides risks reduction factors comparable to those available for typical NPP SSCs. These demonstrations are in themselves sufficient to provide confidence that a performance goal on the order of 10^{-4} has been achieved. Cornell Dir. at A42, A52; Cornell Reb. at A3; see also Tr. 9134, 9180-81, 10154 (Arabasz); Tr. 12844-45 (Bartlett).[36]

449. Specifically, the Holtec beyond-design basis analyses showed maximum cask rotations for the 10,000-year return period earthquake event of approximately 10 to 12 degrees, still providing a factor of safety against tipover on the order of 2 to 3, as measured against the center-of-gravity over corner location of 29.3 degrees at

[36] Dr. Bartlett premised his agreement on a hypothetical basis, assuming no foundation failure and resolution of the cask stability issues raised by Dr. Khan. These issues have already been dealt with above. See Findings in Section III.B supra.

which the cask would tip over on its own accord. Further, many of the 10,000-year beyond design bases evaluations performed by Holtec assumed unrealistic, "worst-case" assumptions regarding soil damping and other factors. The demonstration under such worst-case assumptions that the casks would not tip over, with significant factors of safety still remaining, provides confidence that the casks would not tipover during even a 10,000-year earthquake event. Singh/Soler Dir. at A169; Cornell Dir. at A52; Cornell Reb. at A3; Tr. 6106-08 (Soler).

450. This conclusion is supported by the Sandia analyses which used sophisticated modeling techniques. The Sandia cask stability analyses showed cask rotations on the order of 1 degree for 10,000-year return period earthquake event, suggesting even larger margins of safety against tipover than those demonstrated by Holtec. Luk/Guttman Dir. at A16; Tr. 11661 (Luk).

451. Assuming, however, the casks were to tipover, it has been demonstrated that no breach of the confinement barrier of the canister containing the spent nuclear fuel would occur. Holtec has performed a hypothetical, non-mechanistic tipover analysis that demonstrates the decelerations at the top of the canister due to tipover would remain within the HI-STORM 100 Cask System's 45g design basis limit. Singh/Soler Dir. at A35. Moreover, as is typical of design basis limits, large conservatisms exist in this analysis.[37] In the first place, the actual g limit

[37] Dr. Bartlett expressed the opinion that a tipover under seismic earthquake conditions would have angular velocities greater than the initial zero angular velocity at the center of gravity over corner position used by Holtec in its hypothetical tipover analysis. Tr. 12870-71, 12913-15 (Bartlett). However, such analysis is beyond his area of expertise and he had done no evaluation or analysis of angular velocity during tipover. Id. at 12915. Contrary to Dr. Bartlett, Drs. Singh and Soler concluded from their evaluation of the PFSF beyond design basis analyses and other analyses they have performed that the angular velocity at impact of casks tipping over under seismic conditions would likely be less than that resulting from assuming an initial angular velocity of zero at the center of gravity over corner assumed by Holtec in its hypothetical tipover analysis because of precession of the casks prior to tipover. Singh/Soler Dir. at A170. In any event, as discussed above, large margins exist that would preclude breach of the canister's confinement boundary even if the angular velocity at impact in a tipover event

Footnote continued on next page

for the fuel cladding in the fuel assemblies is at least 63g. Additionally, there are large margins in the design of the MPC canister system that would prevent the release of radioactive material under much larger loadings. It has been demonstrated that the canister can withstand a 25 ft. straight drop, unprotected by a cask onto a hard concrete surface, maintaining confinement when subject to forces up to 300g and maintaining significant margins against reaching the failure strain limit of the material. Singh/Soler Dir. at A23; Tr. 12075 (Singh). These large margins against breach of the radioactive confinement barrier provide additional confidence that a performance objective of 10^{-4} has been met with respect to the HI-STORM 100 Cask System, since the cask will maintain containment of the radioactive matter even if tips over in a beyond-design-basis earthquake. Singh/Soler Dir. at A170-A171; Cornell Dir. at A52; Cornell Reb. at A3; see also Tr. 12075-76 (Singh).

vi) Asserted Need for Fragility Curves

452. In his pre-filed testimony, Dr. Bartlett asserted that a major deficiency in PFS's beyond-design basis analysis of the risk reduction factors based on the conservatims inherent in the PFSF design was its failure to develop fragility curves for the SSCs at the PFSF. See, e.g., Bartlett Section E Dir. at A21, A27. Fragility curves are curves that show the probability of failure of SSCs as a function of earthquake strength. Id.; see also Tr. 12794 (Bartlett). However, as explained by Dr. Cornell in his prefiled testimony, while a fragility curve can be developed to show quantitatively the value of a component's risk reduction factor, a fragility curve is not

Footnote continued from previous page

were larger than that resulting from an initial angular velocity of zero at the center of gravity over corner position.

needed to confirm that a particular component has a risk reduction factor larger than some specified level or can meet a specified seismic performance level. This can be done by various means, including analysis at the desired performance goal level to show such a goal has been met, as was accomplished by Holtec's 10,000-year beyond design basis analysis of cask stability. Cornell Dir. at A65-A66. Dr. Bartlett acknowledged on cross-examination that it was not necessary to develop fragility curves for the SSCs at the PFSF in order to determine whether the specified performance goal was met. Tr. 12852-53, 12874-75 (Bartlett). Therefore, the need for fragility curves is no longer an issue.

vii) Conclusion on State-PFS Disputes on Adequacy of 2,000-Year Return Period DBE for the PFSF

453. Based on our findings above, we conclude that the risk reduction factors, R_R, attributable to the large conservatisms inherent in the design of the SSCs for the PFSF are on the order of 5 or more, and that therefore a performance goal of 10^{-4} against potential failures that might cause radioactive release at the PFSF has been achieved. In particular, the large margins demonstrated against cask tipover and any subsequent breach of the radioactive confinement barrier, even assuming tip-over were to occur, provides great confidence that a performance goal of 10^{-4} has been achieved. The large margins against breach of the radioactive confinement boundary provide a practical answer to many of the concerns raised by the State in this proceeding. As aptly expressed by Mr. Guttman, the Staff's witness, even assuming all of the analysis done by Holtec and Sandia is erroneous and the casks do tip over there will be no significant adverse consequences, even for a 10,000 year return period earthquake. Tr. 7062-64 (Guttman). The showing that a performance goal of 10^{-4} has been achieved establishes that the overall risk from a 2,000-year return period design basis earthquake is sufficiently low that its use as

the DBE for the PFSF is consistent with Commission precedent and policy for protecting public health and safety.

c. *NRC Staff-State Disputes on Adequacy of 2,000-year MRP DBE*

454. Wholly apart from the State's allegations concerning PFS's justification for the exemption, the State raised separate issues challenging the Staff's rationale for approving the exemption. Employing a risk-graded approach, the Staff – like PFS and the State – determined that the mean annual probability of exceedance for the PFSF DBE could be greater than that for a NPP. The Staff set forth its rationale for this conclusion in the Consolidated SER as follows:

- The radiological hazard posed by a dry cask storage facility is inherently lower and the Facility is less vulnerable to earthquake-induced accidents than operating commercial nuclear power plants (Hossain et al., 1997). In its Statement of Consideration accompanying the rulemaking for 10 CFR Part 72, the NRC recognized the reduced radiological hazard associated with dry cask storage facilities and stated that the seismic design basis ground motions for these facilities need not be as high as for commercial nuclear power plants (45 FR 74597, 11/12/80; SECY-98-071; SECY-98-126).

- Seismic design for commercial nuclear power plants is based on a determination of he Safe Shutdown Earthquake ground motion. This ground motion is determined with respect to a reference probability level of 10^{-5} (median annual probability of exceedance) as estimated in a probabilistic seismic hazard analysis (Reference Reg Guide 1.165). The reference probability, which is defined in terms of the median probability of exceedance, corresponds to a mean annual probability of exceedance of 10^{-4} (Murphy et al., 1997). That is, the same design ground motion (which has a median reference probability of 10^{-5}) has a mean annual probability of exceedance of 10^{-4}. Further, analyses of nuclear power plants in the western United States show that the estimated average mean annual probability of exceeding the safe shutdown earthquake is 2.0×10^{-4} (U.S. Department of Energy, 1997).

- On the basis of the foregoing, the mean annual probability of exceedance for the PFS Facility may be defined as greater than 10^{-4} per year.

Staff Exh. C at 2-50 and 2-51. The Staff's SER also cited DOE-STD-1020-94 and the Commission's approval of an exemption authorizing a 2,000-year return period DBE for TMI-2 ISFSI as additional support for approving the PFS exemption:

- The DOE standard, DOE-TD-1020-94 (U.S. Department of energy, 1996), defines four performance categories for structures, systems, and components important to safety. The DOE standard requires that performance Category-3 facilities be designed for the ground motion that has a mean recurrence interval of 2000 yrs (equal to a mean annual probability of exceedance of 5×10^{-4}). Category-3 facilities in the DOE standard have a potential accident consequence similar to a dry spent fuel storage facility.

- The NRC has accepted a design seismic value that envelopes the 2000-yr return period probabilistic ground motion value for the TMI-2 ISFSI license (Nuclear Regulatory Commission, 1998b; Chen and Chowdhury, 1998). The TMI-2 ISFSI was designed to store spent nuclear fuel in dry storage casks similar to the PFS Facility.

Id. at 2-51. The references to DOE-STD-1020 and the TMI-2 ISFSI exemption were considered to be illustrative rather than binding precedents. For example, the Staff used the DOE-STD-1020-94 as illustrative of the acceptability of a MAPE of 5×10^{-4} under a risk-graded approach for ISFSIs, but did not adopt the standard as a regulatory criterion for use in licensing the PFSF. Likewise, the TMI-2 ISFSI – discussed in SECY-98-071 – was not referenced as establishing a regulatory criterion, but as an example of the Commission's general acceptance of PSHA methodology and principles, and of the application of risk-graded approaches to an ISFSI. Stamatakos/Chen/McCann Dir. at A14.

455. In its testimony, the Staff generally referred to the conservatisms inherent in the PFSF design that we have discussed at length above. For example, in discussing DOE-STD-1020, the Staff quoted from SECY-98-071 as follows:

> Considering the minor radiological consequences from a canister failure, and the lack of a credible mechanism to cause a failure, the staff finds that the DOE approach of using the 2000-year return period mean ground motion as the design earthquake for dry storage facilities is adequately conservative.

Stamatakos/Chen/McCann Dir. at A-25. In this respect, the Staff concluded that the HI-STORM System casks would not tip over at the PFSF even under a 10,000 year earthquake event and that, even if cask tipover occurred, no adverse consequences would result. Further, in discussing Basis 6(a) of the contention (concerning whether seismic design requirements for new Utah buildings and highway bridges are more stringent than those under the exemption granted to PFS) the Staff observed that because "SSCs important to safety at the proposed PFS Facility will be designed to NRC seismic design requirement, the resulting factors of safety and conservatism will be greater than those achieved by building codes." Id. at A31.

456. Thus, the Staff recognized the conservatisms in the design of SSCs at the PFSF that enable the SSCs to withstand earthquakes more severe than the DBE 2,000-year mean return period. The Staff did not, however, attempt to formally quantify those conservatisms or to arrive at an applicable risk reduction factor, as done by PFS.

457. The State attacks this absence of a formal determination of risk reduction and the corresponding failure to show the achievement of a specified target performance goal. See, e.g., Tr. 10145 (Arabasz). However, PFS's extensive analysis of the

conservatisms in the PFSF design and its determination of the applicable risk reduction factors would fill any void that may have existed in the Staff's rationale for approving the exemption.

458. In addition, there is no fatal flaw in the Staff's approach, even assuming that PFS had not filled this void. As discussed above, the seismic design criteria and procedures applied to the PFSF were generally the same as those applied to NPPs with which the Staff is thoroughly familiar. The NRC's seismic design criteria and procedures are recognized to contain numerous conservatisms, as reflected by the observation in DOE-STD-1020 that the risk reduction factor of 10 for PC-4 category structures "approaches the provisions for commercial nuclear power plants." The NRC Staff knows and understands the inherent margins in its seismic criteria. Indeed, its reference to DOE-STD-1020 certainly reflects the Staff's awareness of the role and importance of design conservatisms as part of the two-handed approach – explicitly endorsed in DOE-STD-1020 – and, as explained by Dr. Cornell, implicitly embodied as well in the NRC's seismic acceptance criteria.

459. Furthermore, based on its review, the Staff concluded that there were significant conservatisms in the results of the Geomatrix PSHA due to, *inter alia*, a very conservative seismic source characterization. Stamatakos/Chen/McCann Dir. at A8, A12; the Staff Exh. C, Consolidated SER (Sections 2.1.6.1 and 2.1.6.2).

460. Among the factors leading to the Staff's conclusion that the Applicant's PSHA was overly conservative were proprietary industry gravity data that indicated that the West fault near the site was not an independent seismic source as the PSHA had treated it. Stamatakos/Chen/McCann Dir. at A12; Stamatakos, et al. (1999) Staff Exh. Q. The Staff concluded that the West fault is a splay of the larger East fault, incapable of generating large magnitude earthquakes independently. In con-

trast, the Geomatrix PSHA treats the West fault as capable of producing a large magnitude earthquake, and therefore a contributor to the seismic risk at the PFSF. Stamatakos/Chen/McCann Dir. at A12.

461. The Staff also concluded that Applicant's PSHA was conservative in terms of its site-to-source distance models in the ground motion attenuation relationships, and in the development of distributions of maximum earthquake magnitude based on the dimensions of fault rupture. Stamatakos/Chen/McCann Dir. at A12. The Staff undertook an independent "slip tendency analysis," which concluded that the segments of the East fault and the East Cedar Mountain fault nearest the PFSF site have relatively low slip tendency values compared to segments farther north in Skull Valley, making the seismic source characterization in PFS's PSHA overly conservative. Stamatakos/Chen/McCann Dir. at A12; Staff Exh. C at 2-38 to 2-40).

462. The Staff also found Applicant's PSHA to be overly conservative in that it over-estimated the maximum magnitude of the East and East Cedar Mountain faults near the proposed PFSF site. The relatively low slip tendency values found by the Staff would lead to fault models with smaller rupture dimensions – and hence smaller magnitude earthquakes – than those used by PFS. Stamata-kos/Chen/McCann Dir. at A12.

463. The Staff also concluded that the PSHA results obtained by PFS are conservative by comparison of those results to other sites in Utah, especially around Salt Lake City. Despite having fault sources near Salt Lake City that are larger and more seismically active than those near the PFSF site, PFS's PSHA suggests that the seismic conditions at the PFSF site are 1.5 times more likely to produce a peak horizontal ground acceleration of 0.5g or greater than accelerations predicted for

Salt Lake City by the USGS National Earthquake Hazard Reduction Program. Stamatakos/Chen/McCann Dir. at A12; Staff Exh. Q.

464. Likewise, the 2,000-year return period horizontal peak ground acceleration for Skull Valley estimated by PFS was found to be higher than the 2,500-year ground motions for the nine sites along the Wasatch Front, which were evaluated as part of the Utah Department of Transportation I-15 Reconstruction Project. Stamatakos/Chen/McCann Dir. at A12; Staff Exh. Q. The peak horizontal ground acceleration calculated for those nine sites along the I-15 corridor ranged between 0.561g and 0.686g, based on a mean annual probability of exceedance of 4×10^{-4} (2,500-year return period). Despite the fact that the I-15 corridor sites lie close to Wasatch Fault, which has a slip rate nearly ten times that of the Stansbury or East Faults and which is capable of larger magnitude earthquakes, the PSHAs for these sites result in substantially lower ground motions than the .711g horizontal PGA calculated for the PFSF site based on a 2,000-year return period earthquake. Stamatakos/Chen/McCann Dir. at A12.

465. Thus, the Staff concluded that the results of the Applicant's PSHA could be conservative "by as much [as] 50% or more," and that this conservatism "provides an additional margin of safety in the seismic design" of the PFSF. Stamatakos/Chen/McCann Dir. at A12.

466. The State did not challenge the adequacy of the Applicant's PSHA to represent the seismic hazard at the PFSF; indeed Dr. Arabasz concluded that Geomatrix had done a "good job" with respect to the PSHA for the PFSF. Tr. 9119-20, 9965, 9970-71, 9977-78 (Arabasz). Dr. Arabasz also did not take issue with the specific conservatisms that the Staff had identified in the PSHA that Geomatrix performed for the PFSF site (although he did take issue with the comparisons that the Staff

had drawn with the earthquake hazard along the Wasatch front and the earthquake hazard at the PFSF). Tr. 9864-65, 9878-80 (Arabasz).

467. The State also took issue with testimony by the Staff that an appropriate benchmark for a NPP SSE at the PFS site would be a 5,000-year return period earthquake as opposed to a 10,000-year return period earthquake. See Tr. 10091-94 (Arabasz). The State and the Staff agree that whether a 5,000 or 10,000 year earthquake for a NPP at the PFSF site is the appropriate benchmark turns on whether the PFSF is a high seismicity site.

468. In this respect, as testified to by Dr. Stamatakos, the hazard curve produced by Geomatrix for the PFSF site is similar to the hazard curves for many high-seismicity sites along the San Andreas fault. Tr. 12753-54 (Stamatakos). From this similarity, Dr. Stamatakos concluded that if the PFSF is not a high seismicity site, the real hazard curves should not be as high as those produced by Geomatrix, from which it would follow that the PFS facility has been designed to a significantly higher return period than the 2,000 year return period ground motions obtained from the Geomatrix PSHA hazard curves. Tr. 12754 (Stamatakos). If that were the case, the design basis ground motions obtained from the Geomatrix PSHA would be overly conservative.

469. On the other hand, Dr. Stamatakos testified that if the hazard curve produced by Geomatrix accurately reflects the conditions at the PFSF, and the 2,000 year return period earthquake has a horizontal acceleration in excess of 0.7g, then such a high ground acceleration for a 2,000 year return period earthquake would by definition classify the PFSF as a high seismicity site, and it would be appropriate to use a 5,000-year mean return period earthquake as the NPP SSE benchmark. Tr. 12754 (Stamatakos).

470. We do not need to resolve the dispute on whether the PFSF is a high seismicity site such that the appropriate benchmark NNP SSE would be 5,000 years. We note that, although the Staff testified to a 5,000-year NPP benchmark at the hearing, the SER it only concludes that, because the PFSF's risk is lower than that of a NPP, the PFSF may have a design basis earthquake that has a mean annual probability of exceedance greater than 1 x 10^{-4}. The 2,000-year DBE selected for the PFSF design is consistent with the Staff's determination.

471. Further, we note that the Staff has identified what it considers to be many conservatisms in the Geomatrix PSHA. Therefore, the 2,000-year DBE constitutes a conservative prediction of the seismic hazard at the PFSF. This conservatism is above and beyond the inherent conservatisms embodied in the PFSF design, and provides additional confidence that the 2,000-year DBF for the PFSF provides sufficient protection of the public health and safety.

d. *Specific Issues Raised in Subparts of Section E Other than Radiological Dose Consequences*

472. The State raised six specific bases to support what is now Section E of Contention L/QQ. Basis 2 concerns radiation dose consequences, andis discussed Section III.C.4 below. The remaining bases are addressed specifically here.

i) Section E, Basis 1

473. In Basis 1, the State challenged the exemption granted by the NRC Staff to PFS authorizing the use of a 2,000-year return period DBE for the PFSF on the grounds that the exemption failed to conform to the rulemaking plan set forth in SECY-98-126 (June 4, 1998). That SECY discussed three different rulemaking options for the Commission to incorporate PSHA methods into 10 C.F.R. Part 72, with the "preferred" approach being a 1000-year mean return period design basis

earthquake for Category 1 SSCs (those whose failure would result in radiological doses less than the dose limits specified in 10 C.F.R. § 72.104(a)) and a 10,000-year mean return period design basis earthquake for Category 2 SCCs whose failure would result in radiological doses exceeding the dose limits of 10 C.F.R. § 72.104(a). Cornell Dir. at A74.

474. The two-tiered approach set forth in SECY-98-126 is, however, no longer the Commission's preferred approach. In SECY-01-0178, dated September 26, 2001, the NRC Staff recommended to the Commission that the rulemaking plan be modified to add a fourth option. This fourth alternative proposed, as a new "preferred" option, the use of a single 2,000-year mean return period earthquake as the design basis for all ISFSI SSCs. This is the same DBE proposed by PFS in its exemption request. In a Staff Requirements Memorandum dated November 19, 2001, the Commission approved the modification to the rulemaking plan proposed by SECY-01-0178, further instructing the NRC Staff that the proposed rule should solicit comment on a range of exceedance levels from 5.0E-04 through 1.0E-04. Cornell Dir. at A75. Thus the PFSF proposed exemption conforms with this new preferred methodology, rendering the State's concern in Basis 1 obsolete. Cornell Dir. at A76-A77.

475. Furthermore, in admitting Basis 1, both the Commission and the Licensing Board expressly held that PFS was not bound by the rulemaking plan, and that the ultimate issue to be determined is whether the 2000-year design standard is sufficiently protective of public safety and property.

ii) Section E, Basis 2

476. Basis 2 of Section E of Utah L/QQ is discussed in Section III.C.4, below.

iii) Section E, Bases 3-5

477. Bases 3-5 of Section E concern specific issues raised by the State with respect to the logic used by the Staff in approving PFS's exemption request, which have been carried forward as part this proceeding. Tr. at 9158-63 (Arabasz).[38] These bases do not concern whether the PFSF design is sufficiently conservative to withstand an earthquake with a mean return period on the order of 10,000 years discussed in Section III.C.3(b) above. Tr. 9163-64 (Arabasz). Therefore, they do not challenge the justification put forth by PFS for the use of the 2,000 year design basis earthquake.

(a) Section E, Basis 3

478. The claim raised in Basis 3 is that the NRC Staff's "reliance on the reduced radiological hazard of stand-alone ISFSIs as compared to commercial power reactors as justification for granting the PFS exemption is based on incorrect factual and technical assumptions" concerning the relationship between mean and median probabilities for NPP safe shutdown earthquakes ("SSE"). This issue, however, has evolved significantly from the original contention, and indeed even from the pre-filed testimonies. As phrased in both Dr. Arabasz's pre-filed testimony, the issue has metamorphosized into what is the NPP "benchmark" against which to judge the adequacy of the DBE for the PFSF in applying a risk-graded approach. Arabasz Dir. at A10; see also,Cornell Dir. at A83.

479. There appears to be no dispute between PFS and the State on the appropriate NPP SSE benchmark to judge the adequacy of the DBE for the PFSF. Dr. Arabasz in his initial oral testimony in May 2002 stated his belief that the mean annual prob-

[38] Dr. Arabasz was the author of Bases 3-5, in that he provided the technical input for these bases. Tr. at 9115 (Arabasz).

ability of exceedance for a NPP SSE at the PFS site would probably be $1x10^{-4}$. Therefore, Dr. Arabasz concluded that "it would be appropriate in applying the risk graded approach in determining the appropriate design basis [earthquake] for the PFSF" to compare it to a NPP SSE with a mean annual probability of exceedance of $1x10^{-4}$ – i.e., an earthquake with a 10,000 year mean return period. Tr. 9176-79 (Arabasz); see also id. at 9207-08. Dr. Arabasz subsequently reconfirmed his belief that the appropriate NPP benchmark against which to compare a risk graded design basis ground motion for the PFSF would be an earthquake with a 10,000 year mean return period (or a mean annual probability of exceedance of $1x10^{-4}$). Tr. 10124 (Arabasz). This is the same benchmark that Dr. Cornell would use in applying the risk-graded approach to the PFSF. Cornell Dir. at A83. On the other hand, the State does dispute the Staff's use of a 5,000 year mean return period earthquake as the benchmark NPP earthquake against which to judge the adequacy of the PFSF 2,000 year design basis earthquake. However, because we find that PFS has established the sufficiency of a 2,000 year design basis earthquake for the PFSF when judged against a NPP benchmark SSE earthquake of 10,000 years, we do need not resolve the question of whether it would be appropriate to use a lower mean return period earthquake as the applicable NPP SSE benchmark.

(b) Section E, Basis 4

480. In Basis 4, the State challenges the NRC Staff's reliance on DOE-STD-1020 as support for its approval of the exemption. Specifically, the State claims that the Staff inappropriately relied upon DOE-STD-1020 as support for use of a 2,000 year design basis earthquake because it did not couple this design basis earthquake with a target performance goal achieved by conservatisms embodied in the

design acceptance criteria, as called for by DOE 1020. Tr. 9160-61, 9179 (Arabasz). In this respect, Dr. Arabasz acknowledged that if the conservatisms set forth in Dr. Cornell's testimony were "shown to exist," then PFS would have established "a target performance level equivalent to a PC-3 category" structure under DOE-1020. Tr. 9179-81 (Arabasz). Those conservatisms have been "shown to exist." As discussed above, the NRC's SRPs implicitly embody conservatisms that are equal to or greater than those provided for by DOE 1020. In addition, PFS has shown that the PFSF design achieves a performance goal on the order of $1x10^{-4}$, equivalent to the goal for ISFSIs under DOE-STD-1020 (which are classified as category PC-3 structures under DOE-1020). Therefore, the analysis set forth in Dr. Cornell's testimony, which fully embraces the two-handed approach embodied in DOE-1020, addresses and resolves the State's concern raised in Basis 4.

481. While the DBE for category PC3 structures under DOE-1020 has recently been changed from 2,000 years to 2,500 years, the level of conservatism in the applicable design procedures and criteria provided for by DOE 1020 was reduced such that the performance goal for PC3 structures remains unchanged at $1x10^{-4}$. Cornell Dir. at A86-A87; Tr. 9305-06 (Arabasz). Thus, the PFSF would continue to achieve a target performance goal equivalent to that for PC3 structures under DOE 1020. Id.

(c) Section E, Basis 5

482. In Basis 5, the State challenges the NRC Staff's reliance on the 1998 exemption granted to DOE for the Idaho National Engineering and Environmental Laboratory ("INEEL") ISFSI for the Three Mile Island, Unit 2 ("TMI-2") spent facility fuel as support for granting the PFSF exemption. The State claims that the NRC's

reliance on the INEEL exemption is misplaced because the grant of the exemption there was based on circumstances not present with respect to the PFSF.

483. As acknowledged by Dr. Arabasz, however, the potential precedential value of the INEEL exemption does not directly affect the substantive issue of whether PFS has shown sufficient basis to justify its proposed 2,000 year design basis earthquake. See Tr. 9181 (Arabasz). In this respect, as discussed above, we have concluded that PFS has justified its use of a 2,000-year mean return period DBE for the PFSF using well established risk-principles, with which the State fully agrees. Thus, while the appropriateness of the conclusion reached here is corroborated by the similar determination reached with respect to the INEEL ISFSI, it is not dependent upon the INEEL determination.

iv) Section E, Basis 6(a)

484. In Basis 6(a), the State claims that the 2,000-year mean return period for the PFS facility does not ensure an adequate level of conservatism because design ground motion levels for certain new Utah building construction and highway bridges are more stringent. The State's conclusion was based on the observation that, for example, the International Building Code 2000 ("IBC-2000") will, when in effect, require a MRP of approximately 2500 years for the DBE, which is greater than the 2,000-year MRP DBE proposed for PFS. Cornell Dir. at A90. However, the comparison between the two sets of codes based solely on the MRP DBE is completely erroneous. Cornell Dir. at A91.

485. As discussed above, the State "emphatically" agreed with PFS that in order to determine the level of safety achieved by an applicable design one has to take a two-handed approach, addressing both the mean return period of the DBE and the conservatisms embodied in the applicable design procedures and criteria. Cornell

Dir. at A93; Tr. 9120-21 (Arabasz); Tr. 12805 (Bartlett). Therefore, it would be inappropriate to compare solely the 2000 mean return period DBE of the PFSF with the higher MRP DBE of the IBC-2000 or other codes. Tr. 9187-88 (Arabasz); Tr. 12805-09 (Bartlett).

486. The design procedures and acceptance criteria of the IBC-2000 are much less conservative than those specified by the NRC's SRPs. For example, a first step of the IBC-2000 design procedures and criteria is to multiply the DBE by two-thirds, which at the PFSF site would reduce the effective IBC-2000 DBE MRP from 2500 years to about 800 years. Cornell Dir. at A93, Tr. 7898-7902 (Cornell). Only in the case of those "essential structures" that merit the IBC-2000 "importance factor" of 1.5 is this two-thirds reduction, in effect, recovered. Cornell Dir. at A93.

487. Even for those "essential structures" for which this reduction is in effect recovered, the model building codes' design procedures and acceptance criteria are significantly less conservative than those in the SRP. The IBC-2000 and UBC model building codes permit much more liberal allowances for the benefits of post-elastic behavior than either DOE-STD-1020-94 PC-3 and PC-4 criteria, or the NRC SRPs. Cornell Dir. at A94; see also Ebbeson Dir. at A12. The net effect of the UBC design and acceptance criteria is a risk reduction ratio R_R of only 2 for essential buildings and structures, which is similar to that achieved by the IBC. Cornell Dir. at A94. By contrast, facilities designed to the NRC SRPs typically have risk reduction ratios of 5 to 20 or more. These differences represent a factor of 2.5 to 10 or more in increased conservatism (as measured by R_R) in the design procedures for nuclear facilities versus those in model building codes, even if the multiplier of two-thirds in the IBC-2000 is ignored. Cornell Dir. at A91. Thus,

the PFSF structures, even though designed using a lower MRP DBE than the starting point for determining the seismic ground motions under the IBC-2000 or UBC model building codes, would be much stronger and able to withstand greater ground motions than a structure designed to the ostensibly higher MRP DBE specified in IBC-2000

488. Thus, while the MRP DBE under the IBC-2000 is 25% larger than the proposed MRP for the PFSF, the more conservative design procedures and criteria of the ISFSIs SRP will ensure that the SSCs at the PFSF have a mean annual probability of failure that is several times (2 to 8 or more) lower than buildings designed to IBC-2000 standards. Moreover, all PFSF important-to-safety SSCs have risk reduction factors sufficient to provide a probability of failure of 10^{-4} or lower, i.e., at least two times lower than essential facilities designed to the IBC-2000. Additionally, as discussed earlier, a number of key important-to-safety SSCs in the PFSF have great robustness and/or fractional operating periods that reduce their probabilities of failure even further. Cornell Dir. at A92. Therefore, structures and components important to public health and safety at the PFSF would be much less likely to fail in an earthquake than would other facilities essential for public health and safety in the event of an earthquake, such as bridges, hospitals, fire stations, etc.

v) Section E, Basis 6(b)

489. In Basis 6(b), the State claims that the 2,000-year mean return period for the PFS facility does not ensure an adequate level of conservatism because the return period was chosen based on the twenty-year initial licensing period rather than a potential thirty to forty-year operating period. As explained by Dr. Arabasz, this basis originated as a challenge to the Staff's logic set forth in the preliminary SER

that peak ground motion values corresponding to a 2,000-year return period earthquake were adequately conservative because they had a 99% probability of not being exceeded in the 20-year licensing period of the PFSF. Tr. 9183, 9190 (Arabasz). The Staff no longer asserts this rationale as a basis for approving the exemption. See Staff Exh. C. at 2-50, 2-51.

490. As explained by Dr. Cornell, hazards in virtually all areas of public safety are measured in terms of frequency of occurrence (e.g., measured in annual probabilities, in probabilities per 50-year period, or in per human lifetime units), and the same safety criteria are specified regardless of the length of the activity in question, the exposure time, the estimated facility life, or the licensing duration. Cornell Dir. at A94. The purpose of choosing annual risk as a basis for measuring hazard is to avoid logical inconsistencies that would arise from using lifetime risk. For example, under the lifetime risk approach an apartment building with a life of 10 years would be designed to a lesser protective standards (fire, seismic, etc.) than an apartment with a life span of 100 years. This would result in residents living in the "10-year" apartment being exposed to greater annual risk than those living in the "100-year" apartment. Tr. 8004-05 (Cornell). Similarly, for example, under a lifetime risk approach, older workers could logically be subject to greater risks than younger workers, which would lead to reduced work place protection standards for older workers, e.g., less protection against cancer-inducing activities (such as working with asbestos) or no shields around dangerous equipment, etc. Cornell Reb. at A3.

491. Dr. Arabasz in his testimony pointed to standards, such as the national seismic hazard maps, that depict probabilities in units such as 10%, 5% or 2% probability of exceedance in 50 years. Arabasz Dir. at A14-A15. However, as explained by

Dr. Cornell, stating probabilities of exceedance in terms such as a 10% probability of exceedance in 50 years (as opposed to an annual probability of exceedance of $2x10^{-3}$) is just a different way of presenting the frequency of occurrence. Cornell Reb. at A1. This is clearly reflected in Dr. Arabasz's quotation from the National Research Council's Panel on Seismic Hazard Analysis, which directly equates a design seismic hazard level with a 10% probability of exceedance in 50 years to an annual probability of exceedance of $2x10^{-3}$. Arabasz Dir. at 15. The important point is that neither frequency standard is predicated on the lifetime of a facility, nor does the application of the standard vary depending on a facility's projected lifetime. For example, applying a seismic standard of 10% probability of exceedance in 50 years to two buildings, one constructed for a 10-year lifetime and the second for a 100-year lifetime, respectively, would result in the same annual probability of exceedance of $2x10^{-3}$ for each building. Cornell Reb. at A1; see also Tr. 9195-98 (Arabasz).

492. Thus, none of the conventions that are in use for expressing the required seismic safety level are stated in terms that make this level dependent on the life of the building or facility Id. In fact, using a design return period proportional to the duration of the facility lifetime results in potential logical inconsistencies that make such an approach impractical. Cornell Reb. at A2; Tr. 10164-70 (Arabasz).

493. Dr. Arabasz acknowledges that he is not a risk expert and does not have a "firm basis" for saying that one should use an annual or lifetime basis for selecting the appropriate design level earthquake for the PFSF. Tr. 9191-93 (Arabasz). Further, Dr Arabasz agrees that under the DOE-1020 framework, which he generally favors, "the mean annual frequency" would be the "basis for determining the appropriate design basis earthquake," but he questions whether the NRC has a simi-

larly clearly established framework for decision-making based on annual frequencies. Tr. 10170 (Arabasz); see also Cornell Reb. at A2.

494. The NRC has adopted, however, annual frequency risk metrics as the basis for selecting the appropriate level of safety under its risk-informed regulatory framework. For example, both the Commission's Reactor Safety Policy Statement and Regulatory Guide 1.174 clearly set forth annual frequency-based risk acceptance guidelines for NPPs where the performance objectives are Core Damage Frequency and Early Large Release Frequency. While these guidelines are for NPPs, the same general risk-based principles employing frequency based risk metrics as opposed to life-time based risk metrics would apply to the PFSF.

495. Further, adoption of lifetime risk metrics would lead to inconsistent and illogical results. Under lifetime risk metrics, the annual level of risk would change depending on whether the PFSF was planned to be a 10, 20 or 40 year facility, which from a societal risk standpoint is inconsistent with the general risk principles enunciated above. For example, if the spent fuel were not stored at the PFSF it would be stored at another location with attendant risks associated with its storage there. The only way to make such decisions on a comparative risk basis is to use annual risk, and not lifetime risk, as the basis for decision. Further, use of a lifetime risk would raise practical issues on how the appropriate design basis earthquake should be determined in light of potential relicensing of a facility, or how relicensing might affect the already established seismic design basis of a facility. These are practical concerns further support the use frequency risk based metrics in determining the appropriate design basis earthquake for the PFSF. See Cornell Dir. at A94; Cornell Reb. at A2; Tr. 10164-70 (Arabasz).

4. Radiological Dose Consequences

a. Applicable Regulatory Standards for Radiological Dose Consequencess

496. Basis 2 of Section E of Contention Utah L/QQ asserts that "PFS has failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits."

497. 10 C.F.R. Section 72.104(a) provides that "[d]uring 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 . . . ." Thus, notwithstanding the State's claim in Basis 2 of Section E, the radiological dose limits found in 10 C.F.R. § 72.104(a) are for "normal operations and anticipated occurrences," not for seismically–induced events.

498. A cask tipover during a seismic event is a beyond-design-basis accident for which the applicable dose limit is the 5 rem limit of 10 C.F.R. § 72.106(b). See Waters Dir. at A9, A11; Singh/Soler/Redmond Dir. at A14-A15; Tr. 12379 (Resnikoff). For this reason, the dose limits in 10 C.F.R. § 72.104(a) are not applicable to a cask tipover at the PFSF. See Waters Dir. at A11; Singh/Soler/Redmond Dir. at A12-A17; Tr. 12379 (Resnikoff).

499. All parties ultimately agreed that the radiological dose limits in 10 C.F.R. § 72.106(b) would apply to the consequences of a seismic event at the PFSF, not those in 10 C.F.R. § 72.104(a). Waters Dir. at A7; Singh/Soler/Redmond Dir. at A12-A17; Tr. 12379 (Resnikoff). 10 C.F.R. § 72.106(b) provides that "[a]ny individual located on or beyond the nearest boundary of the controlled area may not receive from any design basis accident the more limiting of a total effective dose equivalent of 0.05 Sv (5 rem) . . . ."

500. Although Dr. Resnikoff's prefiled direct testimony discussed the application of the dose limits of 10 C.F.R. § 72.104(a) to a cask tipover accident, the testimony reflected not Dr. Resnikoff's opinion as to the the relevant dose limits, but the directions of the State regarding how to present his testimony. Tr. 12376 (Resnikoff). Dr. Resnikoff testified that he did not believe that § 72.104(a) governs an accident involving a cask tipover during a seismic event, but that § 72.106(b) should apply instead. Tr. 12379 (Resnikoff).

501. Having conceded that 10 C.F.R. §72.106(b) is the controlling regulatory standard, the State raised for the first time during the course of the hearings an issue as to the duration of the postulated accident. This newly-raised issue then became the State's "$64,000.00 Question". Tr. 12367 (Curran). Counsel for the State represented (Tr. 12468) that there was testimony from Dr. Resnikoff that "the accident is a year." But no such testimony existed. Dr. Resnikoff's pre-filed testimony merely calculated dose rates on an annual basis and was silent on the duration of the postulated accident condition. See, e.g., Resnikoff Dir. at A23(b); State Exh. 141; State Exh. 143.

502. The applicable regulations in 10 C.F.R. § 72.106(b) do not place any express limit on the duration of an accident. Tr. 12600 (Resnikoff). There is no regulatory guidance directly on point regarding the duration of an accident for calculation of dose limits, although the NRC assumes a 30 day duration for some analyses, consistent with the loss of containment calculations for accident dose levels for Part 72 facilities described in NUREG-1567, Section 9.5.2. Staff Exh. 53; Tr. 12222 (Waters). Dr. Resnikoff testified that he would consider the dose limit to apply however long the accident condition lasted (Tr. 12600 (Resnikoff)), but did not know how an accident would be defined under NRC regulations or how long it

would last. Tr. 12506-08 (Resnikoff). As discussed below, regardless of the duration assumed for the postulated accident condition, the 5 rem limit of 10 C.F.R. § 72.106(b) will not be exceeded.

b. PFS's Evaluation of Radiological Dose Consequences Arising from a Beyond Design Basis Seismic Event

503. As discussed in Section D above, the analyses undertaken by Holtec and Sandia demonstrate that a cask will not experience any uplift during a design-basis earthquake. Likewise, cask displacements will be on the order of a few inches, precluding cask collision, even during conditions that will maximize sliding of the cask. Cask rotation will be small, with large margins of safety against tipover.

504. During the ground motions associated with a 10,000-year return period earthquake, the Holtec and Sandia analyses show that the casks will not tip over. Findings 132-148, 198-211. Likewise, uplift during such a beyond design basis seismic event was found to be on the order of fractions of an inch. Finding 211. Even under worst case assumptions, neither the Sandia nor the Holtec analyses showed cask-to-cask impacts resulting from sliding. Only in the Holtec simulations that intentionally tried to maximize cask displacements and cask rotations for a 10,000 year beyond-design-basis earthquake did any cask impacts (caused by cask precession or out-of-phase rotations) take place, and the simulations showed that those impacts occurred at relatively low speeds with no damage to the casks or loss of stability. Singh/Soler Dir. at A169; PFS Exh. OO. Even under those unrealistic conditions, maximum cask rotation was on the order of 10 to 12 degrees, representing a factor of safety against tipover of more than 2 when measured against the angle at which a cask would tip over as a result of its own moment. Fingding 135

505. Although it has been demonstrated that the casks will not tip over, PFS analyzed a non-mechanistic hypothetical tipover event in accordance with applicable regulatory guidance. Singh/Soler Dir. at A43; Waters Dir. at A15. The results of this analysis show that all stresses on the storage cask remain within the allowable values of the HI-STORM 100 System Certificate of Compliance ("HI-STORM CoC"), assuring the integrity of the MPC confinement boundary with large margins of safety. Singh/Soler/Redmond Dir. at A19. Therefore, there would be no releases of radioactivity even in the event a of a postulated tipover.

506. Holtec qualitatively evaluated the potential radiological consequences of a hypothetical cask tipover event in its Final Safety Analysis Report for the HI-STORM 100 System and determined that impact of the cask on the pad would only cause localized damage to the concrete and outer shell of the storage cask at the point of impact, reducing somewhat the roundness of the storage cask in the immediate area of impact. Singh/Soler/Redmond Dir. at A19, A38.

507. The HI-STORM 100 System storage cask consists of both a radial concrete shield and an outer steel shell. The concrete is fully encased in a steel structure, and four large steel ribs are located between the inner and outer shell. It is physically impossible for the concrete to be lost in the event of impact damage. A local deformation would not significantly affect the shielding performance of the storage cask, since the same mass of steel and concrete would still be present. Singh/Soler/Redmond Dir. at A38. Because radiation shielding is dependent on mass rather than thickness (Tr. 12479 (Resnikoff)), rearrangement of the mass present in the shielding will not result in significant changes in radiation dose levels, since loss of mass in one location of the cask will be offset by an increase in mass in another location. Tr. 12148-50 (Soler, Redmond); Tr. 12244 (Waters).

Additionally, the local deformations would occur at the top of the storage cask, whereas the radiation doses are greater at the middle of the cask. Tr. 12551-52, 12567-68 (Soler, Redmond). Therefore, any increase in the radiological dose levels due to localized deformation of the cask would at most be minimal. Singh/Soler/Redmond Dir. at A38.

508. Holtec also evaluated the radiological dose consequences resulting from the hypothetical tipover of multiple casks. Singh/Soler/Redmond Dir. at A20-A30. Hypothetical multiple cask tipovers would likely result in similar localized damage for each of the casks tipped over, with no significant aggregate effect on radiological doses at the owner-controlled area ("OCA") boundary. Singh/Soler/Redmond Dir. at A23, A26; Waters Dir. at A18, A19. The greatest potential for increase in radiological doses at the boundary would not be due to damage to the cask or the MPC, but to the possibility that the bottom of the cask, which has less radiation shielding, might face the OCA boundary. Singh/Soler/Redmond Dir. at A23; A26; Waters Dir. at A21; Resnikoff Dir. at A20.

509. Holtec evaluated the effect that 4,000 tipped-over casks would have on the radiation dose at the OCA boundary, compared to the doses due to releases from the casks in their normal upright position. In the upright position, the side of the storage cask is in a direct line of sight from all equidistant locations from the cask, the top is not visible from any location, and the bottom is shielded by the ground. In a tipped-over position, the top or bottom of the cask would be visible from some locations and not from others, while the side of the storage cask cylinder (now horizontal) would also be visible from some locations and not others. Additionally, since the storage cask would be lying on its side, a large portion of the outer radial surface of the cask would be shielded by the ground. From its evaluation of

the geometry of the storage cask Holtec concluded that, overall, the decrease in dose rate from the sides of a tipped-over storage cask should more than compensate for the increase in dose rate from the top or bottom of the cask. Further, in the event of multiple casks tipping over, the orientation of the tipped-over casks would be random and the bottoms and tops of many of the casks would be shielded from the OCA boundary by other casks. Singh/Soler/Redmond Dir. at A23-A26.

510. Thus, in the event of a beyond-design-basis accident that caused the tipover of all, or a significant portion of the 4,000 casks at the PFSF site, the radiological dose levels at the OCA boundary would not be increased from the 5.85 mrem per year for normal operations which had previously been calculated. Thus, there are approximately three orders of magnitude of margin between the expected dose rate at the OCA boundary for 4,000 casks in a tipped-over condition compared to the 5 rem accident dose limit in 10 C.F.R. § 72.106(b). Singh/Soler/Redmond Dir. at A27-A28.

511. In addition, many conservatisms were included in PFS's calculation of the 5.85 mrem/year dose at the OCA boundary. These included:

- The calculation assumed that all 4,000 casks contain fuel with a burnup of 40,000 MWD/MTU and a cooling time of 10 years. This is physically impossible, since the MPCs will be delivered over many years and each additional year of cooling further reduces the radiation source term. A more realistic value of 35,000 MWD/MTU and a cooling time of 20 years has been used in other PFS analyses. These more realistic assumptions result in a greater than 50% reduction in the calculated normal doses at the site boundary, from 5.85 mrem/year to 2.10 mrem/year.

- The calculation assumed that the fuel assemblies inside the casks have the highest gamma and neutron radiation source term in all fuel storage locations, maximizing radiological doses.

- The calculation assumed that the fuel has been subject to a single irradiation cycle in calculating the source term. This ignores the down time during reactor operations for scheduled maintenance and refueling, which would reduce the source term by effectively increasing the cooling time.

Using more realistic assumptions would significantly reduce the calculated radiological dose levels, further decreasing the expected radiation dose consequences of the hypothetical tip over of all 4,000 casks at the PFSF. Singh/Soler/Redmond Dir. at A28.

c. *State Challenges Based on Differences Between the HI-STORM 100 Certificate of Compliance and the PFSF Design Basis Analysis for the HI-STORM 100 Storage Cask*

512. In his prefiled testimony, Dr. Resnikoff noted that there were differences between the HI-STORM CoC and site-specific conditions at the PFSF, and asserted that these differences resulted in a failure of PFS to accurately quantify the consequences of a design basis earthquake at the PFSF. Resnikoff Dir. at A9. Dr. Resnikoff cited three differences between the HI-STORM CoC and the PFSF conditions: differences in ground motion, occupancy time, and the thirty-three hour corrective action time limit in the event of a 100% air inlet duct blockage of storage casks. Resnikoff Dir. at A9 and A22.[39]

513. Dr. Resnikoff's testimony was apparently premised on the assumption that the HI-STORM CoC is supposed to reflect the "fact and conditions" at the PFSF site. Resnikoff Dir. at A8. This assumption is clearly incorrect. Holtec performed general design analyses in its FSAR for the HI-STORM 100 System storage cask, which support the HI-STORM CoC. Singh/Soler/Redmond Dir. at A31. Under

[39] Dr. Resnikoff also asserted that because all the casks could tip over at the PFSF, PFS needed to calculate the dose consequences due to the tipover of an entire field of casks. Id. As discussed above, if such an event were to occur, the dose consequences would be far below the 5 rem limit.

the HI-STORM CoC, nuclear power plant licensees may use the HI-STORM system at their sites under the general license provision of 10 C.F.R. § 72.210 as long as they meet the conditions of both 10 C.F.R. § 72.212 and the CoC. Singh/Soler/Redmond Dir. at A31.

514. However, satisfactory performance of the HI-STORM system may also be demonstrated by site-specific analyses. Holtec has performed such site specific analyses for the PFSF, demonstrating satisfactory performance of the system at the PFSF. Singh/Soler/Redmond Dir. at A31. Thus, the differences claimed by Dr. Resnikoff to exist between the HI-STORM CoC and the PFSF are irrelevant.

i) Design Basis Ground Motion

515. The design-basis ground motion for the PFSF is 0.711g in the horizontal direction and 0.695g in the vertical direction. These values exceed the ground motion limits in the HI-STORM CoC. Resnikoff Dir. at A8a; Singh/Soler/Redmond Dir. at A34. However, Holtec conducted site-specific cask tipover dynamic analyses for the PFSF which demonstrate that the casks do not tip over under the PFSF design basis ground motions, or even under ground motions due to a 10,000-year beyond–design-basis earthquake. See Singh/Soler/Redmond Dir. at A34. Thus, the variance between the ground motions for the PFSF DBE and the analyses supporting the HI-STORM CoC has no significance. Id.; Tr. 12435-36 (Resnikoff).

ii) Occupancy Time

516. The PFS site-specific analysis for radiation dose levels uses a 2,000 hours/year occupancy time for calculating normal operating dose levels (conservatively based on an assumed worker at the site boundary 40 hours a week for 50 weeks a year), whereas the HI-STORM CoC uses 8,760 hours/year to calculate the normal operating dose. Singh/Soler/Redmond Dir. at A29, A32. The dose limits estab-

lished by 10 C.F.R. § 72.104(a) apply to "any real individual who is located beyond the controlled area," not to a hypothetical person at the OCA boundary. Thus, occupancy time for normal operating conditions is determined using a real person standard, which takes into account the site-specific circumstances at a facility. Singh/Soler/Redmond Dir. at A29. This interpretation is endorsed by Staff Regulatory guidance. Id.; see also, Tr. 12067 (Redmond). Likewise, for accident conditions, the 5 rem limit would apply to real individuals, and site-specific circumstances would similarly need to be taken into account, including any remedial measures that may be taken during extended accident conditions (e.g., shielding or moving persons away from OCA boundary). See, Tr. 12072 (Redmond); Tr. 12266-67 (Waters).

517. The PFSF has a buffer zone of two miles on the southern side and a buffer zone of nearly a mile on the eastern side that preclude an individual from being present at the OCA boundary twenty-four hours a day. Tr. 12561-64 (Donnell). The land to the west of the PFSF is owned by the Bureau of Land Management and is used for grazing. Tr. 12564-65 (Donnell). The land immediately to the north of the PFSF is privately owned and used for livestock grazing with concomitantly low expected human occupancy time. Tr. 12564-65 (Donnell). The nearest offsite residence to the PFSF is located over two miles away from the OCA boundary, with intervening high ground blocking any line of sight. Tr. 12557-58 (Redmond); Tr. 12571-72, 12578-79 (Donnell). No witnesses for any party testified as to any plans to change existing land uses surrounding the PFSF. Changes in existing land use are prohibited in the buffer zone surrounding the PFSF. Tr. 12562 (Donnell).

518. Based on the land use surrounding the PFSF, the assumed 2,000 hours per year occupancy time is conservatively high. Tr. 12067-68 (Redmond). The only individuals likely to be present at the OCA boundary would be workers, who are assumed to be present 40 hours a week for 50 weeks a year to produce an upper bound of 2,000 hours per year exposure at the site boundary. Singh/Soler/Redmond Dir. at A29; PFSF SAR §7.3.3.5.

519. Thus, using a conservatively high 2,000 hours/year occupancy time is appropriate for normal operations, given the site-specific circumstances at the PFSF. Singh/Soler/Redmond Dir. at A29; see also Tr. 12263-65 (Waters). Such an occupancy time would also be conservatively high for postulated accident conditions. Tr. 12266-67 (Waters). In addition to measures to limit occupancy of areas of potential radioactive contamination, remedial measures, such as the construction of an earthen berm, could easily be undertaken to assure that radiological dose levels at the boundary of the OCA do not exceed regulatory limits following a beyond-design-basis earthquake. See Tr. 12583-84 (Donnell); Tr. 12622-23 (Resnikoff).

iii) Relevance of Thirty-Three Hour Time Period for Corrective Action of Complete Air Inlet Blockage Under HI-STORM CoC

520. The thermal analysis used to support the HI-STORM CoC provides that in the event of a 100% blockage of the air inlet ducts, the short term temperature limit of the concrete would be expected to be reached in thirty-three hours. Staff Exhibit FF. The thirty-three hour period for correcting a 100% air duct blockage was based on the requirement that the casks be visually inspected every twenty-four hours, allowing an additional eight hours for corrective action to be taken. Tr. 12152 (Singh). The thermal analysis that was used in the HI-STORM CoC makes

the conservative (but unrealistic) assumption that no heat transfer to the surrounding air will occur. In effect, the calculation presumes that the cask not only has its air inlet ducts completely blocked, but that it is shrouded in a "heavy blanket" that prevents any heat transfer. Tr. 12152-53 (Singh). Only under those extreme conditions would the short-term temperature limit of the concrete be reached in thirty-three hours. Id.

521. It is physically impossible for all air inlet ducts of a cask to be blocked due to a cask tip-over. Singh/Soler/Redmond Dir. at A51. Even in a tipped-over condition, heat transfer continues to take place and the air inlet ducts continue to dissipate heat, thus concrete temperature would be expected to remain below the short term limit. Singh/Soler/Redmond Dir. at A53; see also, Tr. 12152-54 (Singh).

522. Further, even assuming all vents were blocked, the bounding steady state temperature for the concrete would be well below the 600°F necessary for extensive sustained water evaporation. Singh/Soler/Redmond Dir. at A53; see also, Tr. 12153-54 (Singh). Both conduction and radiation of heat still occur from a storage cask that has all its air inlet ducts blocked. Tr. 12300-01 (Waters). Therefore, the evaporation of water from the concrete of a tipped-over cask would be minimal, even if the cask remained in a tipped-over position for a period of months. Singh/Soler/Redmond Dir. at A53.

523. Exceedance of the short-term temperature limit of the concrete does not affect public health and safety because it (1) has no effect whatsoever on the containment of the spent fuel within the storage cask; and (2) there would be no significant reduction in the shielding effectiveness of the system. Tr. 12154-55 (Singh); see also, Tr. 12440-41 (Resnikoff).

d. State Challenges to PFS's Evaluation of Cask Damage

524. Dr. Resnikoff identified three possible mechanisms by which damage might occur to a HI-STORM 100 System storage cask during a design basis seismic event: cask tipover, sliding and impact, and uplift. Tr. 12381-83 (Resnikoff). He acknowledged that all the mechanisms that he postulated are based entirely on the Altran Report and the testimony of State witnesses Khan, Ostadan, and Bartlett and, despite language to the contrary in his prefiled testimony, Dr. Resnikoff does not have any independent basis or expertise for assessing whether any of these mechanisms will occur or to what extent. Tr. 12381-85, 12394-98 (Resnikoff). Rather, Dr. Resnikoff presumed a cask tipover (Tr. 12402 (Resnikoff)), despite the fact that the Altran Report did not conclude that such a tipover would occur, and neither did Dr. Khan. See Tr. 12469-73 (Resnikoff). In fact, no State witness has testified that sliding and collision of the casks, tipping of the casks, or uplift of the casks would occur to such an extent as to cause cask tipover.

525. Dr. Resnikoff conceded that he did not know whether a cask impact due to a beyond-design-basis seismic accident at the PFSF would cause flattening or other damage to the storage cask (Tr. 12406 (Resnikoff)), whether or how much cracking of the steel or concrete would occur (Tr. 12407-08 (Resnikoff)), or whether or how much thinning of the steel would occur (Tr. 12406 (Resnikoff)). Dr. Resnikoff acknowledged that the State's allegations relating to damage to the cask, including all the mechanisms postulated in his testimony, were theoretical concerns and that he did not have expertise to determine whether or to what extent they could occur. Tr. 12413-18 (Resnikoff). Nor had he attempted to estimate any effect on radiation doses arising from any postulated damage to the casks. Tr. 12414 (Resnikoff).

526. Dr. Resnikoff speculated that it may be possible for the deformation of a fallen cask to be in a location on the storage cask different than the Holtec analysis suggests due to one cask falling onto another cask, or from some other seismically-induced cask-to-cask interaction. Tr. 12599 (Resnikoff). Dr. Resnikoff, however, did not know whether it is physically possible for one cask to fall on top of another prone cask (Tr. 12613 (Resnikoff)), had no detailed knowledge of the behavior of the casks during a seismic event (Tr. 12613 (Resnikoff)), and had no knowledge of how the casks might interact from a structural engineering standpoint (Tr. 12613 (Resnikoff)).

527. Dr. Resnikoff also acknowledged that he had neither experience nor expertise in measuring or quantifying concrete cracking (PFS Exh. 240 at 42-45, 47, 71), determining the strength of steel or concrete (PFS Exh. 240 at 46), calculating the initial angular velocity of a cask during tipover (PFS Exh. 240 at 70-71; Tr. 12403-04 (Resnikoff)), or measuring or quantifying thinning or flattening of the steel in the cask shell due to impact (PFS Exh. 240 at 80-81). No State witness has provided testimony concerning whether or how much a cask impact from uplift, sliding and collision, or tipover due to a postulated cask tipover event at the PFSF would cause: (1) flattening or other damage to the storage cask, (2) cracking of the steel or concrete, (3) thinning of the steel shell or radial concrete shield, or (4) displacement of the cask lid. Neither has any State witness quantified the effects of any of those mechanisms.

528. Dr. Resnikoff further admitted that he had no background or experience in cask stability analyses (Tr. 12397-98 (Resnikoff)), had not conducted cask stability analyses for the PFSF (Tr. 12396-98 (Resnikoff)), had no knowledge of the behavior of the storage casks from a structural engineering perspective (Tr. 12614

(Resnikoff)), had never modeled or reviewed a simulation of a storage cask drop outside of this case (Tr. 12398-99 (Resnikoff)), and did not know how to evaluate whether a cask lid displacement would occur during tipover (see Tr. 12414-17 (Resnikoff)).

529. Despite this lack of expertise, Dr. Resnikoff testified to three specific concerns that he had with the Holtec cask tipover analysis: (1) the potential unconservativeness of Holtec's assumption of zero initial angular velocity; (2) a related concern that deceleration at the top of the storage cask might exceed 45g; and (3) Holtec's asserted failure to account for the dynamic impulse resulting from displacement of the cask lid upon impact in a tip-over event. Resnikoff Dir. at A16, A21; Tr. 12403 (Resnikoff).

i) Initial Angular Velocity

530. Based on the Altran Report, Dr. Resnikoff postulated that the Holtec analysis of cask tipover was inadequate because the initial angular velocity of a falling cask may be greater than zero. However, Dr. Resnikoff has never calculated an initial angular velocity for any storage cask tipover PFS Exh. 240 at 70-71), nor did he have the expertise to do so. Tr. 12403-04 (Resnikoff). Instead, Dr. Resnikoff testified that he asked "[the State's] other experts what is the angular velocity and is zero correct, and their opinion [was] that the zero initial angular velocity could be greater than zero." Tr. 12403 (Resnikoff).

531. There is no testimony by any State witness that supports the conclusion that an initial angular velocity greater than zero would be either realistic or more appropriate for a cask tipover at the PFSF. State soils expert Dr. Bartlett summarily asserted, in reference to the Holtec non-mechanistic cask tipover analysis, that "the tipover event postulated that the cask would be perched on its edge with zero an-

gular velocity. During an earthquake, that's not true. If we go to tipover, we have some angular velocity." Tr. 12870-71 (Bartlett). However, Dr. Bartlett admitted that he had not been involved in any calculations of cask stability or the results of a tipover event (Tr. 12870 (Bartlett)), and there is no evidence that he has expertise to perform such an analysis.

532. The Holtec analyses of dynamic cask behavior have shown that the behavior of the cask is characterized by tilting from the vertical resulting in a plane of precession for a certain duration in the course of the earthquake event, resulting in an oscillatory rocking motion with limited return to the vertical position until the rocking finally ends when the earthquake subsides. Singh/Soler/Redmond Dir. at A39. If the earthquake ground motions were assumed to be increased to the point at which a cask would tip over, the initiating angular velocity propelling the cask towards the ground would be quite small. Singh/Soler/Redmond Dir. at A39.

533. Furthermore, the precessionary motion of the cask enables it to remain stable after the center of gravity of the cask is well past the "center-of-gravity-over-corner" position. As a result of this precessionary motion, the location of the cask's center of gravity is likely to be much lower than in the static tipover scenario (where tipover begins as soon as the center of gravity crosses the vertical plane containing the axis of overturning rotation). The combination of a shorter distance to fall and a negligible initial angular velocity propelling the tip-over further supports the assumption of an initial angular velocity of zero because a cask tipping away from precessionary motion is expected to have substantially less kinetic energy of collision than one tipping from a zero velocity with the center of gravity over corner. Singh/Soler/Redmond Dir. at A39. Thus, the assumption of an initial angular velocity of zero is appropriate.

ii) Deceleration in Excess of 45g and Design Margin of the MPC

534. Dr. Resnikoff's pre-filed testimony indicated that his concern regarding the possibility of the top of the cask decelerating at a rate in excess of 45g was premised on the initial angular velocity being greater than zero. See Tr. 12410-12 (Resnikoff). He changed his testimony at the hearing and acknowledged that damage to the cladding on fuel rods contained in the fuel assemblies within the storage cask would not be an issue unless the assemblies were subjected to an acceleration of at least 63g. Tr. 12409-10 (Resnikoff). Dr. Resnikoff did not know how large an initial angular velocity would be required to exceed the 63g limit, but conceded that an initial angular velocity of greater than zero would be required. Tr. 12411-12 (Resnikoff).

535. The HI-STORM 100 FSAR places a 45g limit on the deceleration for the top of the HI-STORM 100 storage cask in the event of a cask tipover event. This is a licensing limit that does not represent the actual ability of the storage cask, the MPC, or the fuel assemblies to maintain both containment and radiation shielding. Singh/Soler/Redmond Dir. at A40; Tr. 12158 (Singh). The spent fuel assemblies have design margins that allow them to withstand accelerations up to at least 63g. Singh/Soler/Redmond Dir. at A40; Tr. 12409-11 (Resnikoff); Tr. 12158 (Singh). There has been no analysis of postulated beyond-design-basis accidents that resulted in decelerations greater than the 45g limit in the HI-STORM 100 FSAR, let alone the 63g design limit. Tr. 12411 (Resnikoff).

536. The MPC also has substantial design margins beyond the 45g level. A hypothetical 25 foot end drop of a loaded canister on a hard concrete foundation resulted in a computed strain in the confinement boundary of 41% of the failure strain limits for the canister material. Singh/Soler/Redmond Dir. at A40. The computed strain

showed that the MPC could experience a maximum deceleration of 300g without loss of confinement. Tr. 12075 (Singh).

537. Thus, exceeding the 45g deceleration limit imposed on the top of the canister in the HI-STORM 100 FSAR would not result in increased radiological dose consequences. Decelerations would have to exceed 63g before there was a concern regarding the possible effect of such decelerations on the fuel assemblies contained in the MPC. Tr. 12409-11 (Resnikoff). Moreover, due to the large margins of safety built into the design of the MPC, much larger decelerations than 45g would be required before the containment function of the MPC was compromised. Singh/Soler/Redmond Dir. at A40; Tr. 12158 (Singh).

iii) Cask Lid Displacement

538. In his prefiled direct testimony, Dr. Resnikoff posited that tipover could cause additional "dynamic impulses" to the structure of a cask. He described his concerns as follows:

> In a tipover event, discussed in TSAR Appendix 3.B, the cask walls at the top of the cask are expected to flatten slightly (0.11 inch, p. 3.B-5) when the cask top strikes the ground. On the other hand, the cask lid plate is expected to be displaced as much as 4.9 inches in a tip over event (TSAR, p. 3.A-15). This indicates to me that the 3 ¾ inch thick lid plate is going to strike the ground in a tipover event and send a strong dynamic impulse to the cask wall and canister. It does not appear that this cask detail, that may affect the canister welds, has been modeled.

Resnikoff Dir. at A21. (Footnote omitted.)

539. Dr. Resnikoff's testimony misinterpreted the results of the HI-STORM cask tipover analysis in several significant respects. First, Dr. Resnikoff incorrectly assumed that the displacement reported in the TSAR is a displacement of the cask lid relative to the cask body. Tr. 12549-50 (Soler). In fact, the cask lid and the

cask body move together, not relative to one another, so that the 4.9 inches of displacement applies to both the cask lid and the cask body. Tr. 12550-51 (Soler). Second, Dr. Resnikoff mistakenly assumed that any dynamic forces due to the displacement of the cask lid and cask body are not adequately taken into account in the Holtec analysis, when in fact any dynamic forces due to the impact of the cask lid or body are included in the modeled behavior. Tr. 12551 (Soler). Third, the effect on the canister welds of any such forces are considered in the tipover model and no deleterious effects to the welds occur during a hypothetical tipover event. See Tr. 12551 (Soler). Fourth, to the extent that damage to a cask could hypothetically be caused by a tipover, the analysis demonstrates that any deformations would be small, localized, and would occur within one foot of the top of the cask, where radiation dose consequences are the least significant. Tr. 12551-52 (Soler, Redmond). Thus, Dr. Resnikoff's concern regarding cask lid displacement is unrealistic.

e. State Estimation of Radiological Dose Consequences of a Worst Case, Beyond-Design-Basis-Accident at the PFSF

540. Dr. Resnikoff's prefiled testimony contained two radiation dose calculations: an estimation of the gamma dose from the bottom of eighty prone storage casks, with their bottoms facing the OCA boundary (State Exh. 141), and an estimation of the neutron dose from a cask based on the amount of "water evaporated" from the concrete shielding (State Exh. 143). Beginning with amended State Exh. 141A, Dr. Resnikoff combined both scenarios – cask tip over and loss of hydrogen shielding – to portray a total, worst case radiological dose at the OCA boundary. Both original calculations (as well as the subsequently amended overall dose cal-

culation, State Exh. 141A) contained so many errors that these calculations cannot be given any weight.

541. The dose exposure that Dr. Resnikoff ultimately calculated at the OCA boundary was less than 150 mrem for the first year, assuming a hypothetical person were at the OCA boundary for the entire year (which, as discussed above, is not realistic). Radioactive decay would reduce this dose exposure in subsequent years. Thus, assuming that the casks remained on the ground indefinitely with no remedial actions being taken, the 5 rem limit would never be exceeded for a person continuously stationed at the OCA boundary. Tr. 12619-20 (Resnikoff).

i) Neutron Dose Calculation

542. Dr. Resnikoff's neutron dose calculation, State Exh. 143, purports represent the "increased neutron dose due to reduced shielding" in order to estimate "the increase in dose to workers due to neutrons . . . 1 meter from the cask mid-height if all of the water evaporates from a HI-STORM cask." Resnikoff Dir. at A23(b). In this calculation, Dr. Resnikoff assumed that there is some unspecified temperature at which no hydrogen is present in the concrete or the aggregate material contained in the concrete. Tr. 12420-23 (Resnikoff). Dr. Resnikoff did not try to calculate the actual amount of hydrogen loss that would take place if a HI-STORM 100 cask tipped over, nor did he have any idea how to calculate the thermal degradation of the cask's concrete over time (PFS Exh. 240 at 90-93); nor had he ever used computer programs that computed the temperature of concrete over time (Id.). He also did not know how to estimate the reduction in shielding due to concrete heating up over time (Id. at 93). Indeed, this was his first attempt to examine thermal degradation in concrete and quantify the loss of radiation shielding that may result. Tr. 12418-19 (Resnikoff). Dr. Resnikoff was also not

aware of the actual physics of hydrogen evaporation from concrete when he made his calculations. Tr. 12422 (Resnikoff).

543. The premise of Dr. Resnikoff's calculation of the lack of any hydrogen in the concrete due to evaporation of water is unrealistic. It is not easy to evaporate water within concrete, because it is in a confined space, and as the water evaporates the air pressure increases. In turn, the increased air pressure will convert the water vapor back to liquid. Likewise, concrete does not lose its moisture content as easily as water might evaporate from a free surface. In order for large, extensive, sustained water evaporation from the concrete to occur, exposure to high temperatures for a period of months will be necessary. Moreover, it is physically impossible for cask heat-up to release hydrogen contained in the aggregate within the concrete. Singh/Soler/Redmond Dir. at A53; Waters Dir. at A20. In an actual simulation of the worst case scenario for heat degradation of the HI-STORM 100 cask, the Staff indicated that neutron dose rates due to thermal degradation would result in a much smaller increase of computed neutron dose rates than those predicted using the unrealistic assumptions in Dr. Resnikoff's analysis. Waters Dir. at A20. In addition to the erroneous assumptions made by Dr. Resnikoff, his neutron dose calculation was also in error because he used the wrong neutron dose from the SAR, which inflated his calculated neutron dose by a factor of 2.68. Tr. 12607-08 (Resnikoff).

ii) Gamma Dose and Overall Dose Calculation at the OCA Boundary – St. Exh. 141 and 141A

544. Dr. Resnikoff's gamma dose calculation at the OCA boundary was premised on the bottoms of eighty prone storage casks lined up in a row all facing the OCA boundary. St. Exh. 141 at 3-5, 6-8. Such an arrangement is "highly unrealistic."

Singh/Soler/Redmond Dir. at A24-A26, A44-A53. Further, Dr. Resnikoff made numerous errors in his calculation. After correcting these errors, the 5 rem accident dose limit would never be reached even under the unrealistic conditions assumed in the calculation.

545. Dr. Resnikoff made a total of nine different corrections or changes to his overall dose calculation at four different points in the proceeding. These errors are identified in the testimony of the PFS witness by Dr. Redmond as well as by Dr. Resnikoff in the amendments to his pre-field direct testimony and in oral testimony at the hearing. See e.g., Singh/Soler/Redmond Dir. at A46; Tr. 12428-30 (Resnikoff); State Exh. 141A; Tr. 12374-75 (Resnikoff).

546. It would serve no useful purpose to recite the details of the various errors in Dr. Resnikoff's dose rate calculations. Suffice it to say that they leave this Board with little confidence in the accuracy of his analyses. Even after making several of these corrections to his testimony, Dr. Resnikoff testified that he was "pretty confident" that there were no additional errors in his calculation. Tr. 12430-32 (Resnikoff). Yet additional errors were identified in the course of his examination which required him to make additional adjustments (downwards) to his results. See Tr. 12432, 12503, 12607-08 (Resnikoff).

547. A particularly egregious error in Dr. Resnikoff's dose calculations is that he did not consider the effect of radioactive decay. The majority of the gamma radiation from the spent nuclear fuel comes from the radioactive decay of Cobalt-60 and Cesium-137, with Cobalt-60 being the main gamma emitter for radiation emanating off the bottom of the cask, accounting for ninety percent of the total gamma dose calculated by Dr. Resnikoff. Tr. 12619-20, 12624-25 (Resnikoff). Although the half-life or Cobalt-60 is approximately five years, Dr. Resnikoff neglected to

take radioactive decay into account when arriving at his dose estimates. Tr. 12617-20 (Resnikoff); State Exh. 141, State Exh. 141A.

548. Taking into account only the radioactive decay of just the Cobalt-60 and ignoring the decay of other radioisotopes will result in a total radiation dose over fifty years of 2582.1 mrem, or 2.58 rem.[40] In fact, as Dr. Resnikoff admitted, taking into account radioactive decay, the 5 rem accident limit specified in 10 C.F.R. § 72.106(b) is never reached (Tr. 12620 (Resnikoff)) no matter how long one assumes that the casks remain in a worst case tipover and total loss of hydrogen shielding condition, and disregarding any remedial actions that might be taken in the intervening period by PFS or others.

f. Duration of Accident

549. Upon the State's identification of accident duration as the "$64,000 question," Dr. Resnikoff attempted to testify as to accident duration. Dr. Resnikoff had no idea, however, how long a seismically induced accident condition might exist at the PFSF, indicating only that he was concerned about casks being tipped over for years. Tr. 12440-41, 12507-08 (Resnikoff). The longest duration postulated by Dr. Resnikoff was forty years. Tr. 1257-08 (Resnikoff).

550. Dr. Resnikoff testified that he has no experience with estimating the length of time it would take to correct a seismically induced accident at an ISFSI, nor did he have any knowledge about how long it would take and had not undertaken any analyses to determine what kind of accident durations might occur at the PFSF in

[40] This number is obtained from the data generated by Dr. Resnikoff as follows: 962.1 millirem (cumulative gamma dose from decay of Cobalt-60) + 1068.5 millirem (cumulative gamma dose from Cesium-137 assuming no decay from St. Exh 141A times 50 years) + 551.5 millirem (cumulative neutron dose assuming no radioactive decay from St. Exh 141A times 50 years) = 2582.10 millirem.

the event of a beyond-design-basis accident involving the seismically-induced tipover of storage casks. Tr. 12507-09, 12614-16 (Resnikoff).

551. Further, although he testified that occupational dose limits may prolong the duration of an accident (Tr. 12607 (Resnikoff)), he acknowledged that mitigation measures such use of shielding, can be taken to minimize worker exposure in the event of a beyond design basis accident. Tr. 12607; see also, Singh/Soler/Redmond Dir. at A56. The radiological dose levels for such a beyond design basis accident at the PFSF are lower than radiation dose levels workers in nuclear facilities routinely experience. Singh/Soler/Redmond Dir. at A55.

552. Even assuming a physically impossible, worst case cask tipover and loss of all hydrogen shielding event as postulated by the State, the 5 rem radiological dose limits set by 10 CFR Section 72.106(b) will not be exceeded within at least 50 years of a beyond design basis seismically induced accident. Tr. 12619-20 (Resnikoff). Indeed, the radiation doses resulting from any postulated tipover accident would never reach the regulatory limits no matter how long the accident was assumed to extend, hence the accident duration is not a meaningful parameter for purposes of our decision.

553. The nearest resident to the PFSF is two and a half miles away from the OCA boundary, separated by high ground blocking any line of site (Tr. 12557-58 (Redmond); Tr. 12571-72, 12578-79 (Donnell)), and no changes in land use surrounding the PFSF are planned, and in some cases prohibited. See Tr. 12562 (Donnell). Moreover, remedial actions to lower radiological dose consequences at the OCA boundary, such as the construction of an earthen berm, can easily be taken to assure that radiological dose levels at the boundary of the OCA do not exceed regulatory limits. See Tr. 12583-84 (Donnell); Tr. 12622-23 (Resnikoff).

Therefore, even if concerns remained about potential offsite radiation doses following a beyond-design-basis seismic event, such concerns would be readily alleviated by a number of remedial actions which could, and this Board can reasonably assume would, be taken following the event.

5. SECTION E CONCLUSION

554. The PFS seismic exemption is consistent with well understood and widely accepted risk-graded principles. All parties agreed that the two-handed approach employed – using both the return period of the DBE and the conservatisms of the design procedures and criteria – to determine if a performance goal were met was an appropriate methodology. Likewise, the State agreed that a MAPE of 1×10^{-4} was an appropriate performance goal for the PFSF ISFSI, using a risk-graded approach that takes into account the consequences of the failure of an SSC at an ISFSI

555. The record in the proceeding demonstrates that considerable margins exist in the seismic design of the PFSF due to the design procedures and criteria built into the SRPs and confirmed through numerous seismic PRAs. These SRP design margins apply to all SSCs within the CTB, the CTB structure, and the foundations of both the CTB and the storage pads. In addition, Applicant's witnesses have testified to numerous margins that exist in the design of SSCs within the CTB, the CTB structure, and the foundations of both the CTB and the storage pads.that would enable them to withstand earthquakes with return periods on the order of 10,000 years.

556. Further, with respect to the the HI-STORM 100 storage cask system, cask stability analyses performed by Holtec and Sandia show that the casks will not tip-over

even under a 10,000 year earthquake event with significant excess margin remaining. In addition, even if the casks were to tip-over no breach of the MPC confinement boundary would occur. Significant margins exist with respect to the integrity of the MPC confinement boundary.

557. The State's witnesses agree that if the margins testified to in this proceeding are correct, that the PFSF will meet or exceed the intended performance goal of 1×10^{-4} and that the granting of the exemption would be appropriate. We find that the margins are more than sufficient to enable SSCs at the PFSF to withstand without failure earthquakes with return periods on the order.

558. Further, he ultimate issue with respect to the granting of the seismic exemption is whether the facility dtesign will provide reasonable assurance of protecting public health and safety. As the record indicates, all SSCs at the PFSF and the storage casks themselves meet or exceed performance goals sufficient to protect public health and safety. Under no postulated circumstances would the consequences of a beyond-design-basis accident endanger public health and safety. Even counterfactually assuming that the margins either did not exist or a seismic event sufficiently exceeding the performance goal were to occur, such that a worst-case cask tipover and total loss of hydrogen shielding beyond-design-basis accident would occur, there would be no exceedance of the applicable radiological dose limits. Therefore, there is reasonable assurance that the public health and safety would be protected.

IV. CONCLUSIONS OF LAW

A. Section C of Contention Utah L/QQ

1. Pursuant to 10 C.F.R. §§ 72.102(c) and (d) and 10 C.F.R. § 72.122(b), the Applicant has shown that the program implemented by PFS to determine the characteristics of the soils at the PFSF site provides reasonable assurance that the soil conditions are adequate for the proposed foundation loading.

2. Throughout this proceeding, the State has argued that PFS should be required to demonstrate by testing, prior to licensing, that the design requirements for the soil cement and the cement-treated soil that will be used in the seismic design of the PFSF can be achieved. However, imposing such a requirement as a prerequisite to licensing of the PFSF is contrary to Commission precedent and agency practice. The Commission has long held that matters may be left to the NRC staff for post-hearing resolution "where hearings would not be helpful and the Board can 'make the findings requisite to issuance of the license.'" Long Island Lighting Co. (Shoreham Nuclear Power Station, Unit 1), ALAB-788, 20 NRC 1102, 1159 (1984), quoting Consolidated Edison Co. (Indian Point Station, Unit 2), CLI-74-23, 7 AEC 947, 951-52 (1974). Post-licensing resolution is appropriate for matters where a hearing would be unlikely to affect the result. See Southern California Edison Co. (San Onofre Nuclear Generating Station, Units 2 and 3), LBP-82-39, 15 NRC 1163, 1216 (1982). The design requirements for the soil cement and the cement-treated soil at the PFSF are well-established and uncontested, and so are the requirements of the testing program that PFS is committed to conducting.

Therefore, further licensing action in connection with this matter "would be unlikely to affect the result" of the testing program. Either the testing, once conducted, will demonstrate compliance with the design requirements, or PFS will need to take appropriate remedial steps. There is thus no action that needs to be taken prior to licensing of the facility.

3. Post-licensing staff reviews are appropriate where the NRC staff inquiry is essentially "ministerial". Hydro Resources, Inc., CLI-00-08, 51 NRC 227 (2000); Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Installation), CLI-00-13, 52 NRC 23, 33 (2000). Reviewing the results of a testing program that all parties have accepted, and which they all agree will determine whether the specified material properties of the soil cement and cement-treated soil are achieved, is clearly a ministerial act.

4. Moreover, performing post-licensing reviews falls within the NRC Staff's well established purview of verifying inspections, testing, analyses and acceptance criteria ("ITAACs") established by the Commission for various types of facilities. For example, under the licensing process for the construction of new nuclear power plants, ITAACs are carried out after facility licensing by the NRC Staff in order to provide reasonable assurance that the facility has been built and will operate in accordance with the license and the applicable regulations. See, 10 C.F.R. §§ 52.79(c), 52.97 and 52.99 and SECY-00-0092, Combined License Review Process (April 20, 2000). The same post-licensing verification process applies to facilities for the storage of spent fuel. See, e.g., Northern States Power Co. (Prairie

Island Nuclear Generating Plant; Prairie Island Independent Spent Fuel Storage Installation), 47 N.R.C. 37, DD-98-2 (1998); see also Northern States Power Co. (Prairie Island Nuclear Generating Plant; Prairie Island Independent Spent Fuel Storage Installation), 46 N.R.C. 35, DD-97-18 (1997). There is no reason why the Staff's practice exemplified by the ITAACs cannot be successfully applied in this instance.

5. The Applicant has shown reasonable assurance that the implementation of the program that PFS has developed for the testing and construction of soil cement and cement-treated soil will lead to the installation of soil cement and cement-treated soil mixes that will meet the specified design requirements and will give adequate performance for the life of the PFSF.

B. Section D of Contention Utah L/QQ

6. Pursuant to 10 C.F.R. § 72.122(b)(2), the Applicant has provided reasonable assurance that systems, structures and components important to safety at the PFSF have been designed to withstand earthquake phenomena without impairing their capability to perform their safety functions.

C. Section E of Contention Utah L/QQ

7. The Applicant and the Staff have provided independent justifications, each supported by substantial credible evidence, for a conclusion that the Applicant's exemption request to use a PSHA methodology and a 2,000-year DBE is authorized by law, will not endanger life or property or the common defense or security, and is otherwise in the public interest. The different justifications that have been pro-

vided in support of Applicant's request complement each other and serve to increase the reliability of the evidence propounded in support of the exemption request. It is well established that an Applicant may rely on evidence presented by the Staff in support of its position on an issue. See, e.g., Florida Power and Light Co. (Turkey Point Plant, Unit Nos. 3 and 4), LBP-90-32, 32 NRC 181 (1990). Likewise, a Licensing Board may rely on any reliable, probative and substantial evidence in the record to reach its findings. Pacific Gas and Electric Co. (Diablo Canyon Nuclear Power Plant, Unit 2), ALAB-254, 8 AEC 1184 (1975).

8. The Applicant has shown reasonable assurance that the grant of its request for an exemption from the requirements of 10 C.F.R. § 72.102(f) to allow its use of a PSHA to establish the design earthquake ground motion levels at the proposed PFS Facility based on a 2,000-year mean return period design basis earthquake is authorized by law and will not endanger life or property or the common defense and security and is otherwise in the public interest.

V. CONCLUSION

For the reasons stated in the above proposed findings of fact and conclusions of law, the Applicant respectfully requests that the Board rule in favor of the Applicant on all aspects of Contention L/QQ.

Respectfully submitted,

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September 5, 2002

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
	)	
PRIVATE FUEL STORAGE L.L.C.	)	Docket No. 72-22
	)	
(Private Fuel Storage Facility)	)	ASLBP No. 97-732-02-ISFSI

CERTIFICATE OF SERVICE

I hereby certify that copies of the Applicant's Proposed Findings of Fact and Conclusions of Law on Unified Consolidated Contention Utah L/QQ (Seismic) were served on the persons listed below (unless otherwise noted) by e-mail with conforming copies by U.S. mail, first class, postage prepaid, this 5th day of September, 2002.

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