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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

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| In the Matter of: | ) | Docket No. 72-22-ISFSI |
| PRIVATE FUEL STORAGE, LLC (Independent Spent Fuel Storage Installation) | ) | ASLBP No. 97-732-02-ISFSI |
| | ) | October 16, 2002 |

# STATE OF UTAH'S REPLY TO PROPOSED FINDINGS OF FACT AND CONCLUSIONS OF LAW OF THE APPLICANT AND NRC STAFF ON UNIFIED CONTENTION UTAH L/QQ

The Board has before it about 800 pages of proposed Findings of Fact and Conclusions of Law ("Findings") filed September 5, 2002 by the State, PFS and the NRC Staff relating to the seismic hearings conducted over a nine week period from April through June 2002. The State does not attempt in this Reply to respond in detail to each finding and conclusion of PFS and the Staff with which it disagrees. The parties have stated their positions on issues in contention in their pre-filed testimony; during cross examination, re-direct examination, and rebuttal testimony; and in their proposed Findings. By this stage, the disagreements are plain.

Rather, this Reply will focus on key inconsistencies and conflicting statements that are evident in the proposed Findings filed by PFS and the NRC Staff by first providing an overview, then addressing key points in Soils, Soil Cement, Dynamic Analysis, Cask Stability and Seismic Exemption where the State takes particular issue with PFS, the Staff, or both of them.

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## I. Overview

PFS proposes to construct a facility to store the entire stockpile of spent nuclear fuel from commercial nuclear power plants in the United States at a site with major active seismic faults under and near the site; a site with very soft and compressible soil that is known to have excessive short term and long term settlement; with foundations that are no more than a series of concrete pads placed effectively at ground surface; and with storage casks free-standing on top of the pads. The design philosophy defies the basic laws of physics and offers a risky design most vulnerable to instability under seismic loading. The basis for such unprecedented design elements cannot be found from any past experience data or applicable laboratory data because there are none. The basis is merely from the prediction of a series of computer models of nonlinear dynamic analyses of the cask-pad system, *i.e.*, models which are known to be notorious in providing a wide range of responses depending on the choice of the input parameters. The foregoing cannot be ignored in any review of PFS's seismic design and its justification in support thereof.

### A. Evolution of PFS's Seismic Design and Site Ground Motions

In their findings, both PFS and the Staff claim there are many conservatisms in PFS's seismic design and analyses. The evolution of PFS"s seismic design in response to estimates of site ground motions belies those claims.

When PFS submitted its license application to the NRC in 1997, PFS estimated the deterministic (84$^{th}$ percentile) ground motions to be 0.67 g (horizontal) and 0.69 g (vertical). The nearest identified faults at that time were the East Cedar Mountain fault and the Stansbury fault, both about five to six miles from the site and capable of producing

magnitude 6.8 and 7.0 earthquakes. Consolidated Safety Evaluation Report, March 2002 ("Con-SER") at 2-46 (Staff Exh. C). Discovery in early 1999 of two formerly unknown faults dipping beneath the PFS site (West Fault and East Fault), capable of producing magnitude 6.4 and 6.5 earthquakes, led to revised deterministic (84th percentile) ground motions of 0.72 g (horizontal) and 0.80 g (vertical). Con-SER at 2-46 to -47.

On April 22, 1999, PFS applied to the NRC for an exemption to allow it to use a probabilistic seismic hazard methodology and a 1,000-year recurrence interval earthquake. PFS estimated ground motions for a 1,000-year mean annual return period event to be 0.40 g (horizontal) and 0.39 g (vertical). Con-SER at 2-45; Proposed PFS Exh. 247 at 6. On August 24, 1999, and after recent discussions with the NRC Staff, PFS formally informed the Staff it "has decided to use a 2,000 year recurrence interval" for the design basis earthquake, and noted, "[t]his will provide a greater margin of safety than the 1,000 year recurrence interval specified in [the initial exemption request]." Proposed PFS Exh. 248. At the time, PFS estimated 2,000-year mean return period event ground accelerations to be 0.53 g (horizontal and vertical). Id.

In 2001, PFS discovered it had severely underestimated the ground motions for the design basis earthquake; the peak ground accelerations were estimated to be 0.711 g (horizontal) and 0.695 g (vertical) and updated 84th percentile ground motions to be 1.15g (horizontal) and 1.17g (vertical). Con-SER at 2-46 & 2-34 respectively.

With these plain unvarnished facts that were developed over the course of the past five years, the question to ask is what changes has PFS made to its seismic design to accommodate such dramatic increases in ground motions? What was PFS's design concept

in April 1999 when it requested an exemption and the 1,000-year ground accelerations were estimated to be 0.39 g to 0.40 g? Or four months later when 2,000-year ground motions were estimated to be 0.53 g? How did PFS's design change when it discovered ground motions increased by thirty-five percent to 0.7g? How can PFS now claim that its design is "conservative"? Why is it that since September 2000 the Staff has admitted the reason for PFS requesting an exemption is that estimated deterministic ground motion values exceed the SAR proposed design values, yet the Staff now take the position that PFS's design is adequate? SER dated September 2000 at 2-34, Staff Exh. NN. At the time this statement was first written, ground motions estimated under the deterministic methodology for the PFS site were 0.72 g (horizontal) and 0.80 g (vertical).[1] Id. at 2-33. And why is it that the Staff in September 2000 considered Geomatrix's PSHA "possibly conservative," yet in 2001 Geomatrix had to go back and revise ground motions upwards by thirty-five percent. SER (Sept. 2000) at 2-36. The answers to these questions point out the fallacy in PFS's and the Staff's current position – in particular, claims that PFS's facility can withstand vibratory ground motions from a 10,000-year earthquake. If that were the case, then there is no need at all for PFS to request an exemption to allow it to use a lower design basis earthquake than permitted under the regulations.

PFS initially chose to use soil cement under and around the pads as a way to use the

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[1]The September 2000 SER states: "Taking into account these newly discovered faults with the DSHA methodology, the peak horizontal acceleration and peak vertical acceleration values from the seismic event would be 0.72 g and 0.80 g respectively. These values exceed the SAR proposed design values." SER (Sept. 2000) at 2-33, Staff Exh. NN. In this version of the SER, the Staff "determined that a 2000-year return period is the appropriate value for the PFS Facility site." Id. at 2-40.

eolian slits and to eliminate bringing in structural fill as well as to provide a road base between the columns of pads for the cask transporter. Tr. (Trudeau) at 6291, 11237-28. In 2001, PFS's Safety Analysis Report, Rev. 22, contained significant changes because of the predicted thirty-five percent increase in site ground accelerations. The increase in ground accelerations also necessitated major revisions to PFS's seismic design calculations for sliding, overturning and bearing capacity failure of the storage pads and the Canister Transfer Building ("CTB"). Revision 22 of the SAR; Revisions 7, 8, and 9 of the pad stability calculations[2]; and Revisions 4, 5 and 6 of the CTB stability calculations[3] formed the basis of Contention Utah QQ and two modifications thereto. The major design changes to the storage pads were, first, to reduce the stiffness and depth of soil cement under the pads (*i.e*, to create a one to two foot cement-treated soil mix), and second, to increase the stiffness of soil cement around the pads. Initially, PFS relied upon the passive resistance of soil cement around the pads to resist sliding but later abandoned reliance on passive resistance for its base case. As to the changes in the CTB, in 2000 PFS added a shear key to the building perimeter and in 2001 slightly increased the dimension of the foundation mat and added soil cement extending out one building dimension and five feet deep around the building. *See* State Findings ¶¶ 172, 174; PFS Exh UU, Calc. No. G(B)-04, Rev. 9 at pp. 5-8, and 15-16; and PFS Exh. VV, Calc. No. G(B)-13, Rev. 6 at pp. 3-6; Ostadan/Bartlett Tstmy,

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[2] Stone & Webster calculation G(B)-04, *Stability Analyses of Cask Storage Pads* (PFS Exh. UU).

[3] Stone & Webster calculation G(B)-13, *Stability Analyses of Canister Transfer Building* (PFS Exh. VV).

Post Tr. 7268 at 5-6; Private Fuel Storage, LBP-01-39, 54 NRC 497 (2001); and Contention Utah QQ. In essentials, PFS's seismic design has not changed significantly, yet PFS repeatedly claims in its Findings that its seismic design has extra conservatisms built into it to absorb any and all of the deficiencies raised by the State.

## B. Evidence to Support PFS's Analysis

PFS presents evidence in this case as if it were requesting a license to build a small experimental facility. Instead, PFS is requesting a license from the NRC to construct and operate a nuclear storage site that will be capable of storing the entire current inventory of commercial spent nuclear fuel in the United States. But PFS's design is a function of the least cost and the most aggressive schedule possible. It is a design that is based on a dearth of testing, performance or precedential data relating to fundamental concepts embedded in the design. The lack of hard factual data permeates PFS's engineering calculations and analysis but this does not hold PFS back from its claim that the design is "conservative." On close inspection, there is little in the record to support PFS's inflated rhetoric.

The schedule and cost-driven design has led to meager testing and analysis and lack of data in many key areas. A few examples illustrate the point. PFS has taken one 3 inch diameter sample from the 51 acre pad area to ascertain the undrained shear strength of the foundation clay soils for pad sliding analysis and is now attempting to rationalize why that one sample is from the weakest soils in the 51 acre site. Drilling a few more boreholes, and collecting and testing samples could have put facts in evidence to satisfy this issue – or it could have undermined PFS's design value and its seismic design concept. Another example is PFS's soil cement testing and bonding study, which PFS says will take about eight months

to complete. The program was supposed to be completed by March 2001 and has been on hold since at least the beginning of this year. Trudeau Depo. Tr. at 71-72 (State Exh. 108). Again, PFS could have developed facts to place in evidence to determine whether this precedent-setting application is achievable, but PFS insists that it should be given the privilege of an NRC license; then it will figure out whether its design concept will work or not. A final example is PFS's unvalidated nonlinear computer model to predict the seismic behavior of the unanchored HI-STORM 100 casks. PFS could have obtained experimental test data through shake table testing to validate at least part of its computer model. In this instance, however, there is a downside to this experimental approach because if PFS's nonlinear analysis does not hold, then the entire basis of PFS's design evaporates.

### C. "Newness" of PFS's Seismic Design

PFS seems to think that the State has complained about the "newness" of PFS's seismic design. PFS Findings ¶¶ 244-251. That is not the State's complaint. The State's complaint with PFS's seismic design is that a mass of about 1440 tons from eight very tall cylindrical unanchored casks (with a large height to width ratio), each containing 10 metric tons of spent nuclear fuel, will sit on a 30 foot by 67 foot concrete pad that is only three feet thick, without any other foundations; where there will be only a very small amount of Portland cement mixed into silts due to other design constraints; and all of the foregoing will be placed on very soft compressible clays that are expected to resist 0.7g ground accelerations (or accelerations of 1.15g or more if PFS were to follow current regulations). PFS believes a veneer of soil cement and cement-treated soil around and below the pads is sufficient to provide seismic stability to the foundation while in all major seismic areas in the

United States extensive foundation upgrades are implemented to seismically upgrade the foundations of buildings and infrastructures. *See e.g.*, Tr. (Ostadan) at 7345. The State's complaint is that PFS's seismic design has no experience data to back it up. It has no test data to back it up. It has no precedent to rely upon. It has only a nonlinear computer model – developed by the company who will sell 4,000 storage casks and other accouterments to the Applicant – that is extremely sensitive to the modeler's choice of input parameters, and one that has not been validated with any test data to predict the dynamic performance of PFS's seismic design from strong ground motions. There is no room for error. There is only the hope that the casks will perform as the cask seller predicts. That is the State's complaint.

PFS announces that it is not the first away-from-reactor ISFSI to be licensed by the NRC. PFS Findings ¶ 245. Mr. Guttmann testified (Tr. at 7044) that three other "away-from-reactor" ISFSIs have been licensed by NRC (Fort St. Vrain, INEEL, and GE Morris), but they do not offer any precedent for PFS's private centralized 4,000 unanchored cask storage facility. . Fort St. Vrain is located on the site of a shut down reactor and is owned by the U.S. Department of Energy; INEEL was built to store Three Mile Island fuel, is owned by DOE and located on a 900 square mile federal reservation; and GE Morris was licensed prior to enactment of Part 72 and fuel is not stored in dry casks but in wet storage pools. Tr. (Guttmann) at 7044, 7068-69. PFS also points to San Onofre ISFSI for the proposition of using unanchored casks. PFS Findings ¶ 249. The San Onofre ISFSI uses a completely different cask system than the free-standing cylindrical HI-STORM casks. In contrast to the unanchored tall cylindrical HI-STORM casks, the casks at San Onofre are large concrete

rectangular modules in which the canister is stored horizontally, and the casks are secured together in groups of three to improve resistance to strong ground motions. Tr. (Luk) at 7055.

PFS also says it is not the first or only facility to use shallow concrete pads or unanchored HI-STORM casks. PFS Findings ¶ 245. It is true that the Hatch ISFSI uses unanchored HI-STORM casks. Reference to the seismic design of the Hatch ISFSI, however, is hardly convincing: ground accelerations at Hatch are 0.15 g (horizontal) and 0.1 g (vertical), whereas at the PFS site, ground accelerations under the reduced seismic standard are 0.711 g (horizontal) and 0.695 g (vertical). Tr. (Luk) at 6915. If PFS has to design its facility to current regulatory standards, the ground accelerations would be about 1.15g (horizontal) and 1.17g (vertical). Con-SER at 2-34.

Part of PFS's newness defense relates to soil cement where PFS says there is nothing inherently wrong in an application of soil cement being new and that new applications are being found all the time for soil cement. PFS Findings ¶ 68. What the State objects to here is PFS's use of soil cement without doing testing and analysis prior to obtaining a license. *See* Section III, Soil Cement, *infra*. The State's expert testified that when new uses of soil cement are first introduced, there can be massive failures and problems in the early stages. Tr. (Mitchell) at 11194-95, 11269. Again, it is not the newness of the design that troubles the State but PFS's insistence that it can obtain a license from the NRC without doing a thorough analysis of its untested seismic design concept.

## D. "Beneficial Effect" of Sliding

Throughout its Findings, PFS refers to the "beneficial effect" of sliding – whether it

is sliding of the storage pads, the storage casks, the CTB foundation or even the soil cement surrounding the storage pads. PFS Findings ¶¶ 209, 213, 215, 217, 249, 265, 311, 315, 317, 328. PFS is enamored with sliding because it only views sliding as a way to reduce the seismic forces acting on a structure. In fact, Dr. Soler testified that from the selfish point of cask stability, the more sliding the better. Tr. (Soler) at 10658. PFS's view is one-sided because it gives no consideration whatsoever to the fact that sliding is rarely used in earthquake engineering design. Tr. (Ostadan) at 7345. What is even more startling and disappointing is that a regulatory agency would readily embrace sliding as predicted by a nonlinear computer model as a legitimate element of seismic engineering design. *See* Staff Findings ¶¶ 4.50, 4.127, 5.70, 5.99, 5.105, 5.108, 5.131, 5.182, and 5.184.

The entire seismic design of the PFS project is predicated upon the needs of Holtec's cask stability and tip over calculations. Tr. (Ostadan) at 7350. It is these two Holtec calculations that drive the foundation and soil cement seismic design of the storage pads and the CTB, and the structural seismic design of the storage pads (*i.e.*, rigid under some conditions but flexible under others). It is the Holtec cask stability calculation alone that predicts the performance of the casks under seismic excitation. It is this calculation – a dynamic nonlinear computer model – that produces the seismic forces that are (or should have been) used in the stability analyses for the pads. Tr. (Bartlett) at 7333. There are no other backup calculations or other ways of confirming the performance of PFS's unprecedented seismic design. Tr. (Ostadan) at 7341. The unconventional design proposed by PFS does not allow for comparison to any test data, performance data, or other conventional seismic designs. The entire seismic design offers no redundancy and rests on a

computer model that has not been validated with any experimental data, that is extremely sensitive to the selection of input parameters chosen by the modeler, and that has been developed and modeled by the cask seller who stands to gain a substantial financial windfall if the PFS facility is licensed.

PFS and the Staff focus on sliding as a means of dissipating energy. But that is not the critical question. As to cask sliding, the more appropriate questions are whether PFS should take full credit if the casks slide as a philosophy for design and whether the predicted sliding is accurate with respect to the input parameters. Tr. (Ostadan) at 7349. PFS believes that its nonlinear analysis alone is capable of predicting the complex seismic response of structures, defying the earthquake engineering community's wisdom in providing redundancy, considering the uncertainties, and relying on experience performance data in design rather than computer models alone. Furthermore, those forces that Holtec has derived from its nonlinear analysis that are (or should have been) used in the pad and CTB stability analyses are necessarily reduced because Holtec's black box analysis predicts that the casks will slide and, as PFS and the NRC Staff repeatedly inform us, sliding dissipates energy. Again there are no checks and balances in the design philosophy. For example, nowhere is there any analysis of the casks subject to horizontal and vertical design motions that are analytically pinned to the pads, whereby a comparison can be made of how much the forces have been reduced by Holtec's smooth sliding design model. Tr. (Ostadan) at 10291.

The State will not belabor its discussion again of base isolation systems but refers the Board to State Findings ¶¶ 183 to 187, wherein it is readily apparent that only for a genuine

engineered system, and after computation of one hundred percent of the forces, is a twenty percent reduction in forces acceptable. Base isolation systems, unlike PFS's seismic design, have some performance data but not enough as to allow a full reduction in seismic forces on the foundation system. Tr. (Ostadan) at 10290-91. Significantly, PFS's seismic design is not an engineered base isolation system.

Furthermore, in response to the question whether it is good engineering practice to design a structure that will slide, PFS's witness, Mr. Trudeau testified: "[I]t would benefit the performance of the casks atop these pads if we permitted them to slide. So the answer would be no to your question here. We've heard Dr. Ostadon [sic] speak about base isolated structures. Those are clearly designed to have sliding occur underneath their foundations. so the answer to your question is no." Tr. (Trudeau) at 10930. Mr. Trudeau went on to testify that he has never designed a base isolation structure and did not know the details of how much credit those engineered base isolated structures could take for sliding. Id. What this shows is the person responsible for the pad sliding analysis cannot justify a design concept that allows sliding. Yet, PFS ignores this testimony and promotes pad sliding as a positive seismic design feature.

PFS makes much of the proposition that "the storage cask design does not 'control' sliding, but merely allows it to occur, and the fact that sliding may occur in a 'uniform and controlled manner' is not a design requirement, but the result predicted by the cask stability analyses conducted by Holtec." PFS Findings at 55-56 (*emphasis in original*); *see also* id. ¶¶ 247-48. PFS makes the same argument about pad sliding – it is a consequence not a feature of the design. PFS Findings ¶ 249. With all due respect, whether it is a consequence or a

feature of PFS's seismic design, what is important is "how it really happens out there when the earthquake comes." Tr. (Ostadan) at 7315. And what we have in front of us is a black box prediction that the casks will slide smoothly and in a controlled manner and it is that prediction alone that PFS is relying upon for its seismic design to perform during an earthquake where the design will need to withstand ground motions of at least 0.7 g.

PFS has failed to acknowledge any of the reasonable and legitimate technical inadequacies voiced by the State about PFS's seismic design and analysis; instead, PFS resorts to the beneficial effects of sliding as a means to answer them. The following are but a few examples: even if there is pad to pad interaction, PFS's claims that collision between the pad and the soil cement frame has a beneficial effect on the stability of the casks (PFS Findings ¶ 316); even if PFS erroneously used pga in the pad sliding analysis, pad sliding would be beneficial to cask stability (id. ¶ 328); and even though PFS did not understand the State's concern during the hearings about the load path from pad to pad interaction and accumulation of loads on one pad (Tr. (Ostadan) at 10680-82; 10696-98), PFS now claims that if such effects cause pad sliding it could be beneficial to the stability of the casks. PFS Findings ¶¶ 311, 317; *see also* Staff Findings ¶¶ 4.50, 4.127, 5.70, 5.105, 5.184 (casks are less likely to tip over if the pads are free to do some sliding).

PFS also resorts to the beneficial effects of sliding in an effort to prop up Dr. Luk's model. PFS Findings ¶¶ 209, 213, 215, 217. In particular, PFS claims that instead of using Holtec's limiting modulus of less then 75,000 psi for cement-treated soil under the pads, Dr. Luk's use of an obviously incorrect modulus is a good thing because it is "a conservative design element that addresses, inter alia, the State's concern about underestimating forces

transferred to a cask" and it "demonstrates that sliding of the pads is beneficial to cask stability." PFS Findings ¶ 217. Lost in PFS's hyperbole is the fact that we are dealing with a nonlinear dynamic analysis of earthquake conditions and one that is extremely sensitive to input parameters. By using the wrong input parameters in a complex dynamic analysis, it is irrational to claim that the outcome will be favorable to PFS's design – the outcome simply is unknown. Examples such as this of PFS's overreaching effort to deflate the State's claims should tip the scales against PFS when the Board weighs whether PFS's Findings are based on substance or puffery.

PFS twists Dr. Bartlett's testimony relating to Dr. Luk's model as supporting its hypothesis that sliding is a beneficial condition during an earthquake because it increases cask stability. *See e.g.*, PFS Findings ¶ 215. That is not Dr. Bartlett's testimony. Dr. Bartlett's testimony on this point is unequivocal: it would be bordering on negligence in the practice of earthquake engineering to design a structure to slide. Tr. (Bartlett) at 7656; *see also* DOE Standard 1020 which recommends anchorage as one of the most important factors affecting seismic performance. Further, contrary to the implications in PFS's Findings, Dr. Ostadan offered testimony similar to that of Mr. Bartlett. Tr. (Ostadan) at 7335-36; 7340-46; 7350-53.

As an expert of international repute and one who has spent his career in earthquake analysis, Dr. Ostadan testified that foundations, especially for nuclear facilities, are never designed to allow sliding as a design philosophy or as a consequence of a design basis earthquake because they must be designed with adequate margins of safety. Tr. (Ostadan) 10675-76; 10692-93. Because of the many uncertainties and unknowns of both the capacity

of the system and demand from the earthquake and other forces, earthquake engineering uses factors of safety as a measure to ensure there are margins against these uncertainties and unknowns. Id. at 10676. PFS and the Staff seem to think it is a trivial matter if the foundations slide – whether as a design philosophy or as a consequence of an earthquake. By contrast, Dr. Ostadan testified: "As a foundation engineer, I certainly do not appreciate ignorance of how good or bad the foundation could be. I think [the] foundation is always an important component of any building design, and like any other engineering design, it needs to be treated properly, designed adequately to make sure it performs to its function." Id. 10675-76.

In the world of earthquake engineering, the Board's acceptance of PFS's sliding design concept would be a precedent-setting step that defies seismic engineering practice and follows in the foolhardy footsteps of PFS and the Staff, and their witnesses, who have limited experience in earthquake engineering and soil-structure interaction analysis. The uncertainties and unknowns of soil-structure interaction, the capacity of the soils, soil cement, and the structures are too tenuous to cavalierly adopt a concept that is antithetical to standard earthquake engineering practice. In its evaluation of PFS's seismic design and analysis, the Board should soundly reject PFS's unproven and unprecedented use of sliding for seismic design.

**E. "Conservatism" in PFS's Design.**

During the hearings, the Board analogized design margins to how close one was to a cliff or precipice. In its Findings, PFS attempts to demonstrate its seismic design passes muster by bringing its design to the edge of the precipice (*i.e*, the factors PFS could have

used in its analysis), then stepping back an inch or two (*i.e.*, the factors it used in its analysis) and claiming those margins show "conservativism" in its design. The Board should not be misled into accepting such a bold and unsubstantiated approach to seismic design and analysis. Moreover, this is the first seismic ISFSI design to come before the NRC for a high seismic site. Whatever decision the Board in this proceeding, it will set precedent for other seismic sites. Therefore, the Board should assure itself that it is on sound footing in reaching its decision.

PFS's use of the term "conservative" permeates its Findings and is often used as a substitute for data, analysis or other quantifiable evidence to overcome the State's complaints. Moreover, PFS's stretch to find conservatism is never juxtaposed with the lack of conservatism that is present in PFS's design and analysis. Attached hereto is a sampling of where PFS has used its concept of conservatism as a substitute for providing evidence to support its claims, and the State's response thereto.[4] The mere use of the term "conservatism" without any quantification offers the Board no assurance that PFS's design is not teetering on the edge of the precipice and that it has acceptable margins of safety to withstand a design basis earthquake of 0.7g – or greater if PFS's exemption request is not granted.

## II. Soils

The State's Findings on Soils detail the inefficacies in PFS soils characterization and testing. *See* State Findings ¶¶ 1-65. The State does not repeat those Findings or address each

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[4] *See* Attached table, "Examples from PFS Findings: PFS's Reliance on Conservatism & State Comment."

and every Finding filed by PFS and the Staff. Rather, the State focuses on the overstatements, misstatements and otherwise egregious aspects of the other parties' Findings.

## A. PFS Boring and Laboratory Program

Both PFS and the Staff argue that the initial borings performed in 1996 at the PFS site established the reasonable uniformity of soil properties across the pad area and that later CPT data, obtained in 1999, confirm that uniformity. PFS Findings ¶¶ 13, 17; Staff Findings ¶¶ 4.19, 4.29, 4.35. The initial borings established a two layer system, with layer 1 consisting of 25-30 foot thick relatively compressible top layer, and layer 2 consisting of denser sands. Con-SER at 2-55. Rather than confirm the "uniformity" of site soils, the CPT data further defined the layer 1 soils into four sub-layers (1A, 1B, 1C and 1D) and identified the upper Bonneville clays, Layer 1-B, as the critical layer for engineering analysis. Con-SER at 2-56; PFS Exh. 233; PFS Findings ¶ 22. The five layer system is not in dispute. What can be drawn from the change in a two layer system to a five layer system is that the PFS site is not as uniform in the vertical direction as first contemplated.

The PFS soils investigation and laboratory analysis has progressed from its incipient investigation in 1996, but it is still inadequate to show soil conditions for the upper Bonneville clays (Layer 1-B) are adequate for foundation loadings. PFS and the Staff confuse the record by either relying on data from other layers or focusing on the quantity of data collected and other types of laboratory tests that are irrelevant to the issues in contention. For example, the Staff believes subsurface data Geomatrix obtained in 1999, as part of a geologic fault evaluation study, confirm the horizontal uniformity of the upper 30

feet of the subsoil. Staff Findings ¶ 4.19; *see also* PFS Findings ¶ 26.[5] At issue is the undrained shear strength of the upper Bonneville clays. *See e.g.*, State Findings ¶ 15; PFS Findings ¶¶ 22, 36. The only data germane to the pad sliding analyses are the geotechnical borehole data and laboratory direct shear test data. These data formed the basis of the calculations in PFS's pad sliding analysis. PFS Exh. UU at 9-11.

Further, the total number of samples taken at the PFS site is not the relevant inquiry. Staff Findings ¶ 4.20 (two thirds of 33 undisturbed samples were taken from layer 1-B soils). The important question to ask is how many undisturbed samples from the pad emplacement area were used for direct shear testing. The answer is a sample of one from a single borehole. Moreover, PFS cannot claim, as it does, that this one sample was taken from the weakest layer because this one sample exhibits high void ratio. PFS Findings ¶ 38; *see* State Findings ¶¶ 38-44. There is nothing in the record that establishes undrained shear strength is solely a function of void ratio. There are other factors, such as clay and moisture content, and amount of cementation and dessication that also contribute to undrained shear strength. State Findings ¶¶ 92, 416; Tr. (Bartlett) at 7725-26. Furthermore, the Staff points out that five undisturbed samples were taken and tested from the upper Bonneville clays, layer 1-B.. Staff Findings ¶ 4.37. Therefore, five samples total were available for the critical layer in the entire 51 acre pad area. From this paucity of samples, the Staff advocates that the highest void ratio is found in the northeast quadrant. Id. At best, there would be only one or possibly two samples in any given quadrant; it is possible that certain quadrants may not

[5]The geologic plates referred to in this ¶ 26 are not in evidence.

have been sampled at all. From this acute undersampling, the Staff draws its unsupportable conclusion.

Contrary to the Staff's Findings ¶ 4.22, PFS has not completed its testing program to establish the shear strength of site subsoils. The Staff fails to acknowledge that PFS intends to perform, post license, direct shear tests of site soils with varying amounts of cement and moisture. State Findings ¶ 88; SAR at 2.5-111 (PFS Exh. JJJ). The State agrees that this testing is essential but vigorously disputes that it should be put off until PFS obtains the privilege of obtaining a license from the Commission. *See* Section III, Soil Cement, *infra*.

PFS claims its boring program conformed "to the general guidance in Reg. Guide 1.132 (PFS Exh. 234)" and that because of the "uniform site conditions, there is no need for a denser set of borings." PFS Findings ¶ 27. The plain reading of the following from Reg. Guide 1.132, Section 4.a, General, does not support this finding: "Methods of conducting subsurface investigations are tabulated in Appendix B to this guide, and recommended guidelines for the spacing and depth of borings for safety-related structures, where favorable or uniform geologic conditions exist, are given in Appendix C." Reg. Guide 1.132 at -3. The spacings listed in Appendix C are for sites with favorable or uniform geologic conditions. Hence, the values listed in Appendix C are already the maximum spacings for favorable conditions. As detailed in State Findings ¶¶ 23-27, PFS has not met the minimum spacing of one borehole per 100 linear feet. PFS's claim that a denser set of borings would not have yielded any different results from those that PFS obtained is mere speculation and does not withstand scrutiny. Significantly, PFS itself revised the layering of the upper Bonneville deposits from 2 layers to 5 layers based on more data obtained in subsequent

testing. Hence, more data led to a revision of the site layering. The Staff attempts to confuse the issue by focusing on the density of the CPT soundings in the pad emplacement area. Staff Findings ¶ 4.25. Dr. Bartlett's pre-filed testimony, however, is clear: he computed the density of both the CPT soundings (total of 37) and the boreholes (total of 9) in the 51 acres pad emplacement area to be one per 221 feet. Bartlett Tstmy (Soils), Post Tr. 11822 at 6. There are two separate and distinct concerns. One, the density of the borings (both boreholes and CPT soundings) performed by PFS is inadequate to demonstrate the horizontal uniformity of site soils and the density does not comply with Reg. Guide 1.132. Two, the representativeness of the samples taken from the boreholes (not the CPT soundings) to represent the undrained shear strength of the upper Bonneville clay is utterly inadequate. *See* State Findings ¶¶ 23-53.

B. Cone Penetrometer Testing Data

The dizzying array of uses PFS and the Staff suggest for the CPT data is perhaps one of the most confusing issues before the Board on Unified Contention L/QQ, Part C, Soils. PFS and the Staff attempt to use the CPT data as a palliative to answer many of the State's concerns. The Staff and PFS claim that the CPT data can be used as a surrogate for other testing or confirmation such that there is no need for further testing. To overcome the inadequacies in PFS's site characterization and laboratory testing, PFS and the Staff advocate using CPT data for support of the following: uniformity of site layering (PFS Findings ¶ 17; Staff Findings ¶¶ 4.19, 4.29 ); horizontal uniformity of the upper Bonneville clays (id. ¶¶ 24, 42; ¶ 4.19); lack of weak zones or unstable soils and the allegation that PFS has conducted "continuous" sampling (id. ¶ 33; ¶¶ 4.33, 4.35); and undrained shear strength values (id. ¶ 39;

¶¶ 4.36, 4.44, 4.46-47). *See* State Findings ¶¶ 23-53.

The number of times PFS and the Staff reach for the CPT data to answer the State's criticisms is a good indicator of the inadequacies in PFS's program. It must be remembered that PFS's pad stability calculation – G(B)04, Rev. 9, PFS Exh. UU – rests entirely on the geotechnical borings and laboratory testing, not on the CPT data. Most critically, the pad sliding calculation is based solely on results from direct shear testing, not CPT data. The only mention of CPT data in the "Geotechnical Properties" section of the Pad Sliding Calculations is the following: "The results of the cone penetration testing, presented in ConeTec (1999) and plotted in SAR Figure 2.6-5, Sheets 1 to 14, illustrate that the strength of the soils in the upper layer are much greater at depths below ~10 ft than in the range of ~5 ft to ~10 ft, where most of the triaxial tests were performed." PFS Exh. UU at 10. That is the sole reference to the CPT data and it relates to a fact not in dispute. The remainder of the geotechnical properties discussion is specific to triaxial testing and direct shear testing of soil samples collected at the site. PFS Exh. UU at 10-11 and Attachment C.

Soils characterization and laboratory testing are yet other areas in which PFS has not conducted a comprehensive seismic analysis and ones in which PFS and the Staff use post hoc justification in lieu of adequate data and analysis. The State does not dispute that in some circumstances, CPT data can be used to correlate other test data but there must be a defensible statistical analysis for making such a correlation. Tr. (Bartlett) at 11939, 12010. Also, such a statistical analysis requires that one consider the horizontal and vertical variation of the soils. Id. Dr. Bartlett made such an attempt in State Exh. 99, wherein he plotted the tip resistance in each CPT sounding, but as PFS did not provide the State with an electronic

copy of its CPT data, his hand drawn plots of the data were a reasonable approximation of the CPT plots provided in the ConeTec Report. *See* State Findings ¶ 48. An appropriate statistical assessment of the CPT data would be to determine the maximum, minimum, average and standard deviation of the CPT tip stress and sleeve stress for each of the four sublayers in layer 1 (*i.e.*, layers 1A, 1B, 1C and 1D) at each CPT sounding location. A table of these values for all CPT soundings would allow one to assess lateral variability within each of these layers from location to location. Instead, PFS and the Staff would have the Board merely look at the color diagram of the CPT data, PFS Exh.233A, and eyeball whether the wiggly lines down each vertical column representing cone penetrometer tip resistance and sleeve resistance were consistent or not. *See e.g.*, Tr. (Ofoegbu) at 11808-10; *see also* Tr. 11885-86 (Mr. Trudeau's explanation to the Board of the scale and orientation of PFS Exh. 233A). The CPT plots in PFS Exh. 233A are not a picture postcard; they are a representation of technical data than cannot readily be interpreted by the limited testimony presented to the Board. The State does not believe that the Board has the information in the record or experience in soil mechanics sufficient to evaluate PFS Exh. 233 merely by looking at the shape of the squiggly lines in each CTP soundings. The CTP plots as presented in the hearing do not overcome PFS's failure to determine the horizontal consistency of the upper Bonneville clays or weakness in those clays through appropriate boring spacing and continuous sampling. State Findings ¶¶ 23-37.

The CPT data cannot be used directly to obtain shear strength values. State Findings ¶¶ 32-33, 45-47. In its Findings, PFS, without explanation, states that undrained shear strength can be derived from CPT data. PFS Findings ¶ 39. The Staff refer to an $N_k$ factor

it says was developed by ConeTec. Staff Findings ¶ 4.46; *see also* State Findings ¶¶ 46-47. There is a value for $N_k$ in the ConeTec report but the statistical analysis to develop the $N_k$ factor from the CPT data is not presented either in the report or in any other document reviewed by the State. Tr. (Bartlett) at 11942-44. Because of this void in the record, it is impossible to judge the accuracy, reliability and robustness of ConeTec's reported $N_k$ value. The Staff refers to samples from borings and compares them the corresponding cone tip resistance values observed in the nearest CPT sounding, but the paired borings/soundings are tens and frequently hundreds of feet apart. Staff Findings ¶ 4.46; State Findings ¶¶ 25, 34-35. The attempt to "correlate" data separated by these distances is totally ineffectual because it unabashedly ignores the potential for horizontal variability.

### C. Staff's Position on Shear Strength Values

The Staff takes the curious position of validating PFS's post hoc justification of the representativeness of the shear strength value used in the pad stability design calculation by claiming that all PFS needed to show was that the undrained shear strength value used in that calculation "'was less than the value they could have used based on an interpretation of the CPT data.'" Staff Findings ¶ 4.47, *quoting* Tr. (Ofoegbu) at 11791. As already discussed, shear strength values cannot be directly obtained from the CPT data; they can only be correlated with laboratory test data. Second, if PFS and the Staff believe the site soils are so uniform, at a minimum, the Staff should have required PFS to use the lower undrained shear strength value obtained from the direct shear data from the CTB area. *See* State Findings ¶ 42 (shear resistance to vertical stress of 2 ksf was about 2.1 ksf for the pad area but 1.75 for the CTB).

The Staff also makes the argument that it is the "average" shear strength beneath the pads that is important because "the foundations for the cask storage pads . . . are such wide foundations that the superstructure loads are distributed over a large volume of soil." Staff Findings ¶ 4.48. Maybe the Staff believes that the pads will be locked together by the soil cement and act in unison – a proposition that the State adamantly disputes. Moreover, sliding analysis in highly layered deposits are not controlled not by the average but by the minimum shear strength value in a critical layer. Tr. (Ostadan) 7574-77; (Bartlett) at 11938. Furthermore, it is implausible that one would find an average value of 2.1 ksf, which is based on a sample size of one, especially when the CTB area has a value of 1.75 in one of the two samples tested in this area. As a final refuge to justify the shear strength value obtained from one sample, the Staff resorts to the beneficial effect of pad sliding to increase case stability. Staff Findings ¶ 4.50. This is a concept that is not accepted by the earthquake engineering community. The Board should not give any deference to the Staff's Findings ¶¶ 4.46 to 4.50.

### D. PFS's Conclusion

PFS concludes the Soils section of its Findings by proclaiming its approach to establishing the strength and other characteristics of the soils is "exceptionally conservative." PFS Findings ¶ 58. If taking one sample three inches in diameter from a 51 acre site for laboratory testing of shear strength is PFS's definition of "exceptionally conservative," then this label is meaningless. At a minimum, PFS cannot overcome that it took just one sample to determine the engineering properties of the critical soil layer it is relying upon to dissipate seismic loadings. On this basis alone the Board should reject PFS's license application.

## III. Soil Cement

### A. Soil Cement Precedent

The State's internationally acclaimed expert Dr. James K. Mitchell gave clear and compelling testimony, based on direct involvement in many of the projects PFS and the Staff are relying upon, that PFS's use of soil cement to resist seismic forces is precedent-setting. *See* State Findings ¶¶ 104-115. PFS addresses precedence in Findings ¶¶ 66 and 67 and the Staff in Findings ¶¶ 4.87 to 4.93. The only response to those Findings that is not obvious in the State's Findings is that the Staff's presentation of evidence that Dr. Mitchell agrees there is some similarity between the Boston Tunnel Project and the PFS project in that they both use soil cement to resist lateral loads. Staff Findings ¶ 4.92, n. 42; *see also* PFS Findings ¶ 67. The Staff, however, presents an incomplete picture of Dr. Mitchell's response to whether the two uses are analogous. The remainder of Dr. Mitchell's response shows that the two situations are not analogous: "The PFS application is a dynamic lateral loading, which is not the situation, for example, in the Boston Central Artery Tunnel, and it's not the situation in most retaining wall-type structures that involve soil cement." Tr. (Mitchell) at 11193; *see also* State Findings ¶ 108.

### B. Demonstration that PFS Meets Site Specific Investigations, Characterization, Laboratory Testing, and Analysis for Soil Cement

The central issue here is whether PFS should demonstrate now that its seismic design in the precedent-setting use of soil cement is achievable and that PFS has demonstrated that the site soils with added cement meet the requirements of 10 CFR §§

72.90[6], 72.102(c)[7] and (d), and 72.122(b)(1) and (2).[8] It is significant that for in situ soils, PFS has conducted site characterization, sampling, laboratory testing, and analysis – albeit not adequately – but when it comes to soils with added cement, PFS and the Staff insist that demonstration in meeting the requirements of the foregoing regulations can be deferred and verified post licensing. See e.g., PFS Findings ¶¶ 78-81; Staff Findings ¶¶ 4.97-100, 4.152.

Before the Commission may issue a license, it must find that the proposed site complies with 10 CFR §§ 72.90 to 72.130 (Subpart E). 10 CFR § 72.40(a)(2). One of those regulatory requirements is that "[s]ite-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading." Id. § 72.102(d). PFS is not using a foundation key to ensure that the full strength of the soils beneath the storage pads are engaged to resist sliding as would ordinarily be done because a foundation key would violate Holtec's cask tipover analysis. Calc. No. G(B)-04 at 15 (PFS Exh. UU). Instead, PFS is using cement-treated soil lifts glued together and glued to the underside of the pads and to the top of the upper Bonneville clays to transfer all of the seismic forces vertically down to the cement-treated soil and the underlying upper Bonneville clays. That is the foundation system that must resist and dissipate seismic forces;

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[6]General requirement for the Applicant to investigate and address site characteristics that may directly affect the safety or environmental impact of the ISFSI.

[7]Requires the Applicant to evaluate the site for soil instability due to vibratory ground motion. Section 72.102(d) is discussed *infra*.

[8]Relates to structures, systems, and components ("SSCs"), which must be compatible with site characteristics and environmental conditions associated with normal operations, and be able to withstand postulated accidents and earthquakes without impairing their capability to perform safety functions.

it is PFS's base case in Calc. No. G(B)-04, *Stability Analysis of Cask Storage Pads* (PFS Exh. UU).

The State makes the general claims that PFS's sampling and analysis are inadequate to characterize the site and PFS has not demonstrated that soil conditions are adequate to resist foundation loadings from the design basis earthquake. Unified Contention Utah L/QQ at C. Part of the specific issues in controversy in this proceeding is the State's claim that PFS has not shown: by site specific testing and dynamic analysis that the cement-treated soils will be able to resist dynamic loadings as required by 10 CFR § 72.102(d); that during an earthquake, cement-treated soil will perform as intended; that placement and construction of soil cement will not affect the native soils; and that it has correctly estimated the dynamic Young's modulus of cement-treated soil under dynamic conditions. Unified Contention Utah L/QQ at C(3)(b) through (e).

Under the Atomic Energy Act ("AEA"), the State is entitled to a meaningful hearing on issues in controversy. PFS and the Staff are attempting to truncate the State's Atomic Energy Act hearing rights by advocating that it is appropriate to defer, until some indeterminate date, PFS's testing, analysis, and implementation of its precedent-setting use of soil cement. However, in "any proceeding" for the granting of an operating license to a nuclear facility, "the Commission shall grant a hearing upon the request of any person whose interest may be affected by the proceeding." AEA § 189(a)(1)(A); 42 U.S.C. § 2239(a)(1)(A). The hearing must offer an opportunity for "**meaningful public participation.**" Union of Concerned Scientists v. NRC, 735 F.2d 1437, 1446 (D.C. Cir. 1984), *cert. denied* 469 U.S. 1132 (1985), *quoting* Bellotti v. NRC, 735 F.2d 1380, 1389 (D.C. Cir. 1983) (*emphasis in original*). A

meaningful opportunity to be heard means having the opportunity to be heard on "all material factors bearing on the licensing decision raised by the [hearing] requestor." Id. at 1443.

Rather than address, in a proceeding where the State has been admitted as an intervenor, the soil cement design concept and whether it can be implemented at the PFS site, the Staff intends to allow PFS to defer this demonstration. The Staff concludes that PFS's "commitments, when satisfied, provide adequate assurance that the cement-treated soil and soil-cement will perform their intended safety functions." Staff Findings ¶ 4.152. This statement, however, does not address whether PFS has complied with 10 CFR § 72.102(d) ("[s]ite-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading").

The only PFS commitments to support a licensing decision are contained in PFS Exh. JJJ, SAR, Rev. 22, Section 2.6.4.11. The soil cement discussion in PFS Exh. JJJ consists in large part of a recitation of PFS's design concept, such as how it will improve resistance to pad sliding; descriptions from the literature on soil cement bonding; precedent for using soil cement; and a general description of the dimension and unconfined compressive strength of the soil cement to be used at the site. SAR at 2.6-108 to -108b; 2.6-111 to -117. The only

"commitments" in the SAR are found on page 2.6-111[9] and on page 2.6-117,[10] and although the term "commitment" is not used, a description of PFS's soil cement processes to develop a proper soil cement mix are set out in three bullets on pp. 2.6-118 to -119. That is the sum and substance of PFS's "commitments" and the basis against which the Staff would need to conduct verification inspections post licensing to determine whether those commitment had been met.

The Staff approvingly states that PFS's soil cement laboratory testing program will be conducted by PFS's contractor in compliance with the laboratory testing engineering services scope of work ("ESSOW"), including QA Category 1 requirements. Staff Findings ¶ 4.99.[11] The ESSOW (PFS Exh. GGG), is between Stone and Webster and Applied Geotechnical Engineering Consultants, Inc. ("AGEC").[12] There is nothing in evidence that shows that AGEC's testing has been conducted or will be conducted in compliance with QA category 1

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[9]"PFS has committed to performing site-specific testing to confirm that the required interface strengths are available to resist sliding forces due to an earthquake... [and] to augmenting this field testing program by performing additional site-specific testing of the strengths achieved at the interface between the bottom of the soil cement and the underlying soils."

[10]"PFS has committed to perform direct shear tests of the interface strengths during the design phase of the soil cement to demonstrate that the required interface strength can be achieved, as well as during construction, to demonstrate that they are achieved."

[11]Staff Findings ¶ 4.99 references SAR at 2.6-109 which, written in the present tense, says that the testing program "is being performed" in accordance with the ESSOW and that the program "is being conducted in full compliance with Quality Assurance (QA) Category 1 requirements of the ESSOW." SAR at 2.6-109 (PFS Exh. JJJ).

[12]AGEC is a testing lab, not design engineers. Tr. (Wissa) at 10885. Consequently, Stone and Webster's contract with AGEC is only a small part of PFS's overall lab testing, bonding study, analysis and implementation program to demonstrate that its soil cement program will provide the seismic resistance to pad sliding.

requirements.[13] In fact, there is evidence that AGEC's substandard testing of PFS site soils is the reason, in part, why PFS's laboratory testing program is on hold, as evidenced by the following deposition testimony by Mr. Trudeau:

> Because we've received some results that have indicated that they [AGEC] didn't compact the test specimens properly. We've brought on board Dr. Anwar Wissa as an expert in soil cement to assist us in evaluating why this could have -- how this could have happened, what did they do wrong that would have caused the densities to be so low?
>
> They're supposed to be within 2 percent of the maximum density from the moisture density tests that are performed in accordance with ASTM D558, the standard test method for moisture density relations of soil cement mixtures. They were off by 8 percent or more in some of these specimens. So clearly specimens not compacted to sufficient density would not be expected to pass this durability test regimen.
>
> So that's where we are today. We've, as I said, brought Anwar Wissa on board to assist us in moving ahead. And we're currently involved in this litigation so we're not moving ahead on the lab testing, but we will sooner -- as soon as time permits.

State Exh. 108 at 72-73; *see also* Tr. (Trudeau) at 10976-77. Initially, Mr. Trudeau thought that AGEC "had a bad day" (State Exh. 108 at 73) but in hearing testimony Mr. Trudeau admitted that it is supposition why the tests failed, obviously suggesting that PFS has not made the effort at this stage to enquire further. Tr. at 10977.

The Staff maintains that PFS has proven its design "even without having completed the soil cement testing program." Staff Findings ¶ 4.98. Here the Staff takes a more adversarial approach than PFS's witness Paul Trudeau, who conceded in his deposition

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[13]If AGEC does not have a QA Category 1 program, the ESSOW allows AGEC to conform to the Engineers' Quality Assurance Program as an alternative. PFS Exh. GGG § 4.1; *see also* Tr. (Trudeau) at 10976. There is no evidence that AGEC testing has conformed or will conform to this alternate program.

testimony that only when PFS has completed its soil cement testing program and developed the appropriate soil cement and cement-treated soil mix, as well as successfully completed its cement-treated soil bonding demonstration, will PFS have proven its design. State Exh. 108 at 81.

The adequacy of the Staff's review to determine compliance with regulatory requirements is based, in part, on the following testimony by Dr. Ofoegbu:

> Well, first of all, I didn't allow; I don't have the authority to allow anything of any applicant. What we were tasked to do was review the design submitted, and that is calculations, material properties, and determine whether this design satisfies the regulatory requirement.
>
> Now, of course, in reviewing the material properties, we will look to see whether they have specified something that is, you know, out of this world that is not likely to be achieved, and that's what we did, and our finding based on the information we presented by the applicant and information available in the literature is that the design satisfies the regulatory requirement and that the material properties that are proposed are within the range of what is available in the literature.

Tr. at 11016; Staff Findings ¶ 4.98. The Staff also notes that PFS will be subject to criminal penalties if it makes material false statements. Staff Findings ¶ 4.97 at n. 43. Further says the Staff, if PFS "perceives a need" to change its testing procedures or demonstrate how SSCs may perform, it can do so by getting a license amendment. Id. ¶ 4.100.

The State's issue with the Applicant is not that it has or will make material false statements. The issue is whether PFS's design concept is achievable, not in the abstract, but with PFS site specific soils and whether it will meet the design intent that PFS and the Staff (and ultimately the Commission) are relying upon as the basis for licensing a 4,000 cask, high level nuclear waste storage facility. Moreover, by raising the specter of a license amendment

if PFS's "commitments" do not pan out, the Staff merely reinforces its position that it would rather not deal with substance in this proceeding where the State of Utah will have a fair opportunity to challenge it. Furthermore, to license a facility with the knowledge that the Applicant may need a license amendment to meet the basis for licensing is ample evidence that the basis for licensing is indeed faulty.

PFS and the Staff advocate that the Licensing Board can accept PFS's commitments in the SAR wholly in place of any evidence in this licensing proceeding that its design concept will function as intended and that the design parameters are achievable. PFS Findings ¶¶ 79-81; Staff Findings ¶¶ 4.98 -99; 4.152. Further, PFS claims: "State witnesses pointed to no regulatory rule, regulation or regulatory guidance that requires the Applicant to proceed with the soil cement testing program in advance of licensing . . ." PFS Findings ¶ 79. Whether PFS should conduct testing and analysis now and demonstrate that its design concept can be achieved and implemented is a legal question, which is next addressed.

It is well-established that "the mechanism of post-hearing resolution must not be employed to obviate the basic findings requisite to an operating license – including a reasonable assurance that the facility can be operated safely without endangering the health and safety of the public." Consolidated Edison Company of New York, Inc. (Indian Point Station, Unit No. 2), CLI-74-23, 7 AEC 947, 951-52 (1974). Indian Point further cautions that post-hearing resolution "should be employed sparingly and only in clear cases." Id. at 952. When there are "unresolved aspects" of a licensing review, post-hearing resolution is only suitable for "minor procedural deficiencies." Long Island Lighting Company (Shoreham Nuclear Power Station, Unit 1), LBP-83-57, 18 NRC 445, 543-544 (1983), *quoting*

Indian Point, CLI-74-23, 7 AEC at 951 (minor deficiencies in nonsafety-related equipment program can be resolved by the Staff post-hearing). 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* Indian Point, CLI-74-23, 7 AEC at 951.

According to the Commission, "the important question . . . is whether the NRC staff inspectors are expected to engage in "ministerial"-type compliance checks not suitable for hearings or are expected to themselves exercise a form of adjudicatory discretion. Private Fuel Storage (Independent Spent Fuel Storage Installation), CLI-00-13, 52 NRC 23, 33 n.3 (2000); *see also* Union of Concerned Scientists v. NRC, 735 F.2d 1437, 1449 (D.C. Cir. 1984). In ruling on whether PFS may rely upon a license condition and post license verification by NRC inspectors of PFS's customer service contracts to satisfy its financial assurance demonstration, the Commission held:

> To reconcile post-hearing verification of a license condition by the NRC staff with cases like Union of Concerned Scientists, Shoreham and Indian Point Station, we must insist that the [license] condition be precisely drawn so that the verification of compliance becomes a largely ministerial rather than an adjudicatory act – that is, the Staff verification efforts should be able to verify compliance without having to make overly complex judgments on whether a particular [customer service] contract provision conforms, as a legal and factual matter, to the promises PFS has made.

Private Fuel Storage, CLI-00-13, 52 NRC at 34.[14]

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[14]The Commission remanded the matter back to Judge Bollwerk's Licensing Board with instructions that PFS present a model service contract. The decision is pending on whether the draft model service contract developed by PFS and challenged by the State

As discussed previously, PFS is not applying for a licence to build an experimental facility. The PFS facility will licensed to construct 500 concrete pads on which it may store 40,000 metric tons of high level nuclear waste. PFS and the Staff expect this Board to approve a precedent-setting design concept for which there are no supporting site-specific investigations, data or analysis that the foundation system PFS is relying upon to resist seismic forces of 0.7g will function as contemplated. The State, and the public at large, expect this Board to eschew such a potentially arbitrary, capricious and unlawful scheme.

The State has met its burden of going forward and the Board should not by swayed by arguments from PFS and the Staff that the State has to demonstrate that abstractly PFS's goal is non-attainable. The standard is that, **before** the Commission may granted PFS a license, PFS must demonstrate that "site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading." 10 CFR § 72.102(d). The Staff proclaims that regulatory requirements, sections 72.102(d) and 72.122(b), can be satisfied by "an assessment whether a material with a specified property would be adequate for the proposed foundation loading, and whether the specified property is achievable for that material based on available information." Staff Findings ¶ 4.88. For this proposition the Staff cites to Dr. Ofoegbu's testimony at Tr. 11018. Id. Dr. Ofoegbu testified that it was not his decision to defer testing; rather, he testified that he was comfortable with deferral of testing because he was told that there was regulatory precedence for it. Id. at 11017. Dr. Ofoegbu could not cite to such precedence and it is

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satisfies the Commission's remand and 10 CFR § 72.22(e).

telling that the Staff did not cite to any such precedence in its Findings. Id. at 11018. Whether the Staff has allowed this practice in the past is not substantiated by the evidence, and, in any event, the Board should not endorse a Staff practice that excludes an Intervenor from exercising its legitimate hearing rights.

The Staff acknowledges that PFS has yet to provide site-specific data but says that PFS is confident the soil cement properties to stabilize soils in the foundation system design are achievable and that, notwithstanding the State's doubts, both PFS and the Staff are satisfied that it can be done. Staff Findings ¶¶ 4.132, 4.133. Dr. Mitchell, on the other hand, took the position "show me now."[15] Tr. (Mitchell) 11096. Dr. Mitchell testified:

> [T]here's a great deal hinging on the successful achievement of the properties, proper construction, and the conditions that are going to be required for the design that has been developed. To be in a position to know that you can do it is, I think, a much better position to be in, than to say you're going to be able to do it at some later time, and then have trouble.

Tr. (Mitchell) at 11096. Dr. Mitchell further testified, if he were a consultant to the project he would, in good faith, and as a good engineering practice, encourage PFS to make its demonstration now. Tr. 11100 (Mitchell).

PFS seems to think that the issue is only about time and money. Tr. (Mitchell) at 11100. It is not. Whether PFS's design intent is achievable with PFS site-specific soils is fundamental to the basis upon which the NRC will license this facility. This is not a small

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[15]The State notes that Staff Findings ¶ 4.98 omitted the fact that Dr. Mitchell's response was qualified by the question: "Is there anything you are aware of that necessarily precludes them [PFS] from coming up with the right mix of soil cement and cement-treated soil? I'm not asking whether they will do so, but is there anything that necessarily will keep them from meeting it?" To which Dr. Mitchell responded: "I am unaware of any at this point." Tr. at 11212.

matter. PFS needs first to separate plastic from non-plastic eolian silts, then achieve a concrete mix from the non-plastic eolian silts that is not too stiff and not too deep (1-2') that it violates Holtec's tipover analysis; attain a Young's modulus that is on the extreme low range of any reported literature value;[16] develop bonding strengths for at least four different interfaces[17] that will resist seismic forces from 0.7 g ground accelerations and enable the entire system (pads, cement-treated soil and soil cement) to move as a unit under seismic excitation; and recompact the critical soil layer PFS is relying upon to dissipate earthquake forces (the upper Bonneville clays) such that none of its shear strength is lost. All of this and more will be left to the post license verification and discretion of an NRC inspector to ensure that the unproven seismic design PFS has proposed is attainable. These are hardly unresolved aspects of minor procedural deficiencies referred to in Shoreham as suitable for post-hearing resolution.

Contrary to claims by PFS and the Staff, important substantive determinations must be made in order to assess whether PFS can achieve its soil cement design intent with site specific soils. Can the Board be sure, within acceptable bounds, how easy it will be for NRC verification reviews to determine compliance with the PFS's current commitments? As will be evident from the discussion below, the lack of precision and detail in PFS's commitment

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[16] *See* State Findings ¶ 99.

[17] A bond between the cement-treated soil lifts; a second bond between the cement treated soil and the upper Bonneville clays; a third bond between the cement-treated soil and the underside of the concrete storage pad; a fourth bond between cement-treated soil and soil cement. *See* SAR, Rev. 22 at 1.6-118 to 119 (PFS Exh. JJJ); and Calc No. G(B)-04, Rev. 9 at pp. 15-16, 115 (Fig. 8) (PFS Exh. UU).

and the judgments that will need to be made are not conducive to verification inspections.

The record is clear that AGEC did not accurately carry out durability testing of PFS site soils.[18] Tr. (Trudeau) at 10977. PFS has yet to ascertain why those tests failed. Could an NRC inspector verify such tests passed muster? If PFS still retains AGEC, is there any assurance it will competently carry out the remainder to the tests and will Staff inspectors need to make complex judgments in reviewing those results? Furthermore, if PFS brings Dr. Wissa on board to conduct site testing (or someone else of his caliber), PFS's characterization will need to be re-done. Tr. (Wissa) at 10980 ("you always want to vouch for your work, and it's hard to vouch for someone else's work"). In this case the first step would be to determine the variability of the soils that will be stabilized with cement. Tr. (Wissa) at 10863. Even where some data have been collected, tests performed, and analysis conducted, the State and PFS disagree on the variability of site soils – in this case the upper Bonneville clays. When it comes to site soils to be stabilized with cement, PFS and the Staff expect that the variability of the eolian silts that is the major component of the cement mix can be deferred to verification inspection. We do not imagine NRC inspectors will be prepared to merely rubber stamp whatever PFS puts in front of them and, therefore, they will need to make substantive determinations on the variability of site soils.

The record does not establish who PFS will retain to conduct its soil cement program in which at least the following will need to be done: laboratory testing of site soils;

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[18]Notwithstanding AGEC's poor testing performance, the Staff cite to the ESSOW between Stone and Webster & AGEC to exemplify that PFS's contractor will carry out the testing program in compliance with QA Category 1. Staff Findings ¶ 4.99

developing the correct soil-concrete-water mix; ascertaining whether the mix will achieve Young's modulus of less than 75,000 psi; testing various mixtures for bond strengths, etc. As the AGEC poor quality testing demonstrates, the caliber of the work performed will affect post judgment decisions by NRC inspectors. Also, it is impossible to predict now the percentage of cement that will be needed to be added to the soils. Tr. (Wissa) at 10985 (the amount of cement to add to soils is a function of a number of factors that Dr. Wissa is not now in a position to predict). Consequently, inspectors have no baseline from which they may make a verification judgment. Moreover, the process will proceed step-wise. Will NRC verify every step of the process? Some of these processes are: developing a soil cement mix by adding different amounts of cement to a range of three to five different soils; ascertaining how those soils respond to cement stabilization; designing a mix for each soil; varying the cement content and determining how each performs in terms of durability; and establishing the moisture conditions, compaction condition, etc. Tr. (Wissa) at 10864-66. That merely gets the program to bond testing. Id. at 10866.

PFS admits, "[a] fundamental assumption in the PFS approach is that sufficient bonding and shear transfer between clay and soil cement interfaces can be achieved using various construction techniques." SAR, Rev. 22 at 2.6-117 (Applicant Exh JJJ). Then PFS goes on to say it will perform direct shear tests of the interface strength during the design phase of soil cement to demonstrate bond strength. It is apparent from these statement that PFS is relying on the adequacy of the bond strength and soil conditions for the proposed foundation loading. NRC has required, and PFS has performed, direct shear tests of the upper Bonneville clays prior to licensing to demonstrate compliance with 10 CFR §

72.102(d) ("site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading"). It is totally arbitrary to allow PFS to delay a similar demonstration to post license verification for bonding and shear transfer between clay and soil cement interfaces.

Inspectors will need to make complex judgments and decisions about whether PFS has attained less than 75,000 psi Young's modulus. Testing for this is done by trial and error and what is more, the modulus is expected to increase over time. Tr. (Wissa) at 10914; (Mitchell) at 11216-17. PFS says that it will measure Young's modulus after 28 days of curing to be consistent with how the modulus for the storage pad will be measured. PFS Findings ¶ 118. But this does not necessarily reflect reality because the compressive strength and stiffness are likely to increase over a longer period than 28 days. State Findings ¶ 100. How will inspectors verify the margin needed after 28 days to determine whether, long term, the cement-treated soil will have a Young's modulus of less than 75,000 psi? What test will be conducted to verify the result? How will PFS simulate the modulus under dynamic conditions and drop-tipover conditions?

Another complex judgment involves whether the shear strength of the upper Bonneville clays will retain their shear strength if disturbed during construction and recompaction. The Staff relies on construction techniques used in other soil cement applications as a preventative to disturbance. Staff Findings ¶ 4.95 There are no other projects discussed in this proceeding where it was critical that the underlying clays not be disturbed. PFS says that part of its program is to measure the strength and compressibility properties of the remolded and compacted Bonneville deposits Tr. (Trudeau) at 10900.

Aside from the confusion during the testimony as to whether this would be part of the soil cement program (Tr. (Wissa) at 10900-01), again there are substantive tests being conducted that should be part of the licensing basis because they are needed to determine whether soil conditions are adequate for foundation loading. Without the data there is no assurance what the strength will be of those recompacted clays. Tr. (Bartlett) 11165. Furthermore, Dr. Mitchell testified that recompaction is not as easy to achieve in the field as it is under test conditions. Instead of being over a firm subgrade, in the field, compaction will occur over a rather deformable subgrade, *i.e.*, the underlying Bonneville clay that has not been excavated. Consequently, the amount of compaction achievable is reduced compared to compaction over a hard surface. Tr. 11166 (Mitchell). These issues are too complex to be left to verification inspection.

PFS and the Staff ignore the record that new uses of soil cement are subject to significant failure in the early stages. State Findings ¶ 112. PFS's seismic design is driven by the needs of Holtec's cask stability and tip over analyses and those analyses constrain how PFS can attain its foundation design. In the regulatory arena, it is not only imprudent to leave these complex judgments to Staff verification inspection but it is also arbitrary, capricious and obviates a basis finding requisite to licensing the ISFSI. The State and its citizens rely on this Board to ensure proper adherence to administrative procedures and protection of the State's hearing rights by ordering that PFS's soil cement program in its entirety be conducted now and subject to the challenge by the State in an adjudicatory proceeding. Alternatively, the Board should reject PFS's license application.

## IV. Dynamic Analysis

There are three issues that the State will focus on in this portion of its Reply: soil-structure interaction; pad to pad interaction; and soil springs and damping. Also, the State will briefly comment on pad flexibility. As to the other issues raised in PFS's and the Staff's Findings, the State will rest on its Findings or on the general issues discussed in the Overview section, *supra*.

### A. Soil-structure Interaction

Many of the State's allegation can be grouped under seismic soil-structure interaction ("SSI"); therefore, it is useful to first describe the concept and then address PFS's abysmal attempt at SSI analyses. When forces are applied externally to a structural element or develop internally due to inertia loading caused by seismic excitation, both the structural element and the ground must deform and move in a compatible manner because neither the structural displacements nor the ground displacements are independent of each other as a result of their intimate physical contact. Deformation of the supporting soil impacts the structural motion, and in turn, the inertia of the structure influences the soil and the foundation motion; hence, the response of the foundation and the structure is influenced by the soil-structure interaction effects. Depending on the stiffness of the soil and inertia of the structure, the effect of SSI can be very significant on structural responses and in most cases results in increase of the structural motion at the SSI natural frequency of the soil-structure system. Tr. (Ostadan) at 7455-57, 7516-19; 10307, 10312-13; Bartlett/Ostadan Tstmy, Post Tr. 7268 at 9-10. For this reason, seismic SSI analysis is required for all nuclear structures founded on soil foundations. NUREG-0800, Section 3.7.2 (Seismic System Analysis) (Staff

Exh. CC). There are two components of soil-structure interaction: kinematic interaction, caused by differences in stiffness of the foundation and the soils, and inertial interaction, caused by differences in masses in the foundations and their supported structure and in the supporting soils. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 10.

At the PFS site, inertial forces from the mass of the casks will be applied to the pad and the underlying soils; the inertia of the pad itself will also apply force to the soil. For this reason the motion of the foundation will be different from the motion of the supporting soil in the absence of any structure (*i.e.*, free field soil condition). Tr. (Ostadan) at 7516. Pad flexibility or rigidity, pad settlement, pad to pad interaction, and forces caused by inertial load of the casks and the pads are all issues that bear upon the dynamic response of the soil and the structures under seismic excitation. *See* State Findings ¶¶ 179, 190-93, 195, 198-210, 212-18.

One of PFS's major shortcomings in its dynamic analysis is its failure to conduct a comprehensive and accurate soil-structure interaction analysis. In its sliding stability calculations, for example, PFS used free field ground motions (*i.e.*, peak ground acceleration) to compute the seismic loads for the storage pad and the casks. PFS no longer attempts to defend its use of peak ground acceleration but now attempts to mask its mistake by claiming that peak ground acceleration is a good "proxy" for the response acceleration of the pad. PFS Findings ¶ 321. PFS recognizes, however, that the slim margin in its design becomes even slimmer for its base case against sliding because of its ineptitude in performing the sliding analysis. Id. ¶ 324. The Board should give short shrift to PFS's introduction of last minute calculations by Mr. Trudeau in an attempt to show that there is fifty percent radiation

damping and, therefore, PFS's pad sliding analysis is still above a factor of safety of 1.1. This "proxy" argument and any Trudeau calculations in support thereof, are in direct conflict with PFS's claims that SSI effects for the pads are so significant that it is reasonable to consider damping as high as fifty percent. PFS Findings ¶ 322. When SSI effects are significant, the foundation motion can be significantly higher than the free-field motion, as is evident from the CTB analysis and the Luk Report. *See infra.* In addition to PFS's "proxy" argument being in conflict with the data, PFS relies on a witness, Mr. Trudeau, who admittedly has no expertise whatsoever in soil-structure interaction analysis. *See infra.* Further, there are no design calculations to support PFS's last ditch attempt to rescue a major calculation used to support its license application.

PFS cannot rely upon Dr. Wen Tseng, who was given the dynamic forces of the casks calculated by PFS and whose analysis was limited to the structural design of the pad, to claim that PFS conducted a complete SSI analysis; soil-structure interaction was not within the scope of his work for PFS. Tr. (Ostadan) at 10391 ("I think it's fair to point out, in fairness to Dr. Tseng, that he was not given the opportunity, in my view, to perform a full dynamic soil-structure interaction . . . [H]e was given the forces from the pads coming from Holtec analysis . . . By then 95 percent of SSI effect has been decided by Holtec."). Nor can PFS rely on Holtec because Holtec's entire focus was on cask displacement, and Holtec merely perceived its role as a purveyor of information to Stone and Webster and Dr. Tseng and was detached from how that information was used. Tr. (Soler) at 10609. Moreover, as discussed *infra*, Holtec's analysis using soil springs and damping is a gross approximation of soil structure interaction and is an outdated and unreliable method of analysis.

What PFS is left with is Mr. Trudeau. In its Findings, PFS attempts to paint Mr. Trudeau as having expertise in soil-structure interaction analysis. PFS Findings at 9 and ¶¶ 320-25. That certainly is not the case. A brief glance at Mr. Trudeau's resume reinforces this position – there is no mention of SSI experience or publications. Furthermore, all of Mr. Trudeau's work experience, with the exception of the PFS site, has occurred at low seismic sites. Moreover, Mr. Trudeau confirmed that soil-structure interaction is not part of his geotechnical discipline: "I do not perform soil-structure interaction analyses, no." Tr. (Trudeau) at 6163. When asked to define the various components of soil-structure interaction Mr. Trudeau responded: "You're getting into an area where I would rely on our structural dynamics people." Id. at 6162. Furthermore, Mr. Trudeau was unable to define the term "kinematic" – one of the two key components of soil-structure interaction analysis. Id. at 6235 ("I'm assuming you are referring to the dynamic loads that would be the result of the soil-structure interaction analysis."). *Cf* Bartlett/Ostadan Tstmy, Post Tr. 7268 at 10. The record is clear on this point: Mr. Trudeau has no expertise in soil-structure interaction analysis.

One way in which PFS strives to show the acceptability of using the peak ground acceleration value for the horizontal pad response acceleration is, through Mr. Trudeau, to claim "radiation damping applicable to the soil/pad/cask system is so high (50% for the 'best estimate' soil properties case) that the effects of soil-structure interaction in terms of amplifying the accelerations imparted on the pad are limited." PFS Findings ¶ 322. PFS also relies on Mr. Trudeau to contradict the State's soil-structure interaction expert, Dr. Ostadan, who testified that given the difference in response between the free field motions and the

response of the CTB mat from soil-structure interaction, it is reasonable to expect the pad to be similarly affected. PFS Findings ¶ 325; State Findings ¶ 208.

The engineering calculations for the pad sliding analysis that form part of the licensing basis of the ISFSI, PFS Exh. UU, were computed as follows:

> The way the sliding calculation was done by Mr. Trudeau, he looked at the inertial forces due to the casks and got that from ICEC. Then for the inertial forces of the pads, simply took PGA in the horizontal direction times the weight of the pads. The forces coming from the casks, from ICEC calculation, had already included any sliding effects in them because those forces were obtained from Holtec.

Tr. (Bartlett) at 7631. Obviously, the engineering calculations, PFS Exh. UU, do not account for soil-structure interaction. They use free field ground motions. They also rely on Holtec's cask sliding analysis for the forces from the casks – an analysis that is extremely suspect. Certainly PFS's use of peak ground acceleration has been evident since the State filed its second request to modify Utah QQ in August 2001.[19] PFS has chosen to litigate this issue and use a back door approach to shoring up its calculations. Unified Contention L/QQ, D.1.d. There have been no changes to the engineering design calculations, PFS Exh. UU. Instead, there is an unsupported supposition that there is fifty percent radiation damping from the soil-pad-cask system, which is totally inconsistent with PFS's position that peak ground acceleration is a good proxy for the response acceleration of the pad. Trudeau

[19]In the first request to modify Utah QQ, the State alleged that in Revision 8 to the pad stability calculations, PFS failed to use the inertial force of the combined mass of the pad and the underlying cement-treated soil, and that in correcting this mistake in Revision 9, PFS made another fundamental error in using pga for the inertial force of the pad. *See* State of Utah's Second Request to Modify the Bases of Late-filed Contention Utah QQ in Response to More Revised Calculations from the Applicant, dated August 23, 2001, at 4.

Tstmy, Post Tr. 6135 at 14; PFS Findings ¶ 321. The Board should give no weight to Mr. Trudeau's testimony. PFS did not conduct a SASSI or other recognized soil-structure interaction analysis for the soil-pad-cask system, and testimony by a person who has no expertise in soil-structure interaction analysis does not resuscitate PFS's ailing analysis. Besides, evidence of soil-structure interaction is obvious from other analyses at the PFS site.

In its Findings, the State elaborated on the storage pad foundation system and SSI effects. *See* State Findings ¶¶ 198-210. Suffice it say here, where the CTB is concerned, for the same soil site with the same input ground motion, the foundation is responding to seismic excitation at the 1 g level. Tr. (Ostadan) at 7625-26. Mr. Trudeau's observation that the relatively large size of the CTB compared to the storage pads is the reason SSI effects are more pronounced for the CTB is unconvincing. PFS Findings ¶ 325. First, Mr. Trudeau has no expertise in SSI analysis. Second, the foundation pressure of approximately 2ksf is about the same for the CTB mat foundation as it is for the storage pad (*i.e.*, the weight of the structure divided by the foundation area amounts to the same pressure for the pads as it does for the CTB). Thus, the effect of inertial interaction is expected to be similar for both structures. State Findings ¶ 208; Tr. (Ostadan) 7545-52. Also, notwithstanding the qualifications on Dr. Luk's report, for the storage pads there is significant amplification in the horizontal direction that defies the 0.711 g peak horizontal ground acceleration or the 0.79 g revamped number. State Findings ¶ 209. PFS's comment that the Luk report predicts very little cask displacement under the pad accelerations computed in the model does not answer the question of SSI analysis for the pad stability calculation. PFS Findings ¶ 326. Again, PFS focuses on cask displacement. There are innumerable problems with all the

computer models to predict casks displacement – especially when computer modeling is the sole evidence to predict how the casks will perform during an earthquake. The issue here is the stability of the storage pads. Similar to the record for cask displacement, there is a paucity of evidence to support PFS's seismic pad stability analysis.

PFS's last refuge to support its calculation of the dynamic forces for pad stability is in averaging the factor of safety during the duration of the earthquake and to claim that "peak acceleration, whatever its value, will be applied only at a single point in time in the entire time history." PFS Findings ¶ 327. PFS's claim that having a factor of safety of less than one for a short period of time, as compared to the entire duration of the seismic shaking, is equivalent to saying after one initial failure there will be no more failures. Such a concept has never been accepted in earthquake engineering practice and is one that violates the need for PFS to meet a factor of safety of at least 1.1 for the entire duration of the earthquake. At bottom, it shows the depths to which PFS must descend to find support for its analysis.

## B. Pad to Pad Interaction

One of the major calculations in PFS's seismic analysis is the *Stability Analyses of Cask Storage Pads*, Calc. No. G(B) 04, Rev. 9, PFS Exh. UU. The base case in this analysis assumes that all the forces will be transferred downward from the pads to the cement treated soil then to the top of the upper Bonneville clay. The State has repeatedly raised the more realistic load transfer mechanism and load paths from pad to pad interaction, which will cause some of the load to be transferred horizontally rather than under PFS's idealized base case scenario. *See e.g.*, State Findings ¶¶ 121-35; 212-19.

PFS's unprecedented use of soil cement in seismic design, PFS's unrealistic expectation that the soil cement and pads will move in unison during an earthquake,[20] and PFS's unconventional practice of placing heavily loaded shallow foundations that are spaced only five feet apart on soft clays, all contribute to an unanalyzed seismic condition that is critical to foundation stability, and does not consider the true load transfer and the effect of pad to pad interaction.

Pad to pad interaction is of more than academic interest. PFS Findings ¶ 307. PFS's attempt to criticize Dr. Bartlett for not citing a case of foundation-to-foundation interaction without sliding merely reinforces the unprecedented nature of PFS's design. Even PFS recognizes the unprecedented nature of its foundation design, but rejects conventionality because Holtec's cask stability and tipover analyses drive PFS's design philosophy: "Ordinarily a foundation key would be used to ensure that the full strength of the soils beneath a foundation are engaged to resist sliding. However, the hypothetical cask tipover analysis imposes limitations on the thickness and stiffness of the concrete pad that preclude addition of a foundation key to ensure that the full strength of the underlying soils is engaged to resist sliding." *Stability Analyses of Cask Storage Pads*, PFS Exh. UU at 15.

PFS makes an ineffectual attempt to minimize pad to pad interaction by claiming, based on testimony by Mr. Trudeau and the *Stability Analyses of Cask Storage Pads*, PFS Exh. UU, a "large margin" against pad sliding leads to an unrealistic concern that pad to pad

[20]PFS's witness Dr. Weng Tseng recognized that the soil cement is not structurally connected to the pads; therefore, the pads and soil cement cannot be expected to move in lock-step as a single unit. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 16-17.

interaction will materialize. PFS Findings ¶ 309. In Findings ¶ 309, PFS makes no reference to the "beneficial effect of pad sliding" as it does in PFS Findings ¶¶ 209, 213, 215, 217, 249, 265, 311, 315, 317, 328. Furthermore, there are serious and fatal shortcomings in Mr. Trudeau's testimony and in the seismic pad stability design calculations.

PFS relies on the "uniformity" of site soils and soil strain levels in the free field as part of its response to the State's issue of pad to pad interaction resulting from the deformability and potential shear failure of the upper Bonneville clays. PFS Findings ¶¶ 301-04. PFS's arguments have no merit. First, whether upper Bonneville clays are uniform is vigorously contended in Unified Contention L/QQ, Part C (Soils). Second, one cannot use strain levels developed from the free field ground motion to infer what the strain level might be underneath a fully loaded pad. The free-field strain level will be much smaller because free-field analyses do not account for the soil-structure interaction caused by the inertial loadings coming from the heavy masses of the casks and concrete pad and how these masses interact and deform the underlying Bonneville clay during earthquake cycling. State Findings ¶¶ 198-200. PFS has never calculated the strain level resulting from this inertial interaction under the pads. Nor has it developed a soil test program that includes strain-controlled cyclic triaxial testing to define the soil behavior at high strain levels. State Findings ¶¶ 54-61; Unified Contention Utah L/QQ, C.2.b. Instead, PFS relies solely on Mr. Trudeau's opinion that even though PFS developed strains in the free field, those strains will not vary significantly taking into account the pads and the casks. PFS Findings ¶ 302. This is merely a guess based on a false premise, and a guess by a person who has no expertise in soil-structure interaction. Unless and until PFS performs appropriate calculations and

laboratory testing, PFS does not know the strain levels that will develop underneath a fully loaded pad and how those strain levels may impact the stiffness and strength of the Bonneville clay. Third, compared with soil cement, the upper Bonneville clay is deformable because of its relatively low modulus or stiffness. Tr. (Bartlett) at 11308-09. Although PFS has not completed a soil cement test program to verify the Young's modulus of the soil cement, it is possible that the soil cement will have a Young's modulus that is approximately one order of magnitude higher than the cement-treated soil and two orders of magnitude higher than the Bonneville clay. Id. at 11336. Because of these high stiffness contrasts, kinematic interaction will be introduced and each material will have a significantly different strain behavior during the repeated cyclic loading caused by the seismic event. Id. at 11338-40. The Bonneville clay will have to strain much more to develop its peak capacity to carry load, whereas the much stiffer soil cement will gain its peak capacity at much smaller strain. Id. This is known as strain incompatibility. Tr. (Bartlett) at 11340-41. These severe differences in stiffness will cause the soil cement to pick up most of the inertial load during the initial stress cycle and because it initially carries much of the load, the soil cement will transfer some of this load laterally to the adjacent pads via the 5-foot soil cement plug placed between the pads. Id. at 11308-10. This load transfer mechanism is unavoidable. It will happen even if the clay does not reach the failure state and pad sliding is initiated. It occurs because the high contrast in stiffness and strain incompatibility between these very different materials. Id. at 11308-10. This point can be seen in the 1,900 kips horizontal load transfer in Dr. Soler's simple pad to pad calculation, PFS Exh. 225 at 28.

PFS announces that it does not take credit for passive resistance of the soil cement

"frame" surrounding the pads in its base case. PFS Findings ¶¶ 65, 438. This is not the significant point. The soil cement will surround the pad whether PFS takes credit for it or not. The soil cement is a stiffer material than the cement-treated soil and it will impart horizontal forces during seismic excitation. *See* State Findings ¶¶ 212-18; Bartlett/Ostadan Tstmy, Post Tr. 7268 at 15-17. Even PFS's witness Dr. Soler showed as much in his limited evaluation of pad to pad interaction. PFS Exh. 225 at 28. The significant point is PFS's erroneous expectation that one hundred percent of the forces will be transmitted straight down and that there will be no lateral forces whatsoever. Tr. (Bartlett) at 11205-09. That is not a realistic base case and it certainly is not conservative. Id.

In its Findings, PFS attempts to cloud the record by suggesting that the State's witnesses changed their testimony. PFS Findings ¶¶ 298-99. The State's witnesses, Dr. Bartlett and Dr. Ostadan, are experts in seismic foundation analysis. Their focus is on whether PFS has correctly analyzed the seismic stability of the storage pads. Their long standing concern has been, and continues to be, the load path from the casks-pad-soil cement system and where and how those loads are transferred to the soft soil under the pads. Bartlett/Ostadan Tstmy, Post Tr. 7268 at at 17. PFS mistakenly alludes to the "apparent shift in emphasis" of the State's claim. PFS Findings ¶ 300. The Board should not be distracted by PFS's Findings. The nub of the State's concern is that PFS is employing an unprecedented seismic design by spacing the storage pads very close together, placing them on soft compressible soils surrounded by a stiffer soil cement mix, and it has failed to analyze the load path and load transfer from this unprecedented design. See e.g., Bartlett/Ostadan Tstmy, Post Tr. 7268 at 15-18; Findings ¶ 215-15; Tr. (Ostadan) at 7549.

PFS attempts to portray the State's claims with respect to pad-to-pad interaction as an "apparent shift in emphasis" during the hearing. PFS Findings ¶ 300. The State's response is twofold. First, PFS did not understand (or did not want to understand) the concepts the State raised. Second, the testimony quoted in PFS Findings ¶ 299 was rebuttal testimony at the State Capitol on Saturday, June 8, 2002 in response to a more or less overnight analysis conducted by Dr. Soler on the effect of one fully loaded pad and a neighboring pad loaded with one cask. Tr. (Soler) at 10557-91. Each of these issues will be addressed in turn.

PFS misperceived the State's issues; consequently, PFS's rebuttal evaluation by Dr. Soler did not squarely address the worst case for pad-to-pad interaction. Tr. (Ostadan) at 10687. Part of Dr. Soler's rebuttal related to his evaluation of the stresses that develop in the concrete pad due to pad-to-pad interaction. The structural design of the pad has never been raised by the State with respect to pad-to-pad interaction. Tr. (Ostadan) 10686-88. Moreover, Dr. Soler's focus was, at it has always been, on cask displacement, whereas the focus of the State's foundation experts was the worst case analysis as it relates to the stability of the foundation, and that was not done in Dr. Soler's analysis. The Staff also misperceived the State's concern, which for pad-to-pad interaction, has nothing to do with the different arrival times or frequency of seismic waves. Tr. (Ostadan) at 10711-13.

As part of PFS's rebuttal testimony on Saturday, June 8, 2002, PFS's witness, Dr. Alan Soler, presented a recent computer run to evaluate concerns he perceived from Dr. Ostadan and Dr. Bartlett's testimony on pad to pad interaction. Soler Rebuttal, Post Tr. 10557; PFS Exh. 225 and 225C. For his analysis, Dr. Soler assumes a two pad system with

eight casks on one pad and one on the other. *See* PFS Findings ¶ 305. The results of his analysis contradict PFS's base case assumption that one hundred percent of the forces will be transferred vertically down to the upper Bonneville clays. Dr. Soler's limited analysis of a two pad system shows about thirty percent (1,900 kips) of the force is transferred laterally. Tr. (Ostadan) at 10680-82.

PFS's claims that pad-to-pad interaction forces have no practical significance because, in its view, those forces have essentially no impact on the stability of the pads or the casks. PFS Findings ¶ 307. PFS takes a giant leap in logic by reaching such a conclusion. Dr. Soler's analysis shows a significant change in the load transfer mechanism to the soil from PFS's base case used to support its license application. How the load is transferred to the soil is a critical and missing component in PFS's seismic pad stability design calculation. Does the load all go down directly under the pad? Is some transmitted laterally? Is there a progressive compounding compressive effect from one pad to the other depending on how the pads are loaded? What will be the effect from a column of ten pads? And how will that effect differ under the vast number of loading combinations of one to eight casks on a 10 pad system? We do not know.

Dr. Soler's rebuttal calculation is not a design document. It considered a very simple two pad system with a very small gap (6/10") between the edge of pads and soil cement, and in two instances did not allow for pad sliding. PFS Exh. 225. PFS has given no clear explanation of the amount of horizontal load transfer versus vertical downward load transfer or the effect of pad-to-pad interaction by all adjacent pads to support PFS's assertion that pad-to-pad interaction has no practical significance to pad or cask stability. Again, the State

comes back to the unprecedented nature of PFS's seismic design and PFS's uncompromising reliance on idealized assumptions and lack of engineering design calculations to support its pronouncements on the "conservatism" of its design, and the situational need to sometimes claim pad sliding as a beneficial effect but to deny this effect when it come to pad to pad interaction.

### C. Soil Spring and Damping

One of the State's concerns with the entire PFS project relying on Holtec's non-linear computer analyses is the accuracy of Holtec's soil spring and damping. Tr. (Ostadan) at 7350. Holtec used a soil spring and damping method to consider the dynamic soil structure interaction effects of the cask-pad-soil system. Dr. Ostadan testified:

> Soil springs and damping are typically considered for dynamic analysis of structures such as the pad and the casks to represent the effect of the supporting soil layers as well as the foundation size in the response. These properties are frequency dependent. If the pads are assumed to be rigid, the damping will be larger. If the pads are indeed flexible the damping will be less. The less the damping, the higher the motion of the pads and the seismic loads on the pads. It is important to use the soil spring and damping values at appropriate frequencies corresponding to the foundation frequencies and check the pad rigidity assumption based on the final design.

Bartlett/Ostadan Tstmy, Post Tr. 7268 at 14-15.

Holtec based its soil mass, spring and damping calculations on Newmark and Rosenblueth's Fundamentals of Earthquake Engineering. PFS Findings ¶ 276. This methodology was developed in the 1970s. Tr. (Tseng) at 5638. The results of Holtec's soil spring and damping method are used for both the design and stability evaluation of the storage pads as well to predict the seismic behavior of the casks. The Newmark and Rosenblueth method used by Holtec is dated and neglects many fundamental aspects of the

dynamic behavior of the cask-pad-soil system.

First, the soil spring and damping are extremely frequency dependent because of the layered soil profile at the PFS site. The soil spring and damping dramatically change with frequency because of the reflection of the seismic waves with the soil layers. Tr. (Ostadan) at 7575. This effect is evident in the soil spring and damping calculations performed for the CTB (SG-4, Rev. 2, *Development of Soil Impedance Functions for Canister Transfer Building*, March 21, 2001, SWEC) in which PFS generated the soil spring and damping values. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 14-15. In the CTB calculation, SG-4, the soil spring and damping are plotted as a function of frequency, showing that these parameters are highly dependent on the frequency due to soil layering at the site. Id. Significantly, PFS could have, but choose not to, use the same soil structure interaction method it used to analyze the CTB (in calculation SG-4) for its analyses of the storage pad and cask stability. Instead, Holtec ignored the frequency dependency of the soil spring and damping. There is no evidence that Holtec used the appropriate values for the foundation system or that Holtec did not substantially underestimate the cask behavior.

Second, Holtec assumed the pad was rigid in its soil spring and damping calculations. If the pad is in fact rigid, its vibration at all points of the pad is in-phase and moving together, then energy is dissipated as the pad impacts the soil. Tr. (Ostadan) at 7456, State Exh. 112 at 112. However, if the pad is flexible it does not have as much damping because the pad movement is out-of-phase or in other words the pad moves differently at different locations. Tr. (Ostadan) at 7456, 7458; State Exh. 112 at 112-13. Thus, the assumption that the pad is rigid, results in the overestimation of damping which then underestimates the

seismic responses. The Applicant has not demonstrated that the pad is, in fact, rigid or that it did not underestimated the seismic responses, including cask behavior.

Finally, Holtec failed to account for the effect of numerous rows of pads on the soil spring and damping. This failure also results in the overestimation of damping which in turn underestimates the seismic responses. Bartlett/Ostadan Tstmy, Post Tr. 7268 at 14-15. Consequently, because of the unconservative and over-simplified assumptions used in its soil spring and damping calculations, Holtec underestimated the actual effects of the seismic soil structure interaction and underestimated the seismic load.

While industry standard linear soil structure interaction codes cannot be used for a non-linear analysis, inasmuch as soil structure interaction effects at the PFS site are significant and the Applicant is relying solely on Holtec's non-linear analyses, it is reasonable that Holtec's soil spring and damping values are confirmed with an alternate method. However, for a non-sliding situation, Holtec did not verify the soil spring and damping values generated from its model with a soil structure interaction code such as SASSI. Tr. (Soler) at 5999-6000. Dr. Soler opined that even for non-sliding situations, he could not make a comparison between results from his model and a soil structure interaction code because the cask and pad interaction is non-linear. Id. at 5999-6001. Dr. Soler is mistaken. Soil spring and damping are only a function of the soil and pad properties and are independent of the interaction between the casks and the pad. Thus, whether the interaction between the cask and the pad is non-linear is not a limiting factor to confirm the soil spring and damping values. In fact, PFS witness, Dr. Wen Tseng confirmed that it is "generally true" that Holtec's soil spring and damping values could be verified using a soil structure

interactions code such as SASSI. Tr. (Tseng) at 5643.

Furthermore, Dr. Ostadan opined that the soil spring and damping values could be selected according to the same method PFS used for the CTB (PFS calculation SC-4) by selecting constant values for soil spring and damping that corresponds to the pad frequency. Tr. (Ostadan) at 7576. Constant values are selected because of the non-linear analysis. Id. Holtec should have, at least, determined the frequency of the pad response from their force time histories and performed a simple calculation to ensure their original spring and damping values correctly correspond to the frequency of the pad response. Id. at 75823-83. If the spring and damping values did not correspond, then Holtec should have selected another frequency to calculate its spring and damping values and repeat the modeling, until it converges.[21] Id.

The Applicant attempts to challenge Dr. Ostadan's opinion by claiming that his iterative "method requires knowing the system frequency beforehand" and the predominant frequency changes so the solution may not converge. PFS Findings ¶ 274. Contrary to the Applicant's finding, it is not necessary to know the system frequency to initiate this method. The purpose for Dr. Ostadan's iterative method is based solely on an unknown system frequency.

The Applicant's witness, Dr. Tseng testified that to calculate soil spring and damping values for a non-linear time history response analysis "necessitates a step where you approximate the frequency-dependent foundation by constant parameters which involve the

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[21]PFS refers to Dr. Ostadan's method as an "iterative method." *See* PFS Findings ¶ 274.

constant spring value, constant dashpot value, and that coupled with a constant virtual mass will give you an approximate frequency variation up to the first mode of the soil response." Tr. (Tseng) at 5642. Dr. Ostadan does not dispute starting with this method. However, because of the significant effects of soil structure interaction on the non-linear analysis, Dr. Ostadan maintains that there must be some confirmation that the soil spring and damping values correspond to the appropriate frequency and do not underestimate the cask and pad behavior.

Moreover, while Dr. Tseng testified that Dr. Ostadan's iterative method may not converge, he also testified that "in many cases it may work." Tr. (Tseng) at 10736. While this process could result in multiple iterations causing additional effort, given that PFS's design adequacy relies entirely on Holtec's unproven non-linear computer analysis, it is reasonable to require some assurance that Holtec has not severely underestimated cask behavior. Given the "wide variation of parameters," Dr. Tseng essentially agreed that the "important issue will be . . . whether the cask's motion. . . will have enough margin to accommodate these motions and the casks will be stable." Tr. (Tseng) at 10780.

Furthermore, other than relying solely on his opinion, Dr. Soler did not provide any evidence that the soilspring and damping values he calculated for the PFS seismic analyses adequately account for soil structure interaction effects.[22] Id. at 6000-6001. An assertion of "engineering judgment," without any explanation or reasons for the judgement, is insufficient to support the conclusions of the expert engineering witness. Texas Utilities

[22]Dr. Tseng admitted he did not check the Holtec soil spring and damping calculations. Tr. (Tseng) at 10770.

Generating Co. (Comanche Peak Steam Electric Station, Units 1 and 2), LBP-84-10, 19 NRC 509, 518, 532 (1984). In fact, Dr. Soler admitted that neither he nor Mr. Bullard – the author of Multi Cask Response at PFS ISFSI for 2,000 years, Rev. 0 – had expertise in analyzing soil dynamics and foundation design (calculating the soil spring and damping). Tr. (Soler) at 5992, 5996-57. Besides the analysis for Tennessee Valley Authority, the only soil dynamic work that Dr. Soler has performed is for this instant case. Id. at 5995.

Dr. Luk's testimony is not supportive of Holtec's approach. Dr. Luk testified that he "did not know how the springs in one series can represent the dynamic characteristics of site-specific soil profile data that is tabulated in many different horizontal layers." Tr. (Luk) at 6980. Dr. Luk concluded that using soil springs in modeling "may not or it might not be adequate." Id. Particularly in light of his admitted limited experience in soil dynamics, there is no evidence that support's Dr. Soler's self-proclaimed opinion that his calculations for the soil spring and damping did adequately account for soil structure interaction effects in the Holtec cask stability analyses and that it did not substantially underestimate the behavior of the cask and pad.

The Applicant profess that Holtec simulations using a soil damping value of one percent "indicate 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 [cask] design." PFS Findings ¶ 277. As discussed in Section V, Seismic Nonlinear Analyses of Free Standing Cask Behavior, *infra*, Holtec's simulations of the seismic behavior of casks are not reliable. Most notably, Holtec's one percent soil damping simulations were run at forty percent impact damping which essentially minimized any vertical acceleration to the casks.

Id. Additionally, the high contact stiffness values used in the one percent soil damping simulations may also underestimate cask behavior. Id. Thus, Holtec's simulations using one percent soil damping are insufficient to demonstrate the accuracy or conservatism of Holtec's soil spring and damping values.

Finally, the Applicant also proclaims that Dr. Vincent Luk's seismic analysis of the behavior of casks demonstrated large cask design margins even if Holtec insufficiently accounted for the frequency dependence of the soil spring and damping. PFS Findings ¶ 279. Dr. Luk's analysis did not accurately model the soil conditions at the PFS site. *See* State Findings ¶¶ 406-440. Moreover, Dr. Luk's analysis is not reliable or relevant in this case. *See* Id. ¶¶ 377-440, 444, *see also* Section V, Seismic Nonlinear Analyses of Free Standing Cask Behavior, *infra*.. Thus, Dr. Luk's analysis is not sufficient to eliminate a showing that Holtec's soil spring and damping values do not underestimate cask behavior.

**D. Pad Flexibility**

The State addressed both pad flexibility and pad rigidity in its Findings ¶¶ 188-211 and does not intend to revisit those issues. The State here responds to PFS Findings ¶ 270, in which it refers to "parametric studies" of casks stability and pad flexibility/rigidity conducted by Holtec for another facility, Tennessee Valley Authority's Sequoyah Nuclear Power Plant ("Sequoyah") to draw analogies to the PFS site. *See also* Singh/Soler Tstmy, Post Tr. 5750 at 38-39.

In its model for the PFS site, Holtec assumed the pad was rigid. Singh/Soler Tstmy, Post Tr. 5750 at 38. This assumption is challenged by the State and, moreover, by assuming that the pad is rigid, Holtec has significantly underestimated the dynamic loading atop the

pads and overestimated the foundation damping. *See* Unified Contention L/QQ, Section D.1.b (PFS Exh. 237). Putting aside whether two computer runs constitute a "parametric study," Singh/Soler Tstmy, Post Tr. 5750 at 39, PFS relies on Holtec's Sequoyah pad analysis to make the global argument that whether a pad is assumed to be flexible or rigid, results in negligible dynamic behavior of the casks in a seismic event. PFS Findings ¶ 270.

Dr. Soler had difficulty recollecting the zero period acceleration at the top of the pad at Sequoyah but thought it could have been "in the order of 0.45 to 0.55" g. Tr. (Soler) at 6007. Dr. Soler could not recall the soil characterization at Sequoyah and did not oversee the "quantification of the soil properties." Id. at 6007-08, 6015. Dr. Soler did not know the long term pad settlement estimated for Sequoyah. Id. at 6009. Moreover, the pads have not been finally designed but it is highly unlikely that pads will be supported by cement-treated soil. Id. at 6009. Furthermore, Dr. Soler did not estimate the maximum deflection of the storage pad at Sequoyah. Id. at 6016.

The record is severely lacking to demonstrate that site specific conditions at Sequoyah have any similarity to those at the PFS site or that the conclusions from the effects of pad flexibility at Sequoyah are applicable in this case. In sum, any analysis from Sequoyah has no relevance to pad flexibility/rigidity or cask stability at the PFS site.

## V. Seismic Nonlinear Analyses of Free Standing Cask Behavior

In its proposed findings, the Applicant grasps at a myriad of ways to denigrate the opinions of the State's witnesses. The Applicant's proposed findings with respect to cask stability missed the mark in many respects. Importantly, the evidence does not support the Applicant's ultimate burden to demonstrate that the HI-STORM 100 cask would not tip

over during a 2,000-year earthquake at the proposed PFS site. Staff Finding ¶ 5.110. The linchpin of the Applicant's evidence is Holtec's nonlinear computer analysis of the seismic behavior of unanchored HI-STORM 100 casks subject to approximately 0.7 g ground motion from a 2,000-year earthquake. Although the Applicant devotes 50 pages to justify its seismic design of the casks, the Applicant's Findings fail to counteract four critical facts raised by the State. First, the Applicant is basing its entire case solely on sensitive nonlinear computer models (DYNAMO and VisualNastran) to predict the seismic behavior of unanchored casks. Second, there is no precedent or performance data for storing or predicting the behavior of unanchored cylindrical storage casks at locations with ground motions of 0.7 g or greater, like at the PFS site.[23] Next, Holtec has not validated its nonlinear computer model or its PFS 2,000-year analysis with test data. Finally, as a result, there is no evidence that Holtec's nonlinear models accurately predict the seismic behavior of unanchored HI-STORM 100 casks at ground motions of 0.7 g or greater. Although supportive of PFS, the Staff Findings also fail to show the Applicant has met its burden that the HI-STORM 100 casks will not tip over.

The key issue before the Board is whether the casks will tip over as a result of a seismic event at the PFS site. In fact, the Staff states "[t]he acceptance criterion was that the casks must be stable, in the sense that the center of the top cover of the cask must remain

[23]Clearly, if the Applicant's request to lower the seismic design basis earthquake is rejected, the Applicant must then show that the unanchored casks will not tip over at higher ground motions. Notably, the ground motions for a 10,000-year earthquake at the PFS site is 1.33 g (vertical peak ground acceleration) and approximately 1.25 g (horizontal peak ground accelerations). PFS Findings ¶ 147.

within the original contact circle that the cask makes with the pad." Staff Findings ¶ 5.110. Nonetheless, in an attempt to diminish its burden, the Applicant posits that "[u]litimately, the integrity of the multi-purpose canister ("MPC") contained within the casks is the most crucial concern." PFS Findings ¶ 125. In claiming that the MPC will not be breached even if the casks tips over, the Applicant points to its severely limited testimony regarding Holtec's non-mechanistic drop/tip over analyses.[24] Id. ¶ 136. Nevertheless, the Applicant's and Staff's Findings are flawed in their claims that it is inconsequential if the cask tips over because Holtec's tip over analyses is itself flawed in its assumption that the cask does not in fact tip over. Significantly, Holtec inappropriately used an initial angular velocity of zero based on its assumption a cask would not, in fact, tip over as a result of seismic ground motions. *See* State Findings ¶¶ 365, 515. If Holtec's nonlinear computer estimate is wrong and the casks do, in fact, tip over, then Holtec's conclusions from its tip over analyses are also wrong because if a cask tips over, the initial angular velocity will clearly not be zero. *See* Reply, discussion in Section VI Seismic Exemption Request, *infra*. As a result, Holtec's conclusion that the MPC would not be breached in the event of tip over is suspect, given that there is no evidence that Holtec's nonlinear models and analyses accurately predict cask tip over or that the initial angular velocity due to seismically induced tip over will, in fact, be zero.

While the Applicant has the burden of proof, in this case where the Staff adopts the

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[24]In close harmony with the Applicant, the Staff also maintains Holtec's tip over analysis "provides assurance that a cask tip over will not result in a breach of the confinement barrier." Staff Findings ¶ 5.56, *see also* ¶ 5.111.

Applicant's position, the Staff shares the burden of proof. *See* Philadelphia Electric Company (Peach Bottom Atomic Power Station, Units 2 & 3), ALAB-566, 10 NRC 527, 529, n.3 (1979). One would expect a regulatory agency not to advocate for a regulated entity, as the Staff apparently feels compelled to do in its defense of PFS. Carolina Power and Light Company (Shearon Harris Nuclear Power Plant, Units 1, 2, 3, and 4), LBP-79-19, 10 N.R.C. 37, 107 (1979) (Staff should confine its role to determining whether the application meets the Staff's requirements). In fact, Staff Findings are essentially in lock step with the Applicant's Findings. The Staff, like PFS, has failed to meet its burden in demonstrating a HI-STORM 100 cask would not tip over during a 2,000-year earthquake at PFS.

The Staff Findings are gross generalizations and summaries of various testimony.[25] The Board cannot rely on Staff Findings in that the Board in its findings of fact and conclusions of law has a "duty not only to resolve contested issues but 'to articulate in reasonable detail the basis' for the course of action chosen.'" *See* Public Service Company of New Hampshire, et al. (Seabrook Station, Units 1 and 2), ALAB-422, 6 NRC 33, 41 (1977) (*quoting* Northern States Power Co. (Prairie Island Nuclear Generating Plant, Units 1 and 2), ALAB-104, 6 AEC 179 (1973). As such, the Board must explain its reasons for rejecting contrary opinions from expert witnesses.[26] Id. at 41. In the summary of its findings, the

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[25]The State acknowledges the Staff supplied intense detail when describing the computer simulations in which it commissioned. *See* Staff Findings ¶¶ 5.153-5.187.

[26]In Seabrook, the Appeals Board noted that the Licensing Board decision fell far short of "applicable standards," in part, due to its "lack of reference (much less discussion of) evidence contrary" to what the Board accepted. Seabrook at 41. The Seabrook Appeals Board further reasoned that the Board had an obligation to explain why it did not accept the contrary evidence given "the contrary evidence was reasonable on its face and sponsored by

Staff state "[w]e have carefully considered all of the evidence presented by the parties . . ." and later, "[w]e have weighed specific criticisms . . . proffered by State witnesses [Drs.] Ostadan, Khan and Bartlett, and find them wanting, for the reasons discussed above." Staff Findings ¶¶ 5.180, 5.184. In its findings, the Staff basically ignore the State's testimony and there is no discussion of the basis for the Staff doing so. The Board, as the Staff would have it find, cannot merely proclaim it considered all the evidence – it must actually do more. The Board must explain the basis for its findings. Consequently, the Staff Findings are of little benefit to the Board because they essentially dismisses the State's expert testimony, primarily in footnotes, no less, based on sweeping statements of the Staff and PFS witnesses and fail to "articulate in reasonable detail the basis" for its proposed findings. In general, the State will not address most of the Staff Findings with respect to cask stability in that it is impossible to even surmise the basis for the Staff's proposed findings.

For example, the Staff makes a feeble attempt to attack the results of Dr. Khan's parametric study[27] in a couple of footnotes and a single paragraph. *See* Staff Findings ¶¶ 5.24, n.52, 5.27, n.53, 5.62. The Staff claim Dr. Khan's results are "grossly unreliable" and summarily dismiss all results from Dr. Khan's parametric study in a footnote (n.52). Dr. Khan essentially performed twenty runs (eleven runs without rocking effects and an additional nine 3-dimensional runs with rocking effects). State Exh. 122 at 11, 13.

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well qualified witnesses." Id.

[27]On behalf of the State, Dr. Khan conducted a parametric study to evaluate the seismic behavior of the HI-STORM 100 cask at the PFS site. Khan/Ostadan Tstmy, Post Tr. 7123 at 7; State Findings ¶¶ 295-297.

Nowhere does the Staff address Dr. Khan's specific results or explain why the Staff concludes the results are unreliable. Similarly, the Staff states "Drs. Singh and Soler responded to claims raised by State witness Dr. Khan" and then summarily described twenty-three page of his testimony in two sentences with no discussion of the merit, or lack of merit, of the testimony. Staff Findings ¶ 5.62.

Rather than discuss the basis for its proposed finding, the Staff merely quote conclusory characterizations from PFS witnesses, and at times a Staff witness, and make citations to over twenty-seven pages of testimony without identifying a single basis, including "[*s*]*ee generally*" citations. Staff Findings ¶¶ 5.24, n.52, 5.27, n.53. The Staff makes no attempt to identify any technical basis to support the gross characterizations. In fact, although the Staff quote its witness, Mr. Pomerening, the record does not support any basis for his bald opinion in challenging Dr. Khan's input parameters. There is also no evidence that Mr. Pomerening is qualified to render an expert opinion with respect to the input parameters for a nonlinear computer analysis.

Remarkably, the Staff clearly mischaracterizes Dr. Ostadan's testimony stating he "distanced himself from Dr. Khan's analysis." Id. In fact, the record contravenes the Staff's characterization.[28] Importantly, Dr. Ostadan testified that Dr. Khan's parametric study

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[28]Although Dr. Ostadan opined that he did not believe Dr. Khan's run showing vertical uplift of 2 feet and horizontal movement of 20 to 30 feet was realistic, however, he also stated his opinion the large displacements "could happen." Tr. (Ostadan) at 7391-92. Consistent with Dr. Khan's testimony, Dr. Ostadan testified that Dr. Khan's analysis was not for design purposes but to determine the sensitivity of the cask behavior to input parameters. Id., Khan/Ostadan Tstmy, Post Tr. 7123 at 7, 14. Furthermore, Dr. Ostadan opined that a computer program could make large displacement predictions (*e.g.*, 2 feet uplift and 20 to 30 feet horizontal) or a couple of inches, as in the Holtec simulations. Tr.

supports his (Dr. Ostadan's) opinion that "PFS has not evaluated the cumulative effect of the applicable range of input parameters in evaluating the stability of the casks," and therefore, "the stability of the free standing casks on the pad is not proven." Khan/Ostadan Tstmy, Post Tr. 7123 at 14-15. This example demonstrates it is impossible for the State to speculate on the basis for the Staff's proposed findings. Given the Staff's posture essentially coalesces with the Applicant's position, the State will generally not respond directly to the Staff's Findings.

### A. Dr. Khan's Seismic Modeling Opinions Are Supported By His Professional Experience, Knowledge, and Education.

In an apparent effort to divert the Board's attention from critical issues raised by State expert, Dr. Mohsin Khan, the Applicant, yet again, attempts to discredit Dr. Khan's relevant and extensive qualifications. The Applicant's contempt for Dr. Khan's qualifications in this matter is without merit.

Early in the proceedings, in proclaiming their strong belief in the transferability of professional knowledge, the Board rejected an attack mounted by PFS on Dr. Khan's qualifications. Tr. (Judge Farrar) at 5267. As there are not standards for what weight to give to an expert witness's testimony and in light of PFS's renewed attack, the standard for expert witness qualification is a good guidance for determining the weight given expert testimony. In determining an expert's qualifications, the " test of a witness's qualification is whether his

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(Ostadan) at 7391-92. Because a computer model can predict large or small displacement based on the input parameters, Dr. Ostadan concludes, in total agreement with Dr. Khan's parametric study, it is important to have credible input parameters to insure the results are "varied and reasonably conservative." Id. at 7392 (*emphasis* added).

knowledge of the matter in relation to which his opinion is sought is such that it probably will aid the trier of the question to determine the truth." Illinois Power Co. (Clinton Power Station, Units 1 and 2), LBP-75-59, 2 NRC 579, 588 (1975), aff. ALAB-340, 4 NRC 27 (1976). In fact, in its rejection of PFS's motion in limine, the Board also informed the parties that with respect to expert testimony, the Board would consider "what does the witness say, how strong are the witness's reasons for saying that, what contradictory things may the witness have said before." Tr. (Judge Farrar) at 5268.

Dr. Khan's career has focused on various aspects of seismic analysis and design in high seismic areas. His experience includes the finite element analyses of free standing objects in high seismic areas and seismic qualification of structures, systems, and components at nuclear facilities.[29]

The Applicant's primary complaint repeated throughout its Findings is that Dr. Khan has no experience in analyzing "large" free standing objects, such as casks. *See e.g.*, PFS Findings ¶¶ 163, 182, 194, 222, 223, 225, 226. The Applicant is clearly wrong. Dr. Khan has analyzed the seismic response of free standing spent nuclear fuel racks and rigid blocks in high seismic areas. Moreover, Dr. Khan reviewed seismic scoping analyses prepared for a variety of spent nuclear fuel cask performed by various vendors. Khan/Ostadan Tstmy, Post Tr. 7123 at 3-4.

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[29] As described in Dr. Khan's testimony, he has extensive experience in performing seismic analyses of structures and equipment; he has over 22 years' experience using response spectral data and finite element analyses to predict seismic performance of various structures, systems, and components, including those at nuclear plants, and to predict seismic response. Khan/Ostadan Tstmy, Post Tr. 7123 at 3-4.

Furthermore, attempting to draw a fine line between prior unanchored cask modeling and relevant professional modeling experience and education would discard crucial and probative evidence. Significantly, as Dr. Khan explained, a "change in boundary conditions at the base" is the only difference between modeling free-standing and anchored structures. Tr. (Khan) at 7142. Moreover, the difference in boundary conditions is simply that unanchored structures are allowed unrestricted movement, whereas for anchored structures, the degrees of freedom, the displacement, and rotation at the base of the structure is set at zero. Id.

Testimony by Dr. Khan, a well-qualified expert by virtue of his education and experience, will aid the Board in determining the truth (*i.e.*, accuracy and validity) of the Holtec cask stability analysis proffered by PFS in support of its design. The purpose of Dr. Khan's analysis was to evaluate Holtec's seismic cask stability results by independently modeling portions of a HI-STORM 100 cask sliding and tipping under seismic motion from a 2,000-year earthquake. Khan/Ostadan Tstmy, Post Tr. 7123 at 6. Another purpose the State had in having Dr. Khan conduct his analysis was to evaluate the results of a cask stability analysis when the input parameters are changed within acceptable bounds. Ostadan/Bartlett Tstmy, Post Tr. 7123 at 11.

Of course, Dr. Khan does not have direct experience in conducting analyses of the stability of such an unprecedented, unproven and untested design concept as the one PFS has proposed. Two of the State's experts maintain that PFS has presented a one-of-a kind design and they know of no similar seismic design that uses untested concepts that are inherent in PFS's design. Id. at 5. According to Drs. Ostadan and Bartlett, a unique feature

of Holtec's design concept is controlled sliding. They add:

> Holtec puts forward the proposition that during strong ground motions, the casks will be allowed to slide and such sliding will occur in a uniform and controlled manner without collision or tipping. Such a concept defies observations from major earthquakes and engineering logic. It is unprecedented to design unanchored dry storage casks for a seismically active area with such intense strong ground motions similar to those at the PFS facility. The unconservatism in the design is further compounded when PFS uses its claim of "controlled" cask sliding as a mechanism to reduce the seismic loading to the pad foundations.

Id. PFS attempts to make the point that sliding is not "controlled" but a consequence of the earthquake. PFS Findings ¶ 56; *see also* ¶ 248. However its characterization does not detract from the point that PFS is using untested seismic design concepts. Moreover, no witness in these proceedings identified any site or proposed site that would utilize unanchored, cylindrical dry storage casks at ground motions of 0.7 g or greater. *See e.g.*, State Findings ¶¶ 250-251. In fact, PFS's attacks on Dr. Khan's experience underscores the State's overriding concern with PFS's design: the design is unproven and unprecedented. Therefore, it is illogical to expect any expert in the field, including the cask vendors, to have direct experience in analyzing a design that allows free standing casks and pads to rotate and slide – a design that has never been used at any site with ground motions equivalent to or greater than those for a 0.7 g, 2,000-year return period earthquake, at the PFS site.

The Applicant itself suggests no standard for judging the experience of an expert witness but implies that Dr. Khan's relevant experience pales in comparison to the experience of Drs. Singh and Soler. It is hard to imagine, the locating of any large unanchored object (*e.g.*, tanks or even casks) in seismic areas like at the proposed PFS site. Moreover, the record shows that NRC had issued only eleven site specific licenses for dry

storage casks.[30] Tr. (Guttman) at 7045. Thus, the domain of possible seismic analyses of dry storage casks is severely limited to the few sites with dry storage casks. Moreover, there is no evidence that unanchored, cylindrical casks have been licensed at sites with ground motions of 0.7 g or greater. If the Board were to accept the Applicant's short-sighted standard requiring experience in the modeling of unanchored casks, it would likely limit the expertise in this matter to essentially cask vendors who are primarily responsible for the seismic analyses of free standing casks in order to promote and sell their wares.[31] Clearly the Board cannot adopt a different standard for different witnesses. Thus, Dr. Khan's experience should be reviewed in light of the relevant experience of the witnesses proffered by other parties.

Even the cask vendors have limited experience modeling free standings casks in seismic areas due to the few sites requiring site specific seismic analyses. The State does not contest that to sell and support his commodity, as co-owner, Dr. Soler conducted seismic analyses of casks at five other sites,[32] apparently running the same model as he used at the

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[30] Site specific seismic analyses must only be conducted if the seismic standard specified in the cask specific, generic certificate of compliance is exceeded, as in this case. Tr. (Singh) at 5877, 5884-6. Additionally, Mr. Guttman had no knowledge whether the licensed sites utilized unanchored, cylindrical casks or the range of ground motions at the various sites. Tr. (Guttman) at 7069-71.

[31]Notably, unlike Holtec co-owners, Drs. Singh and Soler, who admittedly could realize hundreds of millions of dollars in cask related sales (Tr. (Singh) at 5915-20), Dr. Khan has no interest in the outcome of this proceeding.

[32]Those five sites are described in State Findings ¶ 262.

PFS site.[33] *See* Singh/Soler Tstmy, Post Tr. 5750 at 14, 17. The fact that Dr. Soler may have run the same model at other sites without any validation against performance or test data does not in and of itself attest to the accuracy of his analysis. Only one site, Dresden, in which Holtec conducted a seismic analysis was actually storing casks at the time of the hearings.[34] Tr. (Singh) at 5918. The ground motion at Dresden is approximately 0.2 g zero period acceleration. State Exh. 121 at 38. Significantly, other than this case, the cask vendors themselves – Drs. Singh and Soler -- do not have any previous specific experience modeling the seismic behavior of free standing, cylindrical storage casks at PFS ground motions of 0.7 for a 2,000-year earthquake or 1.25 to 1.33 g for a 10,000-year earthquake. Tr. (Singh) at 5936.

Although Staff witness, Dr. Vincent Luk, has very limited experience, the Applicant did not challenge Dr. Luk's credentials. In 1999, the NRC commissioned Dr. Luk to conduct a generic study of the seismic behavior of dry storage casks. Tr. (Luk) at 6763. An offshoot of this study is Dr. Luk's study of casks at the PFS site. State Findings ¶ 377-78; Staff Exh. P. Dr. Luk's sole experience modeling unanchored casks is limited to work with this NRC generic study. Id. at 6764, 6914-16, 7051. To date, other than the PFS analyses, Dr. Luk conducted only one other seismic analysis of unanchored, cylindrical casks - at

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[33]The Applicant does not claim Dr. Singh has experience modeling the seismic behavior of casks. *See e.g.,* PFS Findings ¶ 162.

[34]Dr. Singh testified that Holtec casks are also currently stored at Southern Company's Hatch Plant. Tr. (Singh) at 5918. However, there is no evidence that Dr. Soler or anyone representing Holtec modeled the seismic behavior of casks at Hatch. During the hearing, Dr. Singh also thought cask may have been loaded at J.A. Fitzpatrick. Id. The ground motions at J.A. Fitzpartick are not in the record.

Southern Company's Hatch plant. Id. at 6763-67, 6914-15. However, the ground motion at Hatch is only 0.15 g (vertical) and 0.1 g (horizontal). Id. at 6914-15. Thus, Dr. Luk also does not have any previous experience modeling the seismic behavior of free standing, cylindrical casks at ground motions of 0.7 g or greater.

PFS's challenge to Dr. Khan's credentials rings hollow in light of the fact that PFS's witnesses Drs. Singh and Soler and Staff witness, Dr. Luk, all have very little direct experience in analyzing free standing casks at a site with strong ground motions. Based on Dr. Khan's eminent qualifications to testify in this matter, the Board may take comfort in relying on his opinions.

**B. DYNAMO Computer Code at PFS Ground Motions Is Not Reliable.**

To support its ISFSI license application, the Applicant relied upon the Holtec generated document, *Multi Cask Response at PFS ISFSI from 2,000-yr Seismic Event (Revision 2)*, Rev. 1 (August 2001) ("Holtec 2,000-year report") (State Exh. 173 claimed by Holtec to be proprietary). Tr. (Gaukler) at 5939. Holtec used a small deformation computer code DYNAMO, to predict cask behavior in its 2,000-year report. *See* State Exh. 120 at 27; State Exh. 173. Holtec maintains its DYNAMO code as proprietary information. Tr. (Singh) at 5922.

In an attempt to justify the use of the small deformation code, in its Findings, the Applicant states DYNAMO "has been validated and has been reviewed and accepted by the NRC for the licensing of freestanding spent fuel storage systems."[35] PFS Findings ¶ 137.

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[35]Not surprisingly, Staff Findings also reference the Applicant's testimony claiming DYNAMO "has been validated and approved by the NRC." Staff Findings ¶ 5.36. As the

Mere recitation that the Staff accepted DYNAMO in the past is not sufficient for the Board to base its findings upon. "[S]taff views 'are in no way binding upon' the boards; they cannot be accepted without passing the same scrutiny as those of the other parties." Indian Point, ALAB-304, 3 NRC at 6 (*citing* Southern California Edison Co. (San Onofre Units 2 and 3), ALAB-268, 1 N.R.C. 383, 400 (1975)); *see also*, Texas Utilities Generating Co. (Comanche Peak Steam Electric Station, Units 1 and 2), LBP-82-87, 16 NRC 1195, 1200 (1982), *vacated on other grounds*, CLI-83-30, 18 NRC 1164 (1983).

Moreover, the Applicant did not proffer a single case in which the Staff accepted the use of DYNAMO for unanchored, cylindrical casks at ground motions equal to or in excess of the 0.7 g. Diablo Canyon is the only site identified by the Applicant where ground motions are equal to or greater than 0.7 g.[36] State Finding ¶ 262. Notably, NRC has never "accepted" the seismic analysis of cask behavior at Diablo Canyon using Holtec's DYNAMO code. Tr. (Singh) at 5935. Moreover, the recently submitted ISFSI application for Diablo Canyon employs anchored Holtec casks – not unanchored casks. Id. at 5912-13, 5934-35. Nevertheless, the anchored casks at Diablo Canyon are not analyzed with

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record is devoid of any evidence in support of this assertion, Staff Findings do not (nor could they) provide any further discussion concerning the relevance to this case of the Staff's prior acceptance of DYNAMO.

[36]The Holtec witnesses testified that DYNAMO was used to perform seismic analysis of casks at the Excelon Dresden plant, the Entergy Northwest Columbia Generating Station, the Tennessee Valley Sequoyah plant, the Entergy J.A. Fitzpatrick plant, and the Pacific Gas and Electric Diablo Canyon plant. Singh, Soler Tstmy, Post Tr. 5750 at 14. Except at the Diablo Canyon site, there is no evidence that the ground motion at the other sites even approaches 0.7 g (as it does for a 2,000-year earthquake at PFS). *See* State Findings ¶ 262.

DYNAMO. Id. at 5935. There is no evidence that NRC has accepted any seismic analysis of unanchored dry storage casks using DYNAMO, or VisualNastran,[37] where the ground motions are 0.7 g or greater as at the PFS site. Tr. (Singh) at 5935-37. In fact, other than in this case, Dr. Singh admitted that DYNAMO has never been used to analyze a specific site with ground motions at or greater than 0.7 g. As a result, the Applicant's reference to NRC's past acceptance of DYNAMO or Holtec's prior use of DYNAMO at other ISFSIs has no bearing in this case.

Additionally, the Applicant refers to its prefiled testimony that the DYMAMO code has been validated against known solutions or other industry codes such as ANSYS. PFS Findings ¶¶ 139, 153. The Applicant thus concludes that "DYNAMO has been extensively benchmarked" to confirm its adequacy as a nonlinear dynamic code. Id. State Findings challenge and describe flaws in the Applicant's conclusion, including where the Applicant's own witness challenges the accuracy of ANSYS while in a different vain the same witness claims DYNAMO's accuracy is proven by its agreement with ANSYS results. *See* State Findings ¶ 283. The Applicant proffered no evidence that supports its claims that prior "benchmark[ing], validat[ion], and acceptance[]" of DYNAMO are relevant to the issue in this case whether DYNAMO will accurately model and predict the seismic behavior of casks at ground motions of 0.7 g or greater. *See* PFS Findings ¶ 153. It is spurious logic to claim DYNAMO provides accurate results at ground motions of 0.7 g based on comparable results obtained from both DYNAMO and ANSYS to unidentified analytical solutions (*see*

[37]Holtec also performed simulations of the seismic behavior of casks using VisualNastran. *See* Applicant's Exhs. 86, 225.

Singh/Soler Tstmy, Post Tr. 5770 at 20) while concurrently claiming that ANSYS produced "unstable, actually incorrect results" when simulating the seismic behavior of a free standing structure (Tr. (Singh) at 6099).[38] [39] *See* State Findings ¶ 283. Dr. Soler has never used ANSYS to analyze cask stability. PFS Exh. 224 at 36.

To further justify its use of DYNAMO, the Applicant relies, in part, on Dr. Soler's comfort level in using DYNAMO for the 2,000-year analyses. PFS Findings ¶ 151. To support his "comfort" level, although he did not quantify the amount, Dr. Soler presumed DYNAMO could handle large rotations on the order of twenty degrees or less.[40] Id. The Applicant also suggests that Holtec's DYNAMO runs for a 2,000-year earthquake at PFS were validated with VisualNastran runs. Id. ¶ 152.

In its Findings, the Applicant ignores important evidence. First, there is no evidence that Holtec's model run with DYNAMO or VisualNastran have been validated with performance or test data. The Applicant is solely relying on the accuracy of the predictions of nonlinear computer models. When relying solely on the predictions of

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[38]In a deposition, Dr. Soler describe two instances when DYNAMO results were compared with ANSYS results. Applicant's Exh. 224 at 36-37. According to Dr. Soler, DYNAMO was in "good agreement" with ANSYS for a "utility made up problem" that had "no relation" to modeling the spent fuel racks. Id. (*emphasis* added). The other DYNAMO - ANSYS comparison involved the results of spent fuel rack analyses where there were "small" differences in displacements and general agreement in forces. Id. at 37.

[39]Notably, although he later refuted the accuracy of ANSYS, Dr. Singh admitted that ANSYS would be capable of accommodating cask turn over. Tr. (Singh) at 5928.

[40]Dr. Soler previously testified during a deposition that the maximum angle of rotation that DYNAMO is capable of accurately processing is less than fifteen degrees, not the twenty degrees he claimed during the hearings. State Exh. 120 at 29-30.

computer models, as confirmed by the Applicant's witness, the Board should heed caution and not become too enamored with computer models and lose sight of ensuring the models are anchored in reality. *See* Tr. (Cornell) at 8024-25 (*confirming* Judge Farrar's concern with computer models).

Next, in an eight cask per pad analyses, Holtec estimated a maximum displacement of 10.5 inches for cask # 5 using VisualNastran for a 2,000-year earthquake at PFS. Applicant's Exh. 225 at 11, 18, 29. Whereas other eight casks per pad runs with DYNAMO and VisualNastran, Holtec estimated the maximum cask displacements for cask # 1 at two to four inches and for cask # 5 of less than two inches. Id., Applicant's Exh. 86D at 13, State Exh. 173 at 28-29. The cask # 5 displacement with VisualNastran of 10.5 inches is not confirmation of DYNAMO results; instead, it brings into question the reliability and accuracy of DYNAMO results.

Furthermore, Dr. Soler is one of the co-authors and owners of the proprietary claimed code DYNAMO. Tr. (Singh) at 5875. Also, as one of the owners of the privately held corporation Holtec, Dr. Soler has a tremendous personal financial interest in the outcome of this proceeding. *See* State Findings ¶¶ 257-260. Bias or interest in the outcome of a case goes to the persuasiveness or the weight accorded to an expert's testimony. Lousiana Power and Light Company (Waterford Steam electric Station, Unit 3), ALAB-732, 17 NRC 1076, 1091 (1983)(*citing* 11 J. Moore & H. Bendix, Moore's Federal Practice ¶ 702.30[1] (2d ed. 1982)). Inasmuch as Dr. Soler has a huge personal financial stake in the Board's acceptance of his DYNAMO results, Dr. Soler may be too enamored with his own code as one of its co-authors, and in the absence of any validating test data, Dr. Soler's

testimony that he is "comfortable" using DYNAMO to analyze a cask behavior at 0.7 g should be given little or no weight.

In summary, the Applicant has not shown that the results produced by DYNAMO of the seismic behavior of casks for a 2,000-year return period at the PFS site are reliable and accurate. Thus, the Board cannot find that the Applicant met its burden in demonstrating that unanchored casks will not tip over when subject to the 0.7 ground motions for a 2,000-year earthquake at the PFS site.

### C. Applicant's Contact Stiffness Is Not Validated and Underestimates Cask Behavior.

The cask vendors used contact stiffness to model the compression contact at the cask and pad interface. PFS Findings ¶ 143. Contact stiffness is the amount of force applied at the interface points of contact between two bodies. Id. ¶ 154. The value of contact stiffness used in Holtec's nonlinear analyses is in dispute between the parties.

By conducting a parametric study, Dr. Khan determined that nonlinear mathematical models are highly sensitive to the contact stiffness between the cask and the storage pad. Khan/Ostadan Tstmy, Post Tr. 7123 at 6. Dr. Khan testified that in the absence of test data, the purpose of selecting the contact stiffness to correspond with the amplified region of the seismic response spectra curve is to conservatively capture the dynamic behavior of the cask, including cask rotation and rocking. Id. at 12-13.

In discussing Dr. Khan's opinion that contact stiffness should be selected to correspond with a frequency of the cask in the amplified spectral region of the earthquake, the Applicant complains that the vertical response of the cask would be artificially

maximized. PFS Findings ¶¶ 156, 158. Dr. Khan's opinion, contrary to the Applicant's suggestion, is not to artificially inflate cask behavior but to ensure all potential cask behavior is encompassed in the seismic design due to the uncertainty resulting from no test data to confirm a dynamic contact stiffness that would envelop the low rocking frequencies of the casks.[41] Significantly, the Applicant fails to address that its selected vertical contact stiffness value of 454 million pounds per inch results in the vertical frequency of the cask being too rigid.[42] Khan/Ostadan Tstmy, Post Tr. 7123 at 11. If the vertical frequency of the cask is too rigid, then the solution is treated as linear, thereby significantly underestimating the vertical behavior of the casks. Id.

Neither PFS's Findings nor the Applicant's testimony dispute Dr. Khan's opinion that the seismic analysis using a contact stiffness value of 454 million pounds per inch corresponds to a cask frequency where the vertical acceleration is minimized which artifically reduces the vertical displacement of the cask.[43] In fact, Drs. Soler and Singh confirmed Dr.

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[41]Dr. Soler admitted he has no test data to validate the results obtained with his contact stiffness values. Tr. (Soler) at 6054.

[42]Dr. Khan's testimony refers to the contact stiffness value used by Holtec as 450 million pounds per inch as compared to the 454 million pounds per inch stated above and throughout the State Reply.

[43]Holtec simulated cask behavior using a lower contact stiffness of 38.19 million pounds per inch and 4.9 percent critical damping. *See* PFS Exh. 225 at 29. The displacements for cask # 5 increased over five times the displacement estimated for cask # 5 using a contact stiffness of 454 million pounds per inch. Id., State Exh. 173 at 42. Additionally, Holtec simulated cask behavior using a lower contact stiffness of 4.76 million pounds per inch and 40 percent critical damping. PFS Exh. 225 at 29. While Holtec lowered the contact stiffness value, it did not conduct the run at the five percent critical damping that it used in its 2,000-year Report that supports the Applicant's licensing application. State Exh. 173.

Khan's opinion that a static contact stiffness of 454 million pounds per inch corresponds to a cask natural frequency of 111 hertz, which is in the rigid range or outside the amplified region of the earthquake response spectra. Tr. (Soler) at 9635, Tr. (Singh) at 9636; *see also* PFS Findings ¶ 174 (*acknowledging* its contact stiffness value corresponds to a frequency "far above" the spectral range). While the Applicant's witnesses agree that its selected contact stiffness treats the cask in the rigid range (or corresponding to low vertical accelerations such that the vertical displacement is substantially reduced), Dr. Singh testified that whether Holtec's contact stiffness value artificially reduced the vertical displacement of the cask "bears no relevance to the way one would do a structural evaluation." Tr. (Singh) at 9638. Although it has no test data to support its selection of contact stiffness, the Applicant relies on its testimony that its static contact stiffness value is applicable in a nonlinear dynamic analysis notwithstanding its witnesses' acknowledgment that using a contact stiffness of 454 million pounds per inch essentially reduces the vertical behavior of the cask.

PFS Findings claim the cask vendor's contact stiffness of 454 million pounds per inch, which essentially causes the analyses to substantially reduce the vertical behavior of the cask, is supported by (a) Drs. Soler's and Singh's "extensive experience," (b) Dr. Khan's purported lack of experience, (c) the ANSYS training manual, (d) validation of DYNAMO's model against classical solutions, (e) the purported agreement of Dr. Luk's analyses, (f) the additional Holtec simulations, and (g) "the logic of their position." PFS Findings ¶ 161. As discussed earlier, Dr. Khan's focus of his career relates to conducting seismic analysis of structures, systems, and components in high seismic areas, which eminently qualifies him as an expert in this matter and is not a supportable basis to disregard Dr. Khan's opinions and

parametric study. *See supra.* In a shallow attempt to sway the Board, the Applicant emphasizes that Dr. Khan has not previously selected a contact stiffness for a nonlinear seismic analysis of dry storage casks. PFS Findings ¶ 163. The Applicant apparently ignores the fact that Dr. Khan's opinion relies on well-established seismic modeling practices in the nuclear industry, his vast experience performing dynamic seismic analyses, engineering principals, and his parametric study showing the cask displacement results are dependent upon the value of contact stiffness. Importantly, Dr. Khan does not advocate a single contact stiffness without validating test data. In the alternative to validating test data, Dr. Khan opines that the contact stiffness should account for potential cask behavior and not substantially reduce the vertical behavior of the cask as with Holtec's 454 million pounds per inch value. Dr. Khan's approach is conservative, but persuasive in that it is not only founded in seismic analysis practices but provides reasonable assurance that cask behavior is adequately enveloped during an earthquake.

The Applicant claims Drs. Singh and Soler have "extensive" experience which supports their selection of the contact stiffness value. PFS Findings ¶¶ 161-62. However, Dr. Singh's twenty-three years in designing nuclear spent fuel systems and Dr. Soler's oversight of the HI-STORM seismic stability analyses do not *a priori* translate to qualifications in selecting a contact stiffness value for the unanchored cask seismic analyses in this case.[44] *See* id. ¶ 162. In fact, a careful review of Drs. Singh's and Soler's experience

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[44]PFS states that Drs. Singh and Soler have collectively approximately 40 years of experience. PFS Findings ¶ 162. Additionally, PFS proclaims that "Drs. Singh and Soler for Holtec have decades worth of experience in understanding the mechanics and dynamics of storage cask behavior." Id. ¶ 240. Dr. Singh testified that Holtec started to develop its dry

exposes severe limitations in that their only experience in modeling the seismic behavior of unanchored casks at ground motions of 0.7 g or greater is apparently related to this PFS case.[45] The sites in which Drs. Singh and Soler claim experience in estimating the seismic behavior of unanchored casks all have ground motions of at least 0.6 g or below.[46] *See* State Findings ¶ 262. At sites with lower ground motions it may be inconsequential whether the contact stiffness minimizes the vertical acceleration of the cask to where cask rocking, uplift, sliding, and tip over are unlikely because of low ground motions. Accordingly, there is no evidence that Drs. Singh's and Soler's experience when compared to Dr. Khan's experience in selecting a contact stiffness for an unanchored cask weighs in the Applicant's favor.

The Applicant vainly attempts to find support for the 454 million pound per inch contact stiffness from Staff witness, Dr. Vincent Luk. Applicant Findings ¶ 165. Dr. Luk did not input a specific contact stiffness value into his model and testified that his model "deal[s] with the dynamic behavior," including the calculation of contact stiffness based on input characteristics. Tr. (Luk) at 6810-12. The record does not distinguish whether Dr. Luk's model calculated a single static contact stiffness to use throughout the dynamic

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cask technology in 1991 – or eleven years ago. Tr. (Singh) at 5915. Thus, their dry cask experience must be limited to eleven years or less.

[45]The Holtec witnesses testified they conducted a seismic analysis at Diablo Canyon. (Singh/Soler Tstmy, Post Tr. 5770 at 14), but given that anchored casks are proposed at Diablo Canyon (Tr. (Singh) at 5912-13), it is unclear whether that analysis was conducted for anchored casks.

[46]There is no evidence of the ground motions at J.A. Fitzpatrick. However, the Applicant did not proffer any evidence that Drs. Singh or Soler performed any seismic analyses of unanchored casks at sites with ground motions of 0.7 g or greater. Tr. (Singh) at 5935-36.

analysis. Nor does the record support how Dr. Luk's model calculated the contact stiffness or the actual value(s) calculated. Specifically, the fact that the contact stiffness value(s) used in Dr. Luk's model was based on "well established theory" also does not confirm Holtec's approach (described in PFS's Exh. 226) or contact stiffness value. *See* PFS Findings ¶ 165. There is no evidence that Dr. Luk offered an opinion on Holtec's 454 million pound per inch contact stiffness value. Nor is there any evidence that Dr. Luk concurred with the use of a contact stiffness value which underestimates the vertical behavior of the cask. As a result, Dr. Luk's testimony does not support Holtec's use of a 454 million pound per inch contact stiffness value.

Also to support Holtec's contact stiffness value, the Applicant refers to its witnesses' testimony concerning guidance in the ANSYS training material. PFS Findings ¶¶ 167-171; State Findings ¶¶ 315-16. The ANSYS guidance proffered by the Applicant does not support Holtec's contact stiffness values. It should also be noted that Dr. Soler admitted that "no one [contact stiffness] number is necessarily correct." Tr. (Soler) at 6049. Moreover, ANSYS guidance does not specify a contact stiffness value. Id. 5902. The training manual states the "*Hertz contact stiffness* often provides an appropriate basis for the penalty stiffness," but does not emphatically state that the Hertz contact stiffness is, in fact, the appropriate value. PFS Exh. 221 at 3-6 (*emphasis added*). Additionally, the ANSYS training guidance clearly states that "[d]etermining a good stiffness value usually requires some experimentation," suggesting that the Hertz or other contact stiffness value is a starting point in determining the appropriate contact stiffness value. PFS Exh. SS at 3-14 (*emphasis added*). While the training guidance provides some support to use a Hertz contact

stiffness as a starting point for some problems, for a dynamic sliding, uplift, and rocking problem, as in this case, there is no evidence that the ANSYS training guidance confirms Holtec's contact stiffness value to be used for unanchored casks under high ground motion.

Further, while the Applicant attempts to bolster its argument by proclaiming ANSYS devotes over a 100 pages to friction and contact problems, it failed to address Dr. Soler's admission that nowhere in the ANSYS guidance are recommendations for selecting a contact stiffness to analyze sliding or uplift. Tr. (Soler) at 6051-52. Moreover, the fact that there is not a single sliding and uplift problem in more than 500 example problems provided by ANSYS lends support to Dr. Khan's claim that the ANSYS guidance proffered by the Applicant is not applicable for sliding or uplift problems as in this case. Id. The State addressed other issues raised by the Applicant with respect to the ANSYS guidance in the State Findings and will not repeated here. *See* ¶¶ 315-16.

Dr. Khan opined that in the absence of test data, selecting contact stiffness values ranging from 1 million to 10 million pounds per inch that correspond to cask frequencies in the amplified spectral range would conservatively account in the design for potential cask rocking, uplift, sliding, and tip over. Khan/Ostadan Tstmy, Post Tr. 7123 at 12-13. To defend its use of a contact stiffness which minimizes the vertical behavior of the cask, the Applicant attacks Dr. Khan's professional credibility by claiming the lower contact stiffness value of 1 million pounds per inch produce unrealistic static deflection results. PFS Findings ¶¶ 172-179. As detailed in State Findings ¶¶ 298 to 337, the key disputes between the parties is (a) whether in the absence of validating test data, a conservative design approach should be implemented to ensure all potential cask behavior is analyzed, and (b) whether a static

contact stiffness is applicable in a nonlinear dynamic analysis that will accurately capture all cask behavior, *e.g.*, rocking, uplift, and sliding.

Dr. Khan and Dr. Soler agree that a dynamic analysis is performed from a static equilibrium condition. Tr. (Khan) at 7211-12, (Soler) at 5783-84. Prior to the simulation, the dynamic analyses must be in static equilibrium and the time and earthquake velocities set at zero, then the ground motion is added. Tr. (Soler) at 5783-84. At the time the program is initiated the boundary conditions are set. Tr. (Khan) at 7211-12. For this reason, Dr. Khan maintains that initial static deflection in a dynamic analysis has no effect on the dynamic cask response. Id. Provided it is correctly selected, contact stiffness is used to determine whether the cask will experience any vertical amplification, or whether there will be rocking. Id. Thus, the initial static deflection based on the contact stiffness is not relevant in a dynamic analysis, as in this case. Thus, the Applicant's claim that a million pounds per inch contact stiffness produce unrealistic static deflection is pointless with respect to dynamic analysis.

While the higher value of Dr. Khan's recommended range for contact stiffness value of 1 million to 10 million pounds per inch is also substantially lower than Holtec's 454 million pounds per inch value, the Applicant's Findings is conspicuously silent and does not dispute the remainder of Dr. Khan's recommended range. Assuming *arguendo* that a static deflection calculation is relevant to dynamic analyses, a contact stiffness value of 10 million pounds per inch would generate a static deflection of 0.037 inches.[47] Dr. Soler testified that an acceptable static deflection or no physical penetration of the pad is "subjective" (Tr.

[47]The static deflection is equal to the mass of the cask (360 kips) divided by the contact stiffness. Singh/Soler Tstmy, Post Tr. 5770 at 81.

(Soler) at 6037) but that any static deflection value of ".0 something" would represent no visible penetration and correspond to an acceptable contact stiffness value (id. at 6037-38). Thus, even if the Board finds PFS's claims persuasive that the dynamic contact stiffness value must show no visible penetration as touted by Dr. Soler (*see* Singh/Soler Tstmy, Post Tr. 5770 at 80-82), in the absence of validating test data, the Applicant has not shown that the cask will not tip over at a contact stiffness of 10 million pounds per inch or lower, and that Holtec's 454 million pound per inch contact stiffness value which minimizes the vertical behavior of the cask is conservative.

With respect to Holtec's selection of contact stiffness in its computer estimate, Dr. Khan disputes the cask vendors contention that dynamic contact stiffness values between a cask and pad during a seismic event is the same as the static contact stiffness value. *See* State Findings ¶¶ 309-12, 314. Notwithstanding his opinion that the contact stiffness does not change throughout the dynamic seismic event, Dr. Soler admitted that the contact stiffness at a given contact point changes as the cask surface area in contact with the pad changes due to vertical uplift and rocking. Tr. (Soler) at 9645. Additionally, Dr. Singh testified that the contact stiffness could be calculated based on the stress between the cask and pad in the loaded contact patch area. Tr. (Singh) at 9620; *see also* PFS Exh. 226. Given that the loaded contact patch area will change with time as the casks rock, uplift, or tip, then it is reasonable to conclude the contact stiffness will also change with time.

Another way to look at the fundamental dispute between the parties is whether a dynamic contact stiffness value changes with time or is equivalent to the static contact stiffness. The Applicant alleges that "simple mathematical relationships between the natural

frequency of the cask under dynamic conditions" and static deflection support its use of a single static contact stiffness value. PFS Findings ¶¶ 173-74. These simple mathematical relationships, PFS claims, are the same formulas cited by Dr. Khan. Id. ¶ 173. However, the Applicant's arguments are misleading. Dr. Soler testified that he calculated the corresponding frequency of the cask and the pad by calculating the static deflection of a body resting on a surface. Tr. (Soler) at 9633. Next, Dr. Singh clarified that the frequency can be calculated, "which is what Dr. Soler just did," if the cask is assumed connected to the pad. Id. (Singh) at 9636. Or in other words, in order to simply relate the cask and pad frequency to a static deflection, one has to assume the cask is connected to the pad or is not behaving in a dynamic manner. As a result a static contact stiffness is used to determine the static deflection. While Dr. Khan refers to the same frequency formula used by Dr. Soler, the Applicant fails to inform the Board that immediately prior to his reference to the frequency formula, Dr. Khan testified that "[t]he contact stiffness changes with time as the contact between the cask and the pad change with time." Khan/Ostadan Tsmy, Post Tr. 7123 at 12. More importantly, Dr. Khan has never acceded to the use of a static contact stiffness value to analyze nonlinear dynamic behavior. Thus, implying Dr. Khan's testimony supports the Applicant's findings is erroneous.

Moreover, the Applicant again contradicts its own findings when it states based on testimony by the Applicant's witness, Dr. Wen Tseng, that "due to the nonlinear 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." PFS Findings ¶ 274 (*citing* Tseng Rebuttal Tstmny, Post Tr. 10727 at A.8; Tr. (Tseng) 10735-

36, 10752-54, 10772-76). If, according to PFS, (a) the frequency of the cask under dynamic conditions is dependent upon the static contact stiffness and the mass of the cask (PFS Findings ¶ 173; Tr. (Soler) at 9632-33, PFS Exh. 225 at 21-22), and( b) the static contact stiffness does not change with time, or in a dynamic situation (Tr. (Singh) at 9628), then the frequency of the cask and pad system would also not change with time which contradicts the testimony of PFS's other witness, Dr. Tseng. Dr. Khan's opinion that the dynamic contact stiffness changes with time is consistent with Dr. Tseng's testimony that the cask and pad system frequency changes with time. Accordingly, while a static contact stiffness may be used for simplicity to estimate the corresponding frequency of the cask and pad, the relationship does not support the use of a static contact stiffness in dynamic analyses to correctly account for the rocking frequency of the cask and pad system as it changes with time. Thus, PFS has not refuted Dr. Khan's opinion that a dynamic contact stiffness changes with time.

In passing, PFS Findings imply that dynamic contact stiffness would vary based on geographic location and earthquake characteristics if Dr. Khan's contact stiffness recommendations were followed. PFS Findings ¶ 172. While a discussion of the differing opinions in selecting contact stiffness has essentially been exhausted, it should be noted that in the Timoshenko and Goodier method, which the Holtec witness claims is the correct method to determine contact stiffness (Tr. (Singh) at 9623-24), apparently the underlying soil at the site is a factor. *See* PFS Exh. 226 at C-1 (*stating* "[w]e neglect the effect of the underlying soil since this effect is included in the spring set representing the soil behavior"). Thus, based on the Applicant's own exhibit, the contact stiffness, as calculated according to

the Timoshenko and Goodier theory, would change according to site specific soil or geographic location.

To support its contact stiffness value, the Applicant points to NRC's acceptance of DYNAMO in other licensing applications. PFS Findings ¶ 180. Dr. Singh admitted that NRC had not validated DYNAMO for any specific contact stiffness value and that because it is an input value,"the [Holtec] analyst has the burden to ensure, in running the program, that he's using the correct, appropriate [contact stiffness] value." Tr. (Singh) at 6030. The contact stiffness values used in other applications are not in evidence. Additionally, a nonlinear analysis of the cask's behavior may not be as sensitive to the contact stiffness value at ground motions which are lower than 0.7 g for prior NRC license applications. *See supra.* Moreover, NRC's opinions must "pass[] the same scrutiny as those of the other parties" and bear no special weight. Indian Point, ALAB-304, 3 NRC at 6 (string citations omitted), *see also supra.* Thus, NRC's acceptance of the DYNAMO model does not support use of a contact stiffness which essentially restricts the vertical movement of the cask.

Notably, according to a 1998 PFS member note, the Staff state "[t]he response spectrum for the acceleration time history chosen for the nonlinear analysis or confirmatory testing must be enveloped by the response spectrum . . . Furthermore the duration of the seismic event must be consistent with high acceleration levels. Large earthquakes that have high acceleration levels are associated with strong ground motion durations." *See* State Findings ¶ 307 and State Exh. 197A. The Staff's apparent position is consistent with Dr. Khan's opinion that the contact stiffness value must correlate with a frequency that must be enveloped within the amplified range of the response spectra curve. The 454 million pounds

per inch contact stiffness correlates to a frequency outside the amplified range of the response spectra.

Also the Applicant proclaims that VisualNastran simulations using lower contact stiffness values still show displacements in the order of inches. PFS Findings ¶ 181. Based on the Applicant's limited discussion, the State cannot speculate how the results of other simulations using different contact stiffnesses support using the 454 million pound per inch contact stiffness. Thus, the State submits that ¶ 181 does not support Holtec's contact stiffness value in modeling nonlinear cask behavior. Inconsistencies and contradictory results in the Applicant's numerous computer simulations will be addressed below.

Although the cask vendors later admit that their contact stiffness value of 454 million pounds per inch minimizes the vertical behavior of the casks (Tr. (Soler) at 9635, (Singh) at 9636), remarkably, Dr. Soler admitted that "we got acceptable answers in the 2,000-year return earthquake, so there was no incentive for us to lower the contact stiffness" (Tr. (Soler) at 6043). Additionally, in its proposed findings, the Applicant makes several attempts to discredit Dr. Khan's parametric study. Even without Dr. Khan's persuasive parametric study, the Applicant acknowledges that "dynamic analyses are extremely sensitive" to contact stiffness values. PFS Findings ¶ 171.

Therefore, the facts <u>not</u> in dispute are (a) the dynamic behavior of the casks are extremely sensitive to the contact stiffness value, (b) the contact stiffness value used to support PFS's license application essentially limits the dynamic vertical behavior, (c) there is no test data to validate Holtec's contact stiffness value, (d) because the cask vendors had "acceptable results" they did not evaluate the effects of lowering the contact stiffness value

in the license application DYNAMO analyses, and (e) dynamic analyses start from a static equilibrium condition. While the State submits that the issues in dispute weigh substantially against the Applicant, when considering only the facts not in dispute, the Applicant clearly has not validated its contact stiffness of 454 million pounds with any test data nor has the Applicant shown that its seismic analysis does not under estimate the behavior of casks.

**D. Holtec's Damping Value May Underestimate the Seismic Behavior of Casks.**

In its Findings, PFS claims "based on analysis and test data" that the impact damping between steel and concrete is <u>much greater</u> than 40 percent." PFS Findings ¶ 183. As addressed in State Findings ¶ 341, beyond Dr. Singh's oral testimony there is no evidence that the tests referenced by Dr. Singh have any relevance to the impact damping for the casks in this case. In fact, rather than proffer the analysis and test data purported by Dr. Singh, the Applicant presents an animation of falling balls and falling casks. The State details the lack of relevance of the falling casks to this proceedings in its Findings ¶¶ 343-44, 351. Moreover, Dr. Soler in essence contradicted Dr. Singh, and himself, by testifying that the impact damping of a cask "might not be 40 percent, but it is extremely unlikely that [it] would be as low as 1 percent."[48] Tr. (Soler) at 9912. For the Board to accept Dr. Singh's unsupported statements that the impact damping is higher than 40 percent, the Board would also have to find Dr. Soler's inconsistent testimony, as the modeler and analyst of the Holtec simulations, is not credible in this matter.

Notably, PFS Findings are mute as to Dr. Khan's opinion that the ground motion

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[48]In preceding testimony, Dr. Soler stated that critical damping "around 40 percent" is correct. Tr. (Soler) 9911.

acceleration for a 2,000-year earthquake at PFS is highly sensitive to the frequency of the system and impact damping. *See* Tr. (Khan) at 9495-99; State Exh. 195. The response spectra for 40 percent damping shows that the casks will behave in a rigid manner or restrict the vertical behavior of the cask. Tr. (Khan) at 9499-9500, State Exh. 195; *see also* State Findings ¶¶ 331-333. However, Dr. Soler did agree the amplification of a casks vertical oscillation would depend on the damping. Tr. (Soler) at 9635. In deference to the Board, the State will merely reference the State Findings ¶¶ 331 to 334, and 337 to 353, in order not to duplicate the State's findings that Holtec damping values underestimate cask behavior.

The Applicant further argues that Holtec's analysis is conservative because it did not take credit for structural, material, or radiation damping. PFS Findings ¶ 184. There is no evidence that quantifies the purported conservatism or whether the failure to take credit for structural, material, or radiation damping equates to nullifying an unjustified impact damping value. In fact, the Applicant would have the Board rely on unquantified claims of conservatism in the face of uncertainties upon uncertainties; which is a slippery slope that cannot be supported with evidence.

**E. Nonlinear Analyses Show Contradicting Results.**

In an attempt to avert the State's concerns with respect to the 2,000-year Holtec report, Holtec ran a succession of nonlinear computer simulations, but they were hastily and ineptly designed and do not address the State's concerns. In fact, as discussed below, some of the additional nonlinear computer simulations served to contravene the results in the

Holtec 2,000-year report.[49]

The Applicant claims "the Holtec model showed a maximum displacement of the cask on the order of 3 to 4 inches" for a 0.7 g, 2,000-year earthquake at the PFS site. PFS Findings ¶¶ 133, 144. In citing only prefiled testimony, the Applicant fails to adequately address conflicting testimony, including the various simulations run by the cask vendors themselves. Importantly, another simulation performed by Holtec for a 2,000-year earthquake predicted a maximum excursion from the top of cask # 5 of 10.5 inches (PFS Exh. 225 at 29), not the 3 to 4 inches stated in PFS Findings.[50] Holtec's own simulations question the accuracy of other Holtec simulation results and the safety factor claimed in PFS Findings ¶ 133.

Additionally, Holtec ran a 2,000-year simulation using VisualNastran which resulted in a maximum excursion of the top of cask # 1 of 3.7 inches. PFS Exh. 86d at 1, 13. Contrary to the Applicant's claims (*see* PFS Findings ¶ 243), the VisualNastran simulation is not confirmation or validation of the results in the Holtec 2,000-year report. Two critical

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[49]Although Holtec's own results conflict with the results in its 2,000-year report, the Staff states "the Holtec PFS site-specific analysis . . . was performed to provide a bounding solution." Staff Findings ¶ 5.110. The evidence does not, in fact, support that the Holtec 2,000-year report provides a bounding solution.

[50]The 2,000-year simulation showing a maximum excursion of the top of the cask of 10.5 inches was conducted with a contact stiffness of 38.19 million pounds per inch instead of the 454 million pounds per inch used in the Holtec 2,000-year report. PFS Exh. 225 at 29. While the State maintains that the nonlinear analyses are highly sensitive to the contact stiffness value (Khan/Ostadan Tstmy, Post Tr. 7123 at 6, 9), Holtec claims "the PFSF cask stability analyses are <u>not</u> highly sensitive to . . . contact stiffness." <u>Id</u>. at 30 (*emphasis added*). Moreover, PFS itself contradicts Holtec's claims in PFS Findings ¶ 171, when it admitted that dynamic analysis is extremely sensitive to the contact stiffness values.

input parameters - contact stiffness and damping - in the VisualNastran simulation were substantially different. Holtec ran the DYNAMO or the 2,000-year report simulation with a contact stiffness of 454 million pounds per inch versus running the VisualNastran simulation, case 1, with a substantially lower contact stiffness value of 18.86 million pounds per inch. *See* State Exh. 173; PFS Exh. 86D at 1-2. Also, Holtec used an impact damping of 5 percent of critical damping for the DYNAMO run versus 40 percent for the VisualNastran run. Id. In complete agreement with Dr. Khan, the Applicant states that dynamic computer analyses are "extremely sensitive" to the contact stiffness value. PFS Findings ¶ 171. Dr. Khan also determined through his parametric study, that the nonlinear behavior of casks are also sensitive to damping. Khan/Ostadan Tstmy, Post Tr. 7123 at 9, 11. In plotting seismic response spectra curves at various damping values, Dr. Khan showed that for 40 percent damping, the ground motion acceleration is essentially flat (or less than or equal to the zero period acceleration) or the vertical displacement of the cask is substantially reduced. *See* State Findings ¶¶ 331-333, 346-347. While, the maximum excursion of the top of the cask may correlate by happenstance, the VisualNastran and DYNAMO simulations did not use identical input values; thus, contrary to the Applicant's claims, the VisualNastran simulation does not validate the results obtained with DYNAMO.

In another attempt to address the State's concerns, Holtec ran nine simulations for a 10,000-year earthquake. *See* PFS Exh. 86D at 1. The Applicant professes that the casks in these simulations did not tip over; thus, the 10,000-year simulations validate the results in the

Holtec 2,000-year report.[51] PFS Findings ¶ 243. Notably, Holtec substantially increased the impact damping for the 10,000-year simulations from 5 percent critical damping used in the Holtec 2,000-year report to 40 percent, thereby substantially minimizing the vertical acceleration on the cask and restricting its vertical movement.[52] *See also*, PFS Exh. 86 at A-2. As addressed in State Findings ¶ 347, 40 percent impact damping minimizes the effect of vertical ground motion accelerations significantly and substantially reduces the vertical displacement of the casks. Importantly, the record does not support impact damping of 40 percent of critical damping. *See supra*; State Findings ¶¶ 338-353.

The Applicant further contends cask rotations of approximately 10.89 degrees from vertical will occur during a 10,000-year return period earthquake at the PFS site. PFS Findings ¶ 134 (*citing* Singh/Soler Tstmy, Post Tr. 5750 at 27-28), *see also* id. ¶ 148. Although the Applicant claims the 10.89 degree angle of rotation demonstrates the cask will not tip over (id.), Dr. Soler admitted he did not account for the effects of soil-structure interactions in this simulation resulting in a cask angle of rotation of approximately 10.89 degrees (Tr. (Soler) at 6002-03). Moreover, Staff witness, Dr. Luk opined that soil-structure interaction effects at the PFS site were "significant" and increased the ground motion compared to the

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[51]In some of the simulations, Dr. Soler admitted that adjacent casks hit or "appear" to hit one another. Tr. (Soler) at 5784.

[52]For the various cases, Holtec's table shows "[d]amping" of 1%, 5%, lower bound design basis, or Geomatrix lower bound values consistent with 10K. PFS Exh. 86D at 1. The "damping" expressed in the table is the amount of damping used in the soil dampers. *See e.g.*, Tr. (Soler) at 5772. All eleven cases used 40 percent impact damping as each case used a cask contact damping per facet of 4,549.05 pounds - second per inch (PFS Exh. 86D at 2-12) which is calculated based on 40 percent impact damping (PFS Exh. 86 at A-2).

free field. Luk/Guttman Tstmy, Post 6760 at A.20; Tr. (Luk) at 6930, 7012. Staff Findings also proclaim the "importance of the dynamic coupling or soil-structure interaction [] effect of the cask with the soil foundation" and the "presence of a significant [soil structure interaction] effect." Staff Findings ¶ 5.177, n.82. Therefore, the Applicant's simulation without attempting to account for the effects of soil structure interaction is insufficient in this case.[53]

Furthermore, as just discussed, for a 2,000-year earthquake, Dr. Soler himself estimated the maximum cask displacement for cask # 5 more than doubled that estimated for cask # 1. PFS Exh. 225 at 29. Although the Applicant only proffered evidence estimating the displacements of cask # 1 for a 10,000-year earthquake (*see* PFS Exh. 86D at 13, PFS Exh. 225), Dr. Soler admitted that his 10,000-year simulations show casks other than cask # 1 might have greater displacement (Tr. (Soler) at 5762). As a result, the Applicant's assertion in its Findings ¶¶ 134, 135, and 148, that a 10,000-year earthquake showed "maximum rotations on the order of 10 to 12 degrees" confirming margins of safety of 2.69 against tip over, cannot be substantiated in light of the Holtec simulations showing much greater displacement of cask # 5.

Holtec ran a few simulations where it only varied the number of casks on a pad under the same input parameters for a 10,000-year earthquake at the PFS site with 1 percent

[53]Remarkably, although stating that "soil springs were not included in the simulation" and given it's statement in its own findings (Staff findings at n.82) and the testimony of its own witness as to the significance of soil structure interaction effects, the Staff still conclude this simulation demonstrated "large margins of safety against cask tip-over still exists." Staff Findings ¶ 5.53.

soil damping, 40 percent of critical damping, and a coefficient of friction 0.8.[54] *See* PFS Exh. 86D at 1. Surprisingly, the results vastly differ as shown below:

| Case No. | No. of casks per pad | Maximum Excursion Top of Cask # 1 | Rotation Angle (degree)[55] |
|---|---|---|---|
| 4 | 1 | 56 inches | 5.37 |
| 6 | 3 | 18.6 inches | 4.47 |
| 7 | 4 | 52.5 inches | 10.13 |
| 8 | 8 | 39.0 inches | 8.72 |

The table is based on data in PFS Exh. 86D at 1, 13. All input parameters, including the stiffness value based on the mass of one cask, are the same for cases 4 and 6, yet the maximum excursion of cask # 1 is remarkably different - 37.4 inches. Additionally, the maximum displacement of case 4 and 7 are similar, yet, the maximum angle of rotation for case 7 is almost double that for case 4. These additional simulations are inconsistent and provide evidence that Holtec's simulations are, in fact, unreliable.

A Holtec simulation for a 10,000-year earthquake, coefficient of friction of 0.8, and using the damping associated with the Geomatrix lower bound soil values resulted in a maximum excursion of the top of the cask of 22.7 inches and an angle of rotation of 4.51

[54]The stiffness for the 3 casks simulation - case 6 - was bsased on the mass of 1 cask whereas the stiffness for the 4 casks simulation - case 7 - is based on the mass of 8 casks. PFS Exh. 86D at 1.

[55]Holtec's calculation based on half of the maximum peak to peak excursion. PFS Exh. 86D at 13. The State challenges that using half of the maximum peak to peak excursion instead of the maximum excursion underestimates the rotation angle. *See* State Findings ¶ 357.

degrees.[56] However, Dr. Luk's simulation using the same input parameters (10,000-year earthquake, coefficient of friction of 0.8, and the Geomatrix lower bound soil) resulted in a maximum excursion of the top of the cask of only 7.39 inches versus the 22.7 inches estimated by Holtec. Staff Exh. P at 32. These results indicate a significant variation in results and in no respect indicate any confirmation.

### F. Dr. Luk's Nonlinear Model Is Unreliable to Predict Cask Behavior at the PFS Site.

Not surprisingly, the Applicant champions Dr. Luk's nonlinear computer model which estimates the seismic behavior of casks at the PFS site.[57] The Applicant's Findings claim Dr. Luk's analyses confirm that the casks will not tip over. PFS Findings ¶ 198. Dr. Luk did not accurately model the PFS site conditions; thus, Dr. Luk's analysis is not confirmatory of the results in the Holtec 2,000-year report. State Findings describe the fatal flaws with Dr. Luk's analysis. *See* State Findings ¶¶ 377-444.

Additionally, in each of his simulations, Dr. Luk analyzed a single cask on a pad. Dr. Luk claimed that because the casks act independently of each other, there is no difference between the behavior of one, two, or eight casks on a pad.[58] Tr. (Luk) at 6779-80. Holtec's

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[56]Holtec's calculation is based on half of the maximum peak to peak excursion. PFS Exh. 86D at 13. *See* footnote *supra*.

[57]The NRC contracted Vincent Luk to conduct a nonlinear computer analysis of the seismic behavior of the casks at the PFS site. Tr. (Luk) at 6764. Dr. Luk's report was admitted as Staff Exh. P.

[58]Dr. Luk ran a single simulation with the dead load of an additional seven casks and adjacent concrete pads. Staff Exh. P at 30. Dr. Luk did not, however, attempt to determine the behavior of multiple casks on a pad or determine the consequences to one cask from adjacent casks.

results for a 10,000-year earthquake significantly contradict Dr. Luk's opinion. In fact, the maximum cask sliding displacements did vary based on the number of casks on a pad. The displacement differences recorded by Holtec ranged from 18.6 to 56 inches when modeling one to eight casks. *See* Reply *supra*; PFS Exh. 86D at 1, 13. According to Holtec simulations, while the actual maximum excursions of the top of cask # 1 in a single cask simulations versus a 4 cask simulation were similar, the 4 cask simulation almost doubled the angle of rotation: 5.37 to 10.13 degrees. *See* PFS Exh. 86D at 13.

Dr. Luk further testified that the location of the cask on the pad "didn't matter." Tr. (Luk) 6778. Again, Holtec's simulations contradict Dr. Luk's opinion. As previously discussed, a Holtec simulation resulted in the maximum excursion of the top of cask # 1 of 3.4 inches, whereas the excursion of the top of cask # 5 of 10.5 inches was significantly more. *See* PFS Exh. at 225 at 29.

Dr. Luk's simulation of a 10,000-year earthquake with a coefficient of friction between the cask and the pad of 0.8 shows the cask essentially purely <u>sliding</u> in the U2 direction (e.g., at 14.3 seconds, the displacement at the top of the cask is 7.39 inches, at the bottom of the cask is 7.08 inches). Staff Exh. P at 32. In fact, the Applicant's witnesses claimed a coefficient of friction of 0.8 would emphasize tipping and a coefficient of friction of 0.2 would emphasize sliding. Singh/Soler Tstmy, Post Tr. 5750 at 23. Contrary to the Applicant's testimony, Dr. Luk's results of the cask sliding in the U2 direction do not emphasize tipping. This shows that under high seismic ground motions the cask response could not be easily predicted and could be very sensitive to the input parameters, thus arguing for validation with test data.

Dr. Luk failed to accurately model the site conditions at the PFS site. Moreover, Dr. Luk's report does not aid but often contradicts Holtec's results. As a result of the above, including State Findings ¶¶ 377-444, Dr. Luk's analysis is not confirmatory of Holtec's conclusions.

G. Summary.

The sheer number of simulations conducted by both Holtec and Dr. Luk give the appearance that the Applicant and Staff conducted a comprehensive analysis of the potential seismic behavior of the casks at the PFS site.[59] However, an in-depth evaluation of the evidence proffered to support the Applicant's unproven and unprecedented design using unanchored, cylindrical casks in a seismic area reveals:

(a) While the Applicant's own witness, Dr. Allin Cornell, confirmed the Board's concern that it is a "danger" to become too enamored with computer models and not ensure the models are anchored in reality (Tr. (Cornell) at 8024), the Applicant is relying solely on a nonlinear computer simulation to claim the casks will not tip over at the requested design basis earthquake – 2,000-year return interval;

(b) The Applicant has not shown the computer code DYNAMO, which ran the nonlinear computer simulations, is in fact reliable at ground motions of 0.7 g or greater where the potential rotation may exceed the capability of DYNAMO;

(c) In fact, other than in this case, DYNAMO has never been used to analyze the

[59]The Staff accept the results of Holtec's various simulations in which it bases numerous findings without any discussion of the reliability or accuracy of the results. *See e.g.*, Staff Findings ¶ 5.50.

seismic behavior of casks at a site with ground motions of 0.7 g or greater;

(d) Nonlinear computer analyses are sensitive to input parameters (Tr. (Ostadan) at 7335-36, (Tseng) at 5695, (Cornell) at 8009 (*agreeing in part*));

(e) The input values and results of the nonlinear computer simulation have not been validated with either performance or test data;

(f) More specifically, the unvalidated contact stiffness of 454 million pounds per inch used in the nonlinear computer simulation minimizes the vertical movement of the cask (*e.g.*, rocking, uplift, and tip over); thus the displacement is underestimated;

(g) The additional Holtec computer simulations for a 2,000-year earthquake do not validate the DYNAMO results inasmuch as the additional simulations either have conflicting results or cannot be compared because of dissimilar input parameters;

(h) The additional Holtec simulations for a 10,000-year earthquake do not confirm the casks will not tip over because the simulations contravene results from other runs and the impact damping was unjustifiably increased from 5 percent to 40 percent, which again causes the cask behavior to be underestimated; and

(i) Dr. Luk did not model the PFS site and furthermore, he obtained results inconsistent with those obtained by Holtec.

Accordingly, based on the preceding, the Applicant has failed to meet its burden in demonstrating that the unanchored, vertical HI-STORM 100 casks will not tip over at ground motions of approximately 0.7 g for a 2,000-year earthquake at the PFS site.

## VI. Seismic Exemption Request.

### A. Staff's Rationale for Grant of an Exemption to PFS

The State has submitted detailed findings on the Staff's rationale – or lack thereof – in supporting PFS's exemption request and will not reiterate them here. *See* State Findings ¶¶ 447-97. There are, however, a few points footnoted in the Staff Findings to which the State will respond. Specifically, the Staff's rendition of the State's testimony relating to the Geomatrix seismic hazard curve, Dr. Stamatakos's reliance on the Martinez paper, and the geologic slip rates computed by Geomatrix. Further, while PFS offered no testimony in support of the Staff's position, in its Findings PFS attempts to prop up the Staff's position. The State takes exception to conclusions PFS draws from the Staff's position and will respond thereto.

Turning to footnote 110 in Staff's Findings ¶ 6.57, the NRC Staff remarks that "State witness Dr. Arabasz similarly concluded that the PFS seismic hazard curve developed by Geomatrix was conservative. Tr. 9973-75." In the cited testimony, Dr. Arabasz answered "That's correct" in response to NRC counsel's question, "And your conclusion was that that was developed correctly and conservatively." Tr. (Arabasz) at 9974-75. From the compound syntax of the preceding question, one can parse "that" to be referring to "seismic hazard curve," but Dr. Arabasz's expanded testimony makes it clear that what he concluded to have been done "correctly and conservatively" by Geomatrix was "specifically the development of the site response spectra [given a seismic hazard curve] following the standard review plan . . ." Id. at 9974. This testimony was revisited in the State's redirect examination of Dr. Arabasz, and the record is clear that Dr. Arabasz used the descriptor "conservatively" not to mean that Geomatrix overestimated ground motions, but that an appropriate envelope was achieved in translating the hazard results into site-specific

response spectra. Id. at 10054-55. Clearly, the Staff has no support from Dr. Arabasz that Geomatrix's PSHA hazard curve is "conservative" and, as evidenced in the State's Findings, it should also receive none from the Board.

In its Findings, for the first time, PFS also join the Staff on the Geomatrix PSHA conservatism bandwagon. In Findings ¶ 471, PFS concludes that "the 2,000-year DBE constitutes a conservative prediction of the seismic hazard at the PFSF" based on the Staff's identification of "what [the Staff] considers to be many conservatives in the Geomatrix PSHA."[60] To bolster its conclusion, PFS states:

> Dr. Arabasz . . . 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).

PFS Findings ¶ 466.

Dr. Arabasz agreed to the adequacy of Geomatrix's PSHA for the PFS facility site (Tr. 9119-20), but he was deliberate in not agreeing to the Staff's conclusion that Geomatrix's PSHA was conservative (Tr. 9860-63, 9878-79, 10128-31). He, indeed, rejected comparisons the Staff made between the results of Geomatrix's PSHA and results from other seismic hazard analyses along the Wasatch Front, as cited by PFS *supra*. Summarizing varied elements from Dr. Arabasz's testimony and the State's cross-examination of the Staff's witnesses, implicitly involving Dr. Arabasz's concerns, the State's Findings include a key section that systematically addresses and finds against the asserted conservatism of

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[60] *See* argument developed in PFS's Findings ¶¶ 459-465.

Geomatrix's PSHA. *See* State Findings ¶¶ 479-92, 494.

The Staff continues to cling to a scientifically indefensible paper by Martinez to support its slip tendency analysis. Staff Findings ¶¶ 6.66 and 6.72. In footnote 113 to Staff Findings ¶ 6.66, the Staff erroneously claims that "Dr. Arabasz did not criticize the Martinez paper insofar as it relates to the orientations of the principal stresses. *See generally*, Tr. 9865-78, 10128-30." The cited parts of Dr. Arabasz's testimony generally relate to the issue of GPS-derived slip rates. The State's criticism of, and Findings related to, the Staff's inferences of the orientations of principal stresses are summarized in State's Findings ¶¶ 482-83. Footnote 113 reinforces the State's position that the Staff has not justified the grant of an exemption to PFS.

Footnote 114 of the NRC Staff's Findings contains two assertions that cannot be left unchallenged. The first reads as follows:

> As noted above, Dr. Arabasz criticized the Martinez paper's means of establishing the rate and amount of fault slip, based on GPS data. However, while this may constitute a new application of GPS technology to which Dr. Arabasz or others may not subscribe, we have been provided no reason to conclude here that such use is **truly improper**. *See generally*, Tr. 9865-78, 10128-30.

Staff Findings ¶ 6.72, n. 114 (*emphasis added*). Among the reasons that it is not "truly proper" for the Staff to use the Martinez paper for establishing "the rate and amount of fault slip" on the Wasatch fault, based on the GPS data, are the following: (1) Martinez and her co-authors explicitly caution that the GPS measured horizontal deformation field may be due to causes other than loading of the Wasatch fault (Tr. 10129-30); (2) comparison between GPS-derived slip rates in one location with geological slip rates in another location— as opposed

to comparison of two GPS slip rates or two geological slip rates— is indeed like comparing "apples and bananas" (Tr. (Stamatakos) at 8237; *see also* Tr. (Arabasz) at 9873).

The second assertion that cannot be left unchallenged in footnote 114 reads:

> Moreover, the State has conceded that the Geomatrix analysis disregarded slip rate variances along different fault segments, thus rendering the analysis conservative, in its view, by a factor of three. Thus, while Dr. Stamatakos relied on GPS data in finding the Geomatrix analysis to be conservative (by a factor of ten), the only issue in dispute here is the degree to which the Geomatrix analysis may be conservative due to its treatment of slip rate. See Tr. 9875-80.

Staff Findings ¶ 6.72, n. 114 (*emphasis in original*). Under cross-examination, Dr. Stamatakos conceded that in comparing geologic slip rates compiled by Geomatrix, the slip rate for the unsegmented model of the Wasatch fault was a factor of three (not ten) greater than the slip rate for the Stansbury fault. Tr. (Stamatakos) at 8233-36. Similarly, Dr. Arabasz compared the same slip-rate data under cross-examination and reached the same conclusion of a factor of three difference. No simple logic relates the relative slip rate of these two faults to relative conservatism of "the Geomatrix analysis." *See* Tr. (Arabasz) 9878-79. The Staff's conclusion that the Geomatrix PSHA is conservative based on a comparison to PSHA results in and around Salt Lake City is systematically addressed and challenged in State Findings ¶¶ 485-89. The Staff's Findings on these points are unsupportable and do not sustain granting PFS an exemption to use of a 2,000-year seismic design standard.

In its Staff-supportive Findings, PFS summarizes arguments put forward by Dr. Stamatakos that either the design basis ground motions from Geomatrix's PSHA are overly conservative or the PFS facility site warrants classification as a "high seismicity" site and, therefore, a 5,000-year NPP SSE benchmark. PFS Findings ¶¶ 468-69. This false dilemma

is addressed specifically in State Findings ¶ 491, which follows State Findings ¶¶ 479-90 (challenging the Staff's conclusion that Geomatrix's PSHA is conservative) and ¶¶ 451-63 (no support for Staff's 5,000-year NPP SSE benchmark).

As to the high seismicity argument, PFS incorrectly states "[t]he 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." PFS Findings ¶ 467. The State does not agree with this proposition, worded in terms of the Staff's fuzzy thinking. The key issue is the slope of the hazard curve at the PFS site and whether one achieves a risk reduction ratio of 20 or more that can justify, in the case of DOE PC-4 facilities, a 5,000-year reference ground motion versus a 10,000-year ground motion. *See* State Findings ¶ 461-62.

In addition, PFS downplays the significance of the Staff's testifying to a 5,000-year value as the appropriate NPP benchmark to which one should compare the DBE for the PFS facility design; rather, PFS attaches importance simply to the consistency of having the 2,000-year DBE for the PFS facility be lower than that of a NPP, whether the latter corresponds to a 5,000-year or 10,000-year return period value. PFS Findings ¶ 470. The reason the State has spent so much time in this hearing process challenging the Staff's adoption of a 5,000-year NPP benchmark (*e.g.*, State's Findings ¶¶ 451-463) was the avowal in Staff's testimony that it had made a judgment and final determination of 2,000 years as being the appropriate number (*i.e.*, return-period level) for the DBE at the PFS facility by relative bracketing between 1,000 years and 5,000 years. Tr. 8325-26 (McCann). The Staff has not recanted that untenable position and on inspection the Board will find the Staff's

bracketing offers no support for granting PFS an exemption to allow use of a 2,000-year MAPE for the design basis earthquake for its nuclear storage facility.

**B. PFS's Rationale for Grant of an Exemption.**

The State sees no reason to respond to the Staff's Findings relating to PFS's justification of its seismic exemption request. During the hearing, the Staff concentrated its efforts in defending its ad hoc justification in granting a substantial reduction in the design basis earthquake standard. The Staff offered essentially no evidence to support Dr. Cornell's premise for granting PFS's seismic exemption. Now, in its Findings the Staff champions Dr. Cornell's theories. In addition to a last ditch attempt to prop up its justification, the Staff's Findings for the most part are replete with gross generalizations and summaries of various testimony.

As previously discussed, the Board in its findings of fact and conclusions of law has a "duty not only to resolve contested issues but 'to articulate in reasonable detail the basis' for the course of action chosen.'" *See* Public Service Company of New Hampshire, et al. (Seabrook Station, Units 1 and 2), ALAB-422, 6 NRC 33, 40-41 (1977), *aff'd* CLI-78-1, 7 NRC 1, *aff'd sub nom* New England Coalition on Nuclear Pollution v. NRC, 582 F.2d 87 (1st Cir. 1978), (*quoting* Northern States Power Co. (Prairie Island Nuclear Generating Plant, Units 1 and 2), ALAB-104, 6 AEC 179 (1973). As such, the Board must explain its reasons for rejecting contrary opinions from expert witnesses.[61] Id. At 40.

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[61]In Seabrook, the Appeals Board noted that the Licensing Board decision fell short of "applicable standards," in part, due to its "lack of reference (much less discussion) of evidence contrary" to what the Licensing Board accepted. Seabrook at 41. The Seabrook Appeals Board further reasoned that the Licensing Board had an obligation to explain why it

Again the Staff Findings are of little benefit to the Board because they essentially disregard the State's expert testimony wholesale and fail to "articulate in reasonable detail the basis" for its proposed findings. For example, the Staff states that "Dr. Cornell provided a persuasive and informed analysis" and "well-developed support for a finding that the PFSF seismic design, based upon the Applicant's PSHA with a 2,000-year return period ground motion, provides adequate protection of the public health and safety." Staff Findings ¶ 6.49. The Staff summarily states that "Dr. Cornell also responded to claims raised by the State" through his testimony and that "he bases his opinions" on "conservatisms . . . in . . . nuclear power plant design and acceptance criteria." Id. ¶ 6.47. Nowhere does the Staff describe why it finds Dr. Cornell's theory persuasive. Mere mention in a footnote that Dr. Cornell considered "risk reduction factors applicable to nuclear power plants" gives the reader no hint why the Staff may accept Dr. Cornell's theory. Id. at 192, n.102. It is impossible for the State to even surmise the basis for the Staff's proposed findings let alone address specific findings in this Reply. Therefore, the State finds no need to address Staff Findings with respect to Dr. Cornell's theories and now turns to PFS's Findings.

Contrary to the Staff's approach, the State and the Applicant both agree that the adequacy of the design basis earthquake must be adjudged in conjunction with the probability of failure of the SSCs and the level of conservatism in the design (or risk reduction level) – often referred to as the "two handed approach." *See* State Findings ¶ 495; PFS Findings ¶ 415. The State and Applicant also agree that in DOE Standard 1020, the

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did not accept the contrary evidence given "the contrary evidence was reasonable on its face and sponsored by well qualified witnesses." Id.

Department of Energy ("DOE") established acceptable risk-based seismic design standards. State Findings ¶ 498; PFS Findings ¶ 421. A key issue before the Board is the State's vehement disagreement that the Applicant has proffered sufficient evidence to support its findings that the proposed SSCs subject to a 2,000-year earthquake at the PFS site have adequate conservatism in the design (risk reduction factors).

In defending its requested exemption from the existing seismic standards, the Applicant in its Findings relies on various assurances from its witness, Dr. C. Allin Cornell, who in turn relies on (a) estimated risk reduction factors, not for ISFSIs, but for nuclear power plants; (b) opinions and assessments of the SSC designers and/or vendors; and (c) his faith and hope in NRC guidance. PFS Findings ¶ 425; *see also* Cornell Tstmy, Post Tr. 7856 at 24.

Notably Dr. Cornell's experience is severely limited as directly related to the seismic design of ISFSIs.[62] Not only is there an absence of performance or test data which demonstrate the seismic performance of unanchored casks subject to strong ground motions but also there is a failure by Dr. Cornell to quantify the probability of failure or the risk reduction ratio for the SSCs specific to the PFS facility. Significantly, the Applicant cannot even point to a single probabilistic risk assessment or fragility curve for any ISFSI that shows similar risk reduction ratios as claimed by Dr. Cornell. Furthermore, although he admittedly did not take the time to conduct a thorough and independent review of the various SSC

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[62]Dr. Cornell admitted his experience with ISFSI SRPs is limited to this case. Cornell Tstmy, Post Tr. 7856 at 22. Furthermore, other than this case, Dr. Cornell mentions he only became "generally familiar" with ISFSI technologies and issues in his contractual arrangements related to his support of NRC ISFSI seismic rulemaking. Id. at 6.

design calculations and analyses, the record is clear that Dr. Cornell relies on the contested technical analyses and opinions of the designers and vendors themselves.

1. Risk Reduction Factors Calculated for Nuclear Power Plants Are Not Applicable to the PFS Storage Pads and CTB.

Dr. Cornell surmises that the casks, storage pads, and CTB have risk reduction factors of "5 to 20 or greater." PFS Findings ¶ 425. The estimated range of risk reduction ratios for SSCs at PFS, claims Dr. Cornell, is based on the risk reduction ratios for nuclear power plants, the standard review plans applicable to ISFSIs, and the opinions and analyses performed by the vendors and designers of the PFS SSCs. Id.

Based on the hazard curve slope for PFS and performance values for nuclear power plant SSCs, Dr. Cornell estimated a range of risk reduction factors for SSCs at the PFS site. Cornell Tstmy, Post Tr. 7856, Attachment A at 4. However, Dr. Cornell did not employ performance values for PFS SSCs (*i.e.*, the unanchored casks, the storage pad, or the CTB). Thus, there is no quantitative calculation of risk reduction factors for the SSCs proposed at the PFS site.

According to Dr. Cornell, the performance values for nuclear power plant SSCs that he used to estimate risk reduction factors were based on "numerous engineering evaluations of safety margins and 'fragility curves' [for nuclear power plant] SSCs," "on seismic probabilistic risk assessments and seismic margins studies," and on "the actual behavior of [nuclear power plant] SSCs in earthquakes as observed in the field and tested in the lab." Id., Attachment A at 3. There is no similar experience or data with respect to SSCs at ISFSIs. In fact, Dr. Cornell testified that storage casks are not typical of SSCs at nuclear power plants.

Id. at 20; Cornell Rebuttal Tstmy, Post Tr. 12951 at 3. Consequently, risk reduction ratios for SSCs at nuclear power plants are not probative of the risk reduction ratio for the HI-STORM 100 casks at issue in this case.

In footnote 30, PFS claims that "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 . . ." PFS Findings ¶ 427, n.30. This statement is greatly misleading. In his testimony, Dr. Arabasz consistently suspended judgment as to whether Dr. Cornell was correct in concluding that risk reduction factors of 5 to 20 or greater existed for the SSCs at the proposed PFS site. This was because such conclusions were challenged by the State's engineering and dynamic experts, to whom Dr. Arabasz deferred, and he acquiesced to the correctness of Dr. Cornell's conclusions only in the context of hypothetical questions. Tr. (Arabasz) at 9129-33, 9298-9304, 10154, 10157-58.

In paragraph 436 of its Findings, the Applicant claims that risk reduction values of "5 to 20 or more" apply to foundation failure modes such as overturning, loss of bearing capacity and sliding. The Applicant based this finding on Dr. Cornell's "extensive knowledge of this area" and the "seismic margins studies" for nuclear power plants that considered those failure modes. *See* PFS Findings ¶ 436. The Applicant's conclusion is in stark contrast to Dr. Cornell's testimony. Yes, Dr. Cornell testified that the seismic probabilistic risk assessments and margin studies conducted for nuclear power plants, that he relies on, did consider foundation failure modes such as overturning, loss of bearing capacity, and sliding. Cornell Tstmy, Post Tr. 7856 at 24. Nevertheless, Dr. Cornell admitted he is "less familiar with the foundation [seismic probabilistic risk assessments]

results (id.) and he is unaware of the foundation details or the results (Tr. at 12952).

As PFS notes, Dr. Cornell testified that probabilistic risk analyses and seismic margin studies at nuclear power plants did not identify the foundation failure modes such as overturning, loss of bearing capacity and sliding, as critical failure conditions. PFS Findings ¶ 436. If the foundation failure modes were not considered critical failure conditions then it is not possible to conclude anything at all about risk reduction factors for foundation failure modes. Furthermore, foundation failure modes may in fact be more critical for the proposed PFS facility than for nuclear power plants because of the unanchored casks. Nevertheless, there is no evidence to support PFS findings that foundation failure modes have risk reduction ratios of "5 to 20 or more."

Importantly, Dr. Cornell also admitted that it is "not entirely clear" whether the risk reduction ratio values based on NUREG-6728 "[were] intended to apply to foundations." Cornell Tstmy, Post Tr. 7856 at 24. Furthermore, Dr. Cornell acknowledged that the nuclear power plant seismic probabilistic risk assessments and margin studies did in fact identify a "soil problem" at a nuclear power plant (the Midland Plant). Tr. (Cornell) at 12953.

Given that he openly questions whether estimated risk reduction ratios for nuclear power plants are applicable to foundations, it is clear that Dr. Cornell's premise that nuclear power plant foundations have "comparable" risk reduction ratios is merely based on his hope and expectation that NRC, through guidance, does in fact establish comparable conservatism for foundations. *See* Cornell Tstmy, Post Tr. 7856 at 24. Dr. Cornell simply "presumes" that NRC criteria provide conservatism in foundation design. Id.

Notwithstanding whether nuclear power plant risk reduction ratios can be applied to SSCs in this case, there is no evidence beyond Dr. Cornell's hope and expectation that nuclear power plant foundations have risk reduction factor of at least 5.

Contrary to the Applicant's theory, in the DOE paradigm risk reduction ratios have not been established for foundations. Risk reduction ratios generally are a function of the conservatism built into structural and mechanical codes and acceptance criteria (Cornell tstmy, post tr. 7856 at 16) which do not specifically apply to foundation modes of failure such as sliding and overturning. Specifically, DOE Standard 1020 recognizes that "specific acceptance" criteria have not been developed for foundation design and provides an alternative evaluation criterion for sliding and overturning failures modes that are probabilistically based. Tr. (Bartlett) at 12813, *quoting from* DOE-STD-1020-94 at 2-24 (PFS Exh. DDD).

Dr. Cornell calculated a risk reduction ratio using the slope of the seismic hazard curve at the PFS site, presumably, using factor of safety values of 1.25 to 1.5 cited in NUREG/CR-6728. Cornell Tstmy, Post Tr. 7856, Attachment A at 3-4. Notably, NUREG-0800, Section 3.8.5 specifies a factor of safety of 1.1 against overturning and sliding of foundations (Tr. (Bartlett) at 12819) and it is this minimum factor of safety that PFS has used in its design. Assuming *arguendo* that risk reduction ratios apply to foundations, there is no evidence that the risk reduction ratios calculated for nuclear power plants in NUREG/CR-6728, or by Dr. Cornell, are appropriate for the PFS foundations because the calculated risk reduction ratios used more conservative factors of safety (1.5 and 1.25) than the 1.1 minimum factor of safety used in the PFS design. The Applicant claims that the

actual margins in its foundation design exceed 10 percent. PFS Findings ¶ 437. However, as detailed in the State's Findings, the Applicant's claimed additional margins are unfounded and suspect. *See* State Findings ¶¶ 178-246; *see also* Part I, Overview, *supra*.. Also, in evaluating Dr. Cornell's position, the Board should note that the actual design margin for a factor of safety of 1.1 is much greater for nuclear power plant SSCs, which were designed to a larger earthquake (essentially a 10,000-year earthquake), than the design margin associated with the same factor of safety and a 2,000-year earthquake. Tr. (Bartlett) at 12835-40.

There is a paucity of evidence to support PFS's claimed risk reduction ratios for unanchored casks, storage pads, and CTB. No witness testified that any nuclear power plant foundation system was supported by a new application in the use of soil cement to resist earthquake forces. Nor could there be any such testimony because PFS's use of soil cement is precedent-setting. *See* Section III, Soil Cement, *supra*. Furthermore, no witness identified any nuclear power plant structure sited on relative soft clay foundation soils with a foundation that is shallowly embedded, as in this case. In fact, there is testimony that unlike the PFS foundation design, conventional nuclear power plant buildings located on soil sites and subject to severe ground motions are often 10 to 30 feet below ground. Tr. (Ostadan) 7673. As there is no evidence that suggests the risk reduction ratios obtained for nuclear power plants would apply to the storage pad and CTB foundations at the PFS site, the Applicant has not met its evidentiary burden of showing that risk reduction ratios for the casks, storage pads, and CTB in this case are equivalent to those estimated for nuclear power plant SSCs. In sum, there is no reliable evidence to establish values for the risk reduction ratios of the casks, storage pads, and CTB proposed for the PFS site.

2. PFS Failed to Quantify a Reliable Risk Reduction Factor for Storage Casks.

The Applicant concedes that its HI-STORM 100 casks are not typical SCCs found at nuclear power plants for which the nuclear power plant risk reduction ratios of at least 5 "have been demonstrated." PFS Findings ¶ 448. As such, to support adequate risk reduction ratios for the unproven performance of its unanchored casks under high ground motions at the PFS site, the Applicant relies solely on the results of nonlinear computer analyses which have not been validated by performance or test data. As previously discussed, the nonlinear computer analyses proffered by both the Applicant and Staff are not reliable.[63] *See* Section V, Seismic Nonlinear Analyses of Free Standing Cask Behavior, *supra; see also* State Findings ¶¶ 248-446. Accordingly, the Applicant has failed to demonstrate adequate conservatism in its storage cask design and that its requested lower design basis earthquake standard of 2,000 years will not endanger life or property and is in the public interest in accordance with 10 CFR § 72.7.

3. Cask Tip Over Analyses Unreliable.

Apparently realizing the shortcomings of its nonlinear computer analyses absent either validating performance or test data, the Applicant also resorts to arguing that whether a cask in fact tips over is of no consequence. PFS Findings ¶ 451. The Applicant claims its argument is supported by a non-mechanistic cask tip over analyses performed by the owners and benefactors of the Holtec HI-STORM 100 cask design. Id.

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[63]Rather than burden the Board with duplicative arguments, the State rests on its previous arguments concerning the unreliability of the nonliner computer models generated by the cask vendors and the Staff contractor, Dr. Luk. *See* State's Findings ¶¶ 377-444.

Assuming an initial angular velocity of zero when the cask is at a point of incipient tipover, Holtec calculated the tip over deceleration as 43.82 g as the cask strike the pad. Tr. (Tseng) at 5609, *reading from* Holtec's PFSF Site-Specific HI-STORM Drop/Tipover Analyses. The design basis deceleration limit is slightly higher at 45 g. PFS Findings ¶¶ 451, 535, 537. State expert Dr. Bartlett opined that if PFS's sole reliance on computer simulations are wrong and the casks in fact tipped over during a seismic event, the initial angular velocity of the cask at incipient tipover would clearly not be zero due to rocking of the cask before tipover. Tr. (Bartlett) at 12870-71. Given this premise, Dr. Bartlett doubted the accuracy of Holtec's tip over analysis calculations; he questioned calculations that concluded no breach of canister even in the of tip over given the small design deceleration design margin calculated by the cask vendors and their unrealistic assumption of an initial angular velocity of zero. Id. In a footnote, the Applicant attacks Dr. Bartlett's expertise claiming his "opinion is beyond his area of expertise." PFS Findings at 283, n.37. As Dr. Bartlett explained during cross examination, the fact that the initial angular velocity of a tipping cask is not zero due to rocking is a simple dynamic concept and well within his earthquake engineering expertise. Tr. (Bartlett) at 12913-15.

Moreover, the Applicant ignored its own witness, Dr. Cornell, who testified consistent with Dr. Bartlett in this matter, that the initial angular velocity of a cask which tips over would "probably clearly" not be zero. Tr. (Cornell) at 7978. Rather than heed the testimony of its own witness, Dr. Cornell, in this instance, PFS finds it convenient to rely on the cask vendor who has a substantial personal stake in the favorable outcome of this proceeding. In the presence of reliable contradicting testimony, the Board should not be

swayed by testimony that substantially advances the witnesses' personal interests and also defies engineering logic.

The Applicant relied on the opinions of the cask vendors who claim the deceleration at cask impact would be less than that calculated assuming an initial velocity of zero because of cask precession. PFS Findings at 283, n.37. Cask precessing is a helical motion that the cask vendors maintain could prevent a cask from tipping over if the cask tipped at 29.7 degrees (the calculated point from vertical where the location of the cask center of gravity would cause the cask to tip over). Tr. (Singh, Soler) at 6108-6110. Notably, the Applicant fails to reference Dr. Singh's admission to the Board that, regardless of cask precessing any residual motion could in fact tip the cask over if the cask was tipped to 29.7 degrees. Id. at 6110.

The Applicant's witnesses testified that in evaluating the tip over design margins, the design basis tip over deceleration is 45 g for the HI-STORM 100 cask. Tr. (Singh) at 5861, 12158. The Applicant would now have the Board rely on its claimed deceleration limit for fuel cladding as well as its cask drop limit. PFS Findings ¶ 451. Significantly, PFS witnesses give passing mention to the fuel cladding deceleration limit (Tr. (Singh) at 12110, 12158) but no witness testified with respect to the Applicant's theory raised in its Findings that the fuel cladding deceleration limit supports its exemption. Moreover, the Applicant has not proffered any supporting documentation or calculations for either the claimed deceleration limit for fuel cladding or the cask drop limit. In raising the fuel cladding limit, the Applicant is asking the Board to strip away yet another layer of protection to the public, including future PFS on-site employees, in an effort to justify its seismic design exemption request. In

order to rely on the Applicant's claims that the canister would not be breached during tip over, the Board would have to (1) find that based on unproven and suspect nonlinear computer projections the HI-STORM 100 cask would not tip over; (2) agree that the initial angular velocity just prior to tipping is in fact zero even if the cask does tip over due to seismic forces; (3) disregard Staff testimony that the acceptance criteria for dry cask storage is that the cask would not tip over;[64] and (4) accept that if the unvalidated computer projections prove to be wrong, it is sufficient to rely on the last layer of protection, the fuel cladding deceleration value. Importantly, there is no evidence that demonstrates whether the fuel cladding deceleration limit will be exceeded if cask tip over occurs. In assuring the safety of the public, including PFS employees, the State maintains that the Board would find itself teetering on the edge of the regulatory cliff if it relies on the last layer of protection or on the cask vendor's non-mechanistic tip over analyses. There is no margin of safety in such an approach.

In its Findings, the Applicant claims given Dr. Bartlett's acknowledgment that it is not necessary to develop fragility curves for the PFS SSCs to determine whether a performance goal was met that the "need for fragility curves is no longer an issue." PFS Findings ¶ 452. Dr. Bartlett has consistently testified that fragility curves are a desirable

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[64]The Applicant cites Staff witness, Mr. Guttmann's opinion that even if the computer analyses are erroneous and the casks do tip over there will be no significant adverse consequence. PFS Findings ¶ 453 (*citing* Tr. (Guttman) at 7062-64). First, if casks do in fact tip over, then Holtec's analyses are also erroneous. Second, Mr. Guttmann's opinion on consequences should bear no weight in the Board's decision given Mr. Guttmann's admitted lack of expertise in the structural analyses of casks. Tr. (Guttmann) at 6917.

mechanism to determine the performance capability of SSCs but not the only method to determine whether a performance goal has been met. Tr. (Bartlett) at 12852-53; 12874-75. Given that the Applicant has requested a substantial reduction in the seismic design standard, while not unconditionally necessary, it would be reasonable for the Board, in the absence of other probative evidence, to require fragility curves so that any design margins may be compared between the requested seismic design standard and ground motions for other earthquakes, such as a 10,000-year earthquake.

The evidence does not support the Applicant's conclusion that risk reduction factors "are on the order of 5 or more," and "therefore a performance goal of $10^{-4}$ . . . has been achieved." PFS Findings ¶ 453. Looking at the two handed approach, PFS's unconventional design and slim, if any, margins of safety require the design (capacity) side to do all the heavy lifting. PFS has not demonstrated that its risk reduction ratios alone are sufficient to discard the current regulatory scheme, which places a substantial demand (deterministic earthquake) on the seismic design. The Applicant has failed to show that its seismic design would provide adequate conservatism at its requested substantial reduction in the design basis earthquake standard. Based on the foregoing and the proposed State Findings ¶¶ 248-444, the State submits that the Applicant has not met its burden that the requested exemption from the existing design basis earthquake standard is adequate to assure public safety and is in the public interest.

### C. Radiation Dose Analysis

In their proposed findings, PFS and the Staff claim that the issuance of an exemption is justified on the ground that the State has not demonstrated that the dose limits

of 10 C.F.R. § 72.106(b) would be exceeded in a cask tipover accident. This argument fails in three respects. First, PFS and the State oversimplify the question of what is the applicable standard. As discussed below and in the State Findings ¶¶ 553-54, it is possible that 10 C.F.R. § 72.104(a) is applicable to a cask tipover scenario.

Second, PFS and the Staff misapply 10 C.F.R. § 72.106(b) by attempting to hybridize it with portions of 10 C.F.R. § 72.104(a) that yield a more favorable result for PFS. In their Findings, they seek to apply § 72.106(b)'s dose limit of 5 rem for the duration of the accident, with the "real individual" specified in § 72.104(a). This legal error contributes significantly to PFS's and the Staff's gross underestimate of radiation doses from a seismic event.

Third, PFS and the Staff try to shift the factual burden of proof to the State, by arguing that the State has not demonstrated that dose limits would be exceeded in a cask tipover accident. The State has never claimed to perform the type of sophisticated quantitative dose analysis that is necessary to prove that dose limits will be exceeded in a seismic event. That is not the State's burden. The State had the burden of going forward with enough evidence to demonstrate that PFS had failed to meet its burden of proof. The State met its burden, by performing a dose analysis that was sufficiently detailed to illustrate the significance of the deficiencies in PFS's own analysis. As demonstrated by the State, PFS's analysis was fatally deficient in the following key respects: (a) PFS relied on a qualitative analysis that roughly extrapolated normal doses to accident conditions, without making any attempt to quantitatively calculate accident doses; (b) without any factual basis, PFS assumed that the accident would not last more than 30 days; and (c) PFS used a

nonconservative assumption about the configuration of tipped-over casks. State Findings ¶¶ 555-579. The significance of these deficiencies is illustrated by the NRC Staff's own analysis. Just by correcting for the third factor and assuming a cask tipover accident in which the bottoms of casks are facing outward, the Staff found that the dose would increase by almost two orders of magnitude. Staff Findings ¶ 6.216.

1. Applicable Standard

PFS claims "all parties agree that 10 C.F.R. § 72.106(b) governs the radiation dose limits for an accident involving a cask tipover during a seismic event." PFS Findings at 93; *see also* ¶ 499. This statement is an oversimplification of the State's position. The State generally agrees that § 72.106(b) applies to an analysis of accident doses. In reality, however, PFS does not advocate the full application of § 72.106; instead, it would substitute a portion of § 72.104(a) to obtain a lower dose estimate than would otherwise be yielded. Moreover, there is substantial disagreement among the parties as to what constitutes an accident, thus leading to some confusion as to whether § 72.106(b) or § 72.104(a) applies. As Dr. Resnikoff testified, an accident should be presumed to continue until the facility has been restored to the condition for which it was licensed. PFS seems to believe that an accident ends when temporary measures are taken to reduce offsite doses, such as the erection of barriers or the evacuation of nearby residents.

The State does not believe that PFS's interpretation of the law yields a rational result. However, assuming for purposes of argument that PFS's approach is correct, and "normal" operation can be considered to resume after such contingency measures are taken, then the lower dose limit of 25 millirem per year to a real individual must be applied to the cask

tipover scenario. PFS has not undertaken such an analysis at all.

2. PFS Incorrectly Applies 10 CFR § 72.106(b)

PFS claims that the duration of an accident was "newly-raised" by the State during the hearing. PFS Findings ¶ 501. In making this argument, PFS tries to divert the Board's attention from the fact that PFS, which bears the burden of proof in this proceeding, completely failed to address the issue of the duration of the accident in its own written direct testimony. PFS has consistently argued that 10 C.F.R. § 72.106(b) is the applicable standard for the dose analysis that must be performed to justify a regulatory exemption. *See* PFS Findings ¶ 499. Further, the Staff has acknowledged that determining the duration of an accident is a necessary step in demonstrating compliance with the standard. Tr. (Waters) at 1226. Thus, PFS should have addressed the issue in its own initial testimony. Instead, PFS addressed the question of accident doses in terms of a "dose rate," *i.e.*, the amount of radioactivity that would be received in a year by a person who is at the site boundary for 2,000 hours per year. PFS confirmed in the hearing that it had not made an estimate of the duration of an accident. Tr. (Redmond) at 12093. Singh/Soler/Redmond Tstmy, Post Tr. 12044 at 7-8. The State appropriately used its opportunity for rebuttal testimony to address this omission from PFS's testimony. Dr. Resnikoff testified that an accident would last until the facility is restored to its original condition. Tr. (Resnikoff) 12506-08. If casks are damaged, this accident period could last for 20, 30, or 40 years. Id. at 12508. PFS, by its own admission, had performed no analysis that would show otherwise.

PFS also argues that in any event, the duration of an accident is moot because (a) there would be no tipover of a cask under any earthquake scenario, and (b) even if a cask

were to tip over, the dose limit of 5 rem in 10 C.F.R. § 72.106(b) "would never be reached." PFS Findings at 93. With respect to factor (a), PFS has not met its burden of demonstrating that a cask will not tip over in an earthquake. The inadequacy of PFS's tipover analysis is discussed at length in Section V, Seismic Nonlinear Analyses of Free Standing Cask Behavior, *supra*. Given the poor analysis performed by PFS, this prediction amounts to nothing more than wishful thinking.

With respect to factor (b), the evidence shows that even using a rudimentary analysis, doses from an array of tipped-over casks will be much closer to the dose limit than predicted by PFS. According to PFS, the margin between the expected dose rate at the owner controlled area ("OCA") boundary and the 5 rem dose limit is approximately three orders of magnitude. PFS Findings ¶ 510. In contrast, Dr. Resnikoff made a rough calculation of 150 mrem/year. State Findings ¶ 550. His analysis did not include several components that would increase the dose, such as neutron collisions producing gamma rays and ground reflection of gamma rays. If an accident lasted for 20 years, the dose would be 3 rems. The NRC Staff also estimated that in the worst case, the normal off-site dose rates could increase by a factor of 97.6. NRC Staff Findings ¶ 6.218. That is almost two orders of magnitude. If one applies this factor to PFS's normal dose calculation and also corrects for PFS's error in assuming a "real" person at the fence post who is exposed for 2,000 hours rather than "any individual" who is exposed for 8,760 hours, then the accident dose at the fencepost would be 2.5 rem.[65] If the accident lasted for two years, the dose limit in 10 C.F.R. § 72.106(b)

[65]The equation is: 97.6 x 5.85 x 8760/2000.

would be exceeded. Thus, the evidentiary record contradicts PFS's assertion that accident doses would be orders of magnitude less than the dose limit in § 72.106(b).

Unfortunately, PFS's presumption that casks will not tip over fatally tainted the quality of the tipover analysis that it did perform. Despite the ready availability of the Monte Carlo computer program for calculating doses in a tipover scenario involving multiple casks, and PFS's witness' ability to model such a scenario, PFS failed to perform any such analysis. Instead, it took the results of the quantitative analysis that it performed for **normal** operations and extrapolated them to the tipover of a **single** cask. Singh/Soler/Redmond Post Tr. 12044 at 10-11. From this qualitative extrapolation, PFS extrapolated still further that a multiple cask tipover would be similar to the dose if all the casks were standing upright. Id. When asked why PFS failed to perform a quantitative accident analysis, PFS's expert merely said he did not need to because it was a "hypothetical condition." Tr. (Redmond) at 12123. This dismissive attitude is also reflected by PFS's characterization of a cask tipover during a seismic event as a "beyond-design-basis accident." PFS Findings ¶ 498. The characterization of a cask tipover as "beyond-design-basis" is completely inapt, because it carries the inference that the analysis at issue is a severe accident analysis that is not directly related to the design of the facility. PFS completely misses the point that the purpose of the analysis under consideration is to determine what the design basis should be.

As a result of its "it won't happen here" attitude, PFS performed only a cursory and qualitative analysis of accident doses. In performing its qualitative analysis, PFS failed to account for a significant distinction between a tipped-over and an upright cask: because it is not completely shielded, the bottom of a tipped-over cask emanates much more radioactivity

than the sides of an upright cask. Resnikoff Tstmny, Post Tr. 12349 at 9-10.

PFS also failed to account for the effects of flattening or thinning of the concrete caused by the impact of falling casks on one another. PFS claims that "[n]o State witness has provided testimony" that a cask impact from uplift, sliding and collision, or tipover of a cask, would cause flattening or thinning of the concrete or stainless steel in a cask. PFS Findings at 94; ¶ 527. PFS also attacks Dr. Resnikoff's qualifications to testify regarding these effects. PFS Findings ¶ 526. However, as Dr. Resnikoff testified, it is a matter of common sense that it is possible for casks to fall on each other while the earth is going up and down. Tr. (Resnikoff) at 12613. Certainly, he has observed this phenomenon in train wrecks. Id. By attacking Dr. Resnikoff, PFS attempts to avoid the more important issue, which is that PFS has not done any quantitative analysis of flattening or other physical effects of falling casks during a seismic event.

3. Occupancy Time

As discussed in the State Findings ¶ 559, the occupancy time at the fencepost of the exposed individual is determined by the language of the applicable regulation. If the regulation refers to "any real individual," then it is acceptable to take into account how often a person is likely to be at the fencepost. However, if the regulation refers to "any individual," it must be assumed that the individual is at the fencepost 24 hours a day and 365 days a year.

PFS claims that 10 C.F.R. § 72.106(b) applies, rather than § 72.104(a). PFS Findings ¶ 498. Section 72.106(b) limits the radiation dose to "any individual," while § 72.104(a) limits the dose to "any real individual." Instead of consistently applying § 72.106(b) in its

analysis, however, PFS actually applies a made-up hybrid of § 72.106(b) and § 72.104(a). *See* PFS Findings ¶ 516. PFS applies the 5 rem limit in § 72.106(b), but that is all PFS takes from § 72.106(b). Instead of applying the 5 rem limit to "any individual," as prescribed by § 72.106(b), PFS applies the limit to a "real individual," as per § 72.104(a). And instead of applying the dose over the duration of the accident, as § 72.106(b) requires, PFS assumes that the 5 rem limit is an annual exposure limit.

PFS cannot have it both ways. Either § 72.106(b) must be applied in its entirety, or § 72.104(a) must be applied in its entirety.

PFS argues that an occupancy time of 2,000 hours would be "conservatively high" for accident conditions. PFS Findings ¶ 519. However, the regulations do not permit PFS to make an independent circumstantial estimate of occupancy times at the site boundary. Section 72.106(b) requires PFS to assume that an individual is at the site boundary for 8,760 hours.

Moreover, whether or not PFS's assumption of an occupancy time of 2,000 hours is "conservative" for purposes of a dose analysis under § 72.104(a), *see* PFS Findings at 98, the alleged conservatism is irrelevant for purposes of a § 72.106(b) analysis. Conservatism is built into § 72.106(b) by the use of the term "any person" to describe the exposed individual. By providing that the person at the fence post is "any person," rather than a "real person," § 72.106(b) necessarily assumes a person who is at the fencepost all the time, *i.e.*, 24 hours a day for 365 days a year, or 8,760 hours altogether.

4. Duration of Accident

The duration of the accident is a key consideration in evaluating compliance with 10

C.F.R. § 72.106(b). As PFS correctly notes, 10 C.F.R. § 72.106(b) does not define an accident duration, "and no regulatory guidance is directly on point." PFS Findings at 93. Yet, PFS made no attempt to assess the duration of a tipover accident in performing its dose analysis. In contrast, Dr. Resnikoff's dose estimate assumed that an accident would last a year, or 8,760 hours. Resnikoff Tstmny, Post Tr. 12349 at 6. Dr. Resnikoff also pointed out that the duration of an accident is something that must be determined in each individual case. Tr. (Resnikoff) at 12600.

PFS also asserts that "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." PFS Findings ¶ 519; *see also* ¶ 551. As discussed in the State's Findings, however, PFS is not entitled to rely on contingency measures in evaluating its satisfaction of 10 C.F.R. § 72.106(b). Moreover, the contingency measures described by PFS are completely unplanned. Thus, there is no assurance that they will be carried out. *See* State Findings ¶¶ 569-572.

The length of time that casks are in a tipped-over state may also affect the integrity of the concrete for purposes of blocking neutron radiation. If air circulation is suppressed, the casks may heat up and water may evaporate from the concrete, thus inhibiting the ability of the concrete to absorb neutron radiation. State Findings ¶ 562. PFS argues that it is "impossible for all the air inlet ducts to be blocked, and even in a tipped-over condition heat transfer through the ducts and heat radiation and conduction would occur such that one would not expect the short-term limit to be exceeded." PFS Findings at 99; ¶ 521. But this argument misses the State's point, which is that PFS has not analyzed the problem at all.

Having failed to do any quantitative analysis of the problem, PFS falls back on pure speculation. PFS's assertions that concrete temperatures will remain below 600 degrees F (id. ¶ 522) is entirely unsupported by any calculations, as is PFS's assertion that it would be months before water is lost from concrete. Id. ¶ 543.

5. Orientation and Deformation of Casks

PFS claims that Holtec evaluated the potential effect of multiple cask tipovers and found that "no significant adverse consequences were likely to occur due to the random orientation of the casks and the localized damage to storage casks." PFS Findings at 100; *see also* ¶¶ 506, 508-509. But PFS did not do a quantitative analysis of any of the relevant aspects: orientation of the casks, radiation doses from the casks, or flattening or stretching of casks. PFS simply has no basis for these assertions.

According to PFS, a local deformation of a cask "would not significantly affect the shielding performance of the storage cask, since the same mass of steel and concrete would still be present." PFS Findings ¶507. PFS also argues that "the local deformations would occur at the top of the storage cask, where the radiation doses are greater at the middle of the cask." Id. Therefore, according to PFS, "any increase in the radiological dose levels due to localized deformation of the cask would at most be minimal." Id. As Dr. Resnikoff testified, however, if concrete is thinned in the area where fuel is located, the dose rate will increase. If concrete is pushed upward, to an area where there is no fuel, shielding in that area is essentially wasted. Tr. (Resnikoff) at 12479-80.

6. Dose Calculations

PFS argues that Dr. Resnikoff prepared "worst case" dose calculations that have "so

many errors that these calculations cannot be given any weight." PFS Findings ¶ 540. Once again, PFS tries to shift the burden of proof to the State. Dr. Resnikoff never claimed to do the type of dose calculations that are necessary to *prove* the size of the radiation doses in a cask tipover accident. That was PFS's burden. What Dr. Resnikoff provided was a set of rough calculations, providing the parameters that must be examined through the use of a quantitative method such as the Monte Carlo model. If a mirror is held up to each of the criticisms leveled by PFS at Dr. Resnikoff, it can be seen that PFS has not made any attempt to consider the variable. For instance, PFS's dose estimate does not consider hydrogen loss through the heatup of the concrete casks. PFS Findings ¶ 542. PFS complains that Dr. Resnikoff uses inflated neutron dose figures (id. ¶ 543), but in fact Dr. Resnikoff presents a range of dose rates as a function of hydrogen loss. In any case, the dose at the fence post calculated by Dr. Resnikoff did not take into account degradation of concrete. He stated that exposures would be a factor in the duration of the recovery period from an accident. PFS also failed to consider the production of gammas from neutrons. Tr. (Resnikoff) at 12639[66]; *see also* 12493-95. PFS and the Staff both criticize Dr. Resnikoff's calculations of radiation doses from the bottom of overturned casks, but PFS itself made no attempt to

[66]The NRC Staff argues that Dr. Resnikoff's understanding of the HI-STORM 100 cask design was "significantly flawed." Staff Findings ¶ 6.155. However, the error that the Staff points to, regarding the thickness of the pedestal base plate, was inconsequential. The Staff is correct that the MPC base plate is 2 1/2" thick, the overpack base plate is 2" thick, and the pedestal steel plate is 5" thick. But the MPC base plate and overpack base plate thicknesses were taken into account by Dr. Resnikoff in State Exh. 141a, the correction to State Exh. 141. Dr. Resnikoff essentially assumed the pedestal base plate was of infinite thickness, that is, no radiation came through the pedestal base plate. The only radiation from the bottom of the cask was in a ring, not shielded by the pedestal base plate. So this was not an error on Dr. Resnikoff's part, just an underestimate of how much radiation was emanating from the bottom of the HI-STORM cask.

calculate these doses. As discussed above, the Staff's own calculations show that doses from overturned casks would be much higher than the doses estimated by PFS.

As stated above, Dr. Resnikoff did not include important components of the dose, such as ground reflection of gamma rays and neutron production of gamma rays. Only by a Monte Carlo analysis can these components be captured. Tr. (Resnikoff) at 12495. While it is true that Co-60 declines with a 5.27 year half-life, the neutron emitters and Cs-137 are long-lived. Tr. (Resnikoff) at 12619-20, 12625.

7. Alleged Conservatisms

PFS claims that "many conservatisms" were included in PFS's "calculations" of the 5.85 mrem/year dose at the OCA boundary. PFS Findings ¶ 511. However, PFS has performed no "calculations" of the radiation dose at the OCA boundary due to a tipover accident. Thus, the alleged conservatisms are not attached to any calculation of accident doses for the PFS facility. Nor, for the most part, has PFS quantified the three conservatisms it claims. PFS gives a value for only alleged conservatism, *i.e.*, its assumption that all 4,000 casks contain fuel with a burnup of 40,000 MWD/MTU and a cooling time of 10 years. PFS Findings ¶ 511. According to PFS, more realistic assumptions would have decreased the dose estimate by fifty percent. Id. For the other two alleged conservatisms – the assumption that fuel assemblies inside the casks have the highest gamma and neutron radiation source term in all fuel storage locations, and the assumption that the fuel has been subject to a single irradiation cycle – PFS has not provided a quantification of the value of the conservatism.

Thus, neither PFS's dose estimates nor its three claimed conservatisms provide the

necessary level of confidence that the accident dose in a cask tipover will be below regulatory limits. PFS must perform a dose calculation, and it must also quantify any alleged conservatisms on which it relies.[67]

PFS's assumption regarding the orientation of fallen casks was also nonconservative. PFS assumed that some casks would fall on others and that not all cask bottoms would face the fence. PFS also opined that the dose from the cask bottoms and horizontal casks would likely be less than the dose to stand-up casks, but this analysis was entirely qualitative. However, as Dr. Resnikoff testified, it was conservative to estimate that the casks fall with their bottoms facing the fence. State Findings ¶ 576. PFS calls this a "worst-case" analysis, rather than recognizing it for what it is: a conservatism. PFS Findings 540.

D. **Annual vs. Lifetime Risk.**

In its Findings, PFS argues that "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." PFS Findings ¶ 490. The State disagrees with this point of view. *See* State Findings ¶ 585. The expected lifetime of the proposed facility is relevant because, for the purpose of determining whether or not the level of risk to the environment or to society as a whole is acceptable, the most relevant probability to

[67]PFS also argues that its assumption that the exposed person is at the fencepost 2,000 hours a year is "conservative." PFS Findings at 98; ¶¶ 518, 519. As discussed above, however, there is nothing conservative at all about PFS's assumption. PFS should assume the exposed person is at the fencepost 8,760 hours a year.

consider is the probability of failure during the expected lifetime of the facility. Id.; Arabasz Tstmy, Post Tr. 9098 at 16.

As discussed by PFS, a probability of exceedence, such as a probability per 50 year period, can be converted mathematically to a probability of occurrence in any arbitrary time period including on an annual basis. *See* PFS Findings ¶ 491; *see also* Arabasz Tstmy, Post Tr. 9098 at 15 (annual probability of exceedance of $2 \times 10^{-3}$ equates to a design seismic hazard level with a 10 percent probability of exceedance in 50 years). However, the fact that probability of exceedence values based on different time periods can be expressed as an annual probability of occurrence does not in itself imply that the original time period for which the failure probability is evaluated is immaterial.

Among other arguments, PFS attempts to justify its point of view by the following analogies:

> [U]nder 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.
>
> . . .
>
> [U]nder 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.

PFS Findings ¶ 490. These analogies are specious because they both involve the notion of regulatory standards which, in practice, would be non-uniform and consequently unfair. In the case of apartment buildings, unlike an ISFSI, the length of time that the building will be used is generally indeterminate and unregulated by law. In the case of workplace safety

standards, the ages and employment durations of adult workers are also generally indeterminate and unregulated. Therefore, as a practical matter, the safety standards have to be based on probabilities computed using the average lifetime of buildings and the average (or maximum) duration of employment.

The issue at hand in this proceeding is a request from PFS for an exemption from the applicable regulatory requirements for a facility that would be unique in terms of its size, design, and storage capability. In this case, it is quite appropriate that factors specific to the proposed facility, including the expected lifetime, be taken into account. Perhaps a better analogy than the ones put forth by PFS is the determination of rates for automobile insurance, which is traditionally based on many factors specific to the insurance policy. The rates for automobile insurance increase with the estimated number of miles per year that the vehicle is driven because of the increased fraction of time that the vehicle is exposed to traffic-related hazards. Thus, the insurance rates for specific vehicles are based on risk estimates which explicitly take into account the anticipated exposure time for the hazard.

The Board is confronted with a difficult task in determining the appropriate design basis earthquake that the unprecedented PFS design must meet. It should not limit the available tools, such as lifetime risk, that it may use in reaching its decision.

## E. Public Interest

The Commission may grant an exemption from the requirements of duly promulgated regulations if it determines, *inter alia*, that the exemption is in the public interest. 10 CFR § 72.7. It is telling that neither the Staff nor PFS address the public interest in their Findings. Consequently, a response from either party that describes the public interest in

granting PFS an exemption will merely be manufactured to address the State's Findings. The record is clear: PFS's request for an exemption is because the regulatory required deterministic design values exceed the proposed design values in PFS's Safety Analysis Report. Con-SER at 2-34; *see also* State Findings at 10-13, ¶¶ 586-88. The Board should eschew any notion that the public interest under the exemption standard is akin to a NEPA analysis wherein PFS may argue the need for the facility. The issue here is whether it is in the public interest to allow PFS to design to a lower earthquake design value than required by the law. The answer from a State in which its citizens have suffered the effects from federal actions involving nuclear fallout is categorically, no.

CONCLUSION

The State has presented to the Board, through its well-qualified and experienced experts, significant and genuine concerns that PFS's seismic design is vulnerable to instability under high earthquake accelerations predicted at the PFS site. Two known capable faults (magnitude 6.4 and 6.5) dip under the site and the Stansbury fault, about 5 miles away, has been storing up energy for the past 8,000 years and is capable of delivering a large earthquake (magnitude 7.0) but it is unknown whether the next earthquake will be tomorrow or in thousands of years. PFS's seismic design must have sufficient capacity to meet the demands of the potential energy that may be unleashed on the PFS site. When looking at the capacity of the system to respond to an earthquake, there are extreme sensitivities in the dynamic analyses of the casks, foundations and soils that require experts in a very narrow and specialized field of earthquake engineering to conduct supportable analyses. Uncertainty abounds in the record before the Board. Starting with PFS's failure to collect

adequate soil samples to test soil strength and to conduct an adequate evaluation of foundation loadings, continuing through PFS's deferral of soil cement testing, and culminating in PFS's total reliance on Holtec's computer model (not validated with any test data) to predict cask movement, PFS and the Staff would have the Board rely on uncertainty, upon uncertainty, upon uncertainty. And they would have the Board do this by relying, in many instances, on witnesses who are not qualified or competent to conduct analyses in soil structure interaction, foundation loading, and cask modeling.

The Skull Valley site, directly below an active fault, is not a choice site on which to store 4,000 casks of high level nuclear waste but it is the only site available to PFS. Given the earthquake potential of the site, it is incumbent on PFS to present an adequate and supportable design that can reasonably assure the health and safety of Utah citizens. Yet, PFS has not done so. Instead PFS requests to lower the seismic design standard, refuses to validate its design claims with any test data, such as shake table tests, or to change its design to by-pass the soft Bonneville clays. Moreover, to address the question of uncertainties it its design, PFS merely claims its design analyses are conservative, and attempts a back end justification by pointing to other assumptions or analyses, which are themselves laden with uncertainties.

PFS's case, as support by the Staff, generally defies accepted engineering concepts. There are no conservatisms in PFS analyses and design as claimed by PFS. Rather PFS has presented a risky design that the Board should soundly reject.

DATED this 16th day of October, 2002.

Respectfully submitted,

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CERTIFICATE OF SERVICE

I hereby certify that a copy of STATE OF UTAH'S REPLY TO PROPOSED FINDINGS OF FACT AND CONCLUSIONS OF LAW OF THE APPLICANT AND NRC STAFF ON UNIFIED CONTENTION UTAH L/QQ was served on the persons listed below by electronic mail (unless otherwise noted) with conforming copies by United States mail first class, this 16th day of October, 2002:

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