RAS 4836

DOCKETED
USNRC

September 10, 2002 (3:23PM)

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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

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

APPLICANT'S PROPOSED FINDINGS OF FACT AND CONCLUSIONS OF LAW ON UNIFIED CONSOLIDATED CONTENTION UTAH L/QQ (SEISMIC)

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Dated: September 5, 2002

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Template = SECY-057

SECY-02

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Pursuant to 10 C.F.R. § 2.754 and the Order of the Atomic Safety and Licensing Board ("Licensing Board" or "Board") dated August 21, 2002, Applicant Private Fuel Storage L.L.C. ("Applicant" or "PFS") submits in the form of a partial initial decision its proposed findings of fact and conclusions of law concerning Unified Consolidated Contention Utah L/QQ ("Contention Utah L/QQ"). The proposed partial initial decision is organized into four sections. Section I, Introduction and Background, presents the history of the seismic contention in its various forms to date and introduces the witnesses who provided testimony on the various issues. Section II, Overview of the Seismic Issues, describes the issues that comprise Contention Utah L/QQ and provides a narrative discussion of how the issues interrelate, what evidence has been presented with respect to each issue, and what the evidence shows. Section III, Findings of Fact, presents Applicant's proposed findings of fact on each section of Contention L/QQ, in sequentially numbered paragraphs. Section IV, Conclusions of Law, presents Applicant's proposed conclusions of law on each section of the contention, also in sequentially numbered paragraphs.

I. INTRODUCTION AND BACKGROUND

A. Background – Unified Contention L/QQ

1. Utah Contention L

Utah Contention L ("Utah L"), Geotechnical, was submitted by the State (in a supplemental petition filed in November 1997). The contention stated as follows (footnote omitted):

> The Applicant has not demonstrated the suitability of the proposed ISFSI site because the License Application and [Safety Analysis Report] do not adequately address site and subsurface investigations necessary to determine geologic conditions, potential seismicity, ground motion, soil stability and foundation loading.

The State submitted four bases to support Utah L. "State of Utah's Contentions on the Construction and Operating License Application . . .", pp. 80-95 (November 23, 1997). The four bases raised concerns about : (1) surface faulting; (2) ground motion; (3) characterization of subsurface soils, including (a) subsurface investigations, (b) sampling and analysis, and (c) physical property testing for engineering analysis; and (4) soil stability and foundation loading. [1]

Utah L was admitted into the proceeding in March (?) 1998. LBP-98-7, 47 NRC 142, 253, reconsideration granted in part and denied in part on other grounds, LPB-98-10, 47 NRC 288, aff'd on other grounds, CLI-98-13, 48 NRC 26 (1998). Utah L, as admitted, raised issues concerning the adequacy of PFS's efforts to identify and characterize faulting in the site vicinity (Basis 1), the alleged failure by PFS to account for spatial variations in ground motion amplitude and duration because of near surface traces of potentially capable faults (Basis 2), the characterization of subsurface soils (Basis 3), and

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[1] See State of Utah's Contentions on the Construction and Operating License Application by Private Fuel Storage, LLC for an Independent Spent Fuel Storage Facility, pp. 80-95 (November 23, 1997) ("State's Contentions").

soil stability (Basis 4). The substance of these claims is not affected by the determination whether the design-basis earthquake should be calculated using a deterministic or probabilistic methodology. Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Installation), LBP-99-21, 49 NRC 431, 436 (1999).

On April 2, 1999, PFS submitted an exemption request, pursuant to 10 C.F.R. § 72.7, which sought NRC Staff approval for using a probabilistic seismic hazard analysis ("PSHA") methodology based on a 1,000-year return period earthquake instead of the deterministic methodology otherwise required by 10 C.F.R. Part 72.[2] PFS Exh. 247. Currently, 10 C.F.R. Part 72 provides for an ISFSI applicant to perform its seismic analyses using a deterministic approach for characterizing earthquake motion. 10 C.F.R. § 72.102(c) This is the same analytical approach that was previously required in licensing nuclear power plants prior to the amendment of 10 C.F.R. Part 100, pursuant to which new nuclear power plants may use a probabilistic analysis methodology. See 61 Fed. Reg. 65, 176 (1996).

On August 24, 1999, PFS modified its exemption request to reflect a probabilistic analysis based on a 2,000-year return period earthquake, as a result of comments received from the Staff. PFS Exh. 248. Shortly thereafter, PFS revised its License Application to use a 2,000-year return period earthquake as the design basis earthquake.

On December 15, 1999, the NRC Staff issued a Safety Evaluation Report ("SER") for the Private Fuel Storage Facility ("PFSF"), in which it indicated that the use of a PSHA and a 2,000-year return period earthquake would be an acceptable methodology.

2 Letter from John Parkyn, PFS, to Mark Delligatti, NRC, dated April 2, 1999. The License Application was amended on May 19, 1999 to change the design basis earthquake to the 1,000-year return period earthquake. See SAR at 2.6-38 [Rev. 3].

On September 29, 2000, the Staff issued an updated SER in which it formally approved Applicant's exemption request.

The State filed a Request to modify basis 2 of Utah L on November 9, 2000.[3] In its Request, the State sought to require either the use of a PSHA with a return period of 10,000 years for the design basis for earthquake ground motions, or the use of a deterministic seismic hazards analysis ("DSHA"). As an alternative to these two approaches, the State sought to require use of a PSHA with an unspecified return period "significantly greater than 2,000 years to avoid placing undue risk on public safety and the environment." State Request at 5.

After discovery was completed on Utah L, PFS filed its summary disposition motion on December 30, 2000. PFS argued that no genuine issue of material fact existed and discussed its performance of extensive geotechnical studies and investigations addressing all of the State's concerns since the filing of Utah L. On January 30, 2001, the State of Utah filed its response opposing the grant of summary disposition on Utah L and the Staff filed a response in support of PFS's motion.

On January 31, 2001, the Licensing Board issued a Memorandum and Order ruling on the State's request to modify basis 2. The Board acknowledged, in accordance with its previous decisions, that because the State's Request challenged an exemption from the Commission's regulation it was not subject to challenge in an adjudicatory proceeding absent a Commission directive to the contrary. LBP-01-03, 53 NRC 84 (2001). In accordance with Applicant's suggestion, however, the Board went on to determine

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[3] State of Utah's Request for Admissiion of Late-filed Modification to Basis 2 of Contentin Utah L (November 9, 2000) ("State's Request".) The State had previously sought to amend Utah L to challenge the use of a 2,000-year return earthquake in the probabilistic seismic analysis for the PFSF. The State's earlier requests were denied by the Board on the ground that they were not yet ripe for determination. LBP-99-21, supra, 49 NRC at 437; LBP-00-15, 51 NRC 313, 318 (2000).

whether the State had raised what would otherwise be an admissible contention. While rejecting some parts of the State's proposed contention, it held that the bulk of the State's request would constitute an admissible contention under the NRC's rules of practice. The Board also clarified and restated (from its reading of the State's pleadings) the matters in controversy. The Board certified to the Commission the question of "whether the State's challenge . . . to the April 1999 PFS seismic exemption request should be litigated in this proceeding along with a referral of [its] rulings . . . on the admissibility of the items the State has framed in support of its challenge." Id. at 101.

By an order dated June 14, 2001, the Commission specifically authorized this Board to resolve the State's challenge to the NRC Staff's grant of an exemption pursuant to 10 C.F.R. §72.102(f) to allow the PFSF to be designed using a PSHA methodology and a 2,000-year return period DBE. Private Fuel Storage L.L.C. (Independent Spent Fuel Storage Installation), CLI-01-12, 53 NRC 459, aff'g LBP-01-03, 53 NRC 84 (2001). This issue was denominated Part B of Utah L and the original contention and its four bases was denominated Part A.

Discovery was held on Part B of Utah L and on November 9, 2001, PFS moved for summary disposition in its favor on this part of the contention. On December 7, 2001, the State filed a response opposing PFS's motion for summary disposition. The Staff filed a response supporting the motion. On January 9, 2002, the Board denied PFS's motion for summary disposition on Part B of Utah L. Private Fuel Storage L.L.C. (Independent Spent Fuel Storage Installation), LBP-02-01, 55 NRC 11 (2002). The Board directed the parties, *inter alia*, to combine Part B of Utah L into the unified geotechnical contention with other pending seismic issues, as discussed below.

2. Utah Contention QQ

On December 16, 1999, Applicant filed License Application ("LA") Amendment No. 8 ("LA 8"). This amendment incorporated the use of soil cement beneath and around the spent fuel cask storage pads. The amendment to the SAR filed with LA 8 included specifications for the use of soil cement, calling for the use of American Concrete Institute ("ACI") standards to govern the placement and treatment of the soil cement. The calculations for the sliding stability of the cask storage pads under seismic loads were also revised to incorporate the increased stability afforded by the use of soil cement. PFS submitted an additional License Application Amendment taking into account the effects of soil cement in increasing storage pad stability and the use of soil cement as a buttress around the CTB. The design basis ground motions were further revised on March 30, 2001, and PFS submitted a License Application Amendment 22.

On May 16, 2001, the State filed a motion to admit Contention Utah QQ, dealing with seismic matters involving revised calculations submitted by the applicant, including the use of soil cement as part of the PFSF seismic design. See State of Utah's Request for Admission of Late-Filed Contention Utah QQ (Seismic Stability). The Applicant responded on May 30, arguing that the contention should not be admitted into the proceeding ad the Staff generally opposed admission of the contention.

In response to a request for additional information by the NRC Staff, PFS submitted further revisions to two calculations on May 31, 2001. On June 19, 2001, the State filed its First Request to modify the bases of Proposed Utah QQ based on the revisions PFS had made to the two above calculations. On July 3, 2001, PFS filed its Response to the State's First Request, opposing the State's request to modify Proposed Utah QQ. The NRC Staff also generally opposed the First Request.

On July 27, 2001, PFS responded to additional requests for information from the Staff by filing additional revisions to storage pad and CTB stability calculations, including added a hypothetical analysis of the potential sliding of the cask storage pad in a seismic event under "obviously conservative" assumptions. On August 7, Applicant filed an analysis performed by Holtec of the effect of hypothetical sliding of the pad on Holtec's cask stability analysis.

On August 23, 2001, the State filed a second request to modify the bases of Proposed Utah QQ based on the calculations that PFS had filed in response to the NRC's information requests. State of Utah's Second Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to More Revised Calculations for the Applicant. On September 7, 2001 PFS filed a response opposing the State's second request and the NRC Staff filed a response generally opposing the request.

3. Unified Contention Utah L/QQ

On December 26, 2001, in the same order denying PFS's motion for summary disposition of Part A of Utah L, Utah QQ was also admitted, and the State was granted leave to amend the bases of Utah QQ to reflect more recent analyses by PFS. LBP-01-39, 54 NRC 497 (2001). On January 9, 2002, the Board further ruled that the issues raised in Part B of Utah L should be incorporated into the unified geotechnical contention by January 17, 2002. LBP-02-01, 55 NRC at 13.

On January 16, 2002, the parties filed a "Joint Submittal of Unified Geotechnical Contention, Utah L and Utah QQ," setting forth all remaining geotechnical issues and the bases the State asserted in support of those issues. The joint submittal of Unified Contention L/QQ is included in the record of this proceeding as PFS Exhibit 237. The unified contention comprised five sections:

- Section A was basis 1 of Utah L, Part A;
- Section B was basis 2 of Utah L, Part A;
- Section C included basis 3 of Utah L, Part A and the portion of Utah QQ that raised soil cement issues;
- Section D included the remainder of the Utah QQ issues;

Section E was Utah L, Part B.[4]

Hearings on Unified Contention L/QQ were held in Salt Lake City from April 29, 2002 through May 13; on May 16 and 17; and June 3 through June 8, 2002. An additional two weeks of seismic hearings were held from June 17 through June 27, 2002 at the NRC headquarters in Rockville, Maryland.

B. Witnesses

1. PFS Witnesses

a. Section C of Utah Contention L/QQ

PFS's witnesses on seismic issues relating to Section C soil characterization and soil cement issues of State Contention Utah L/QQ, were Paul J. Trudeau, who addressed both soil characterization and soil cement issues, and Anwar E.Z. Wissa who addressed soil cement issues. Joint Testimony of Paul J. Trudeau and Anwar E. Z. Wissa on Section C of Unified Contention Utah L/QQ (inserted into the record after Tr. 10834 and/or Tr. 11724) ("Trudeau/Wissa Dir."). Mr. Trudeau and Dr. Wissa also presented rebuttal testimony regarding precedents for the use of soil cement at the PFSF, and the effects of dynamic loading and other factors on the performance of soil cement. Rebuttal Testimony of Paul J. Trudeau and Anwar E.Z. Wissa to Direct Testimony of State of Utah Witnesses Dr. Steven F. Bartlett and James K. Mitchell on Section C of Unified Conten-

[4] The parties filed a Joint Stipulation of Facts and Issues not in Dispute on January 31, 2002 which resolved Sections A and B of the unified contention.

tion Utah L/QQ (inserted into the record after Tr. 11232) ("Trudeau/Wissa Reb."). Mr. Trudeau filed individual rebuttal testimony on the soils characterization issue, addressing, factors of safety against sliding, the spacing of soil borings for the pad emplacement area, various aspects of the PFS soil sampling and testing programs, as well as the strength of the soils in the CTB area. Rebuttal Testimony of Paul J. Trudeau to Testimony of State of Utah Witness Dr. Stephen F. Bartlett on Section C of Unified Contention Utah L/QQ (Soils Characterization) (inserted into the record after Tr. 11724) ("Trudeau Soils Reb.").

Paul J. Trudeau is a Senior Lead Geotechnical Engineer at Stone & Webster, Inc., a Shaw Group Company ("S&W") in Stoughton, Massachusetts. He received a Master of Science in Civil Engineering from MIT and has twenty-nine years of experience in geotechnical engineering, including the performance of subsurface soil investigations; the performance and supervision of the analysis of foundations in support of the design of structures; the performance of laboratory tests of soils, including index property tests, consolidation tests, static and dynamic triaxial tests, as well as other tests; the performance of analyses of the performance of soils and structures under static and dynamic conditions; the development of geotechnical design criteria for other engineering disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical; and the preparation of the geotechnical sections of Preliminary and Final Safety Analyses Reports and Environmental Reports.

Dr. Anwar E. Z. Wissa is the President of Ardaman and Associates ("A&A") in Orlando, Florida. Dr. Wissa received a Doctor of Science in Geotechnical Engineering from MIT, he also received a Master of Science in Civil Engineering from MIT and a Master of Arts from Oxford University. A&A provides numerous services, including subsurface investigations, foundation engineering, laboratory testing, construction materials testing and inspection, and contamination remediation. Dr. Wissa has been a Fellow of the American Society of Civil Engineers since 1983, serving on the Committee on Placement and Improvement of Soil for nine years. He also has been a member of Committee D-18 on Soil and Rock for the American Society of Testing and Materials ("ASTM") since 1966. Mr. Wissa has been extensively involved in projects employing soil cement, including reservoirs and pavements, over his professional career, and has authored several publications on the use of soil cement. He is co-author of the "State of the Art Report" on soil cement issued by the American Concrete Institute, a standard reference work on soil cement applications. See PFS Exh. HHH.

b. Section D of Utah Contention L/QQ

PFS's testimony on the dynamic analyses related to the design of the PFSF was provided by four panels consisting of six witnesses. Paul J. Trudeau provided testimony on the dynamic analyses of the foundations of the storage pads and the CTB at the PFSF. Testimony of Paul J. Trudeau on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 6135) ("Trudeau Section D Dir."). Mr. Trudeau also provided rebuttal testimony addressing the accelerations use in the dynamic analysis of the storage pads, long term pad settlement, and pad-to-pad interaction. Rebuttal Testimony of Paul J. Trudeau on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 11275 ("Trudeau Section D Reb.").

Dr. Robert Y. Youngs and Dr. Wen Tseng provided joint testimony. Dr. Youngs addressed the soil input parameters used in PFS's dynamic analyses and the incorporation of fault fling in time histories used for PFS's seismic design. Both Dr. Youngs and Dr. Tseng addressed the effect of non-vertically propagating seismic waves on the storage pads at the PFSF and the frequency dependency of the soil spring and damper valves used in the PFS cask stability analyses. Dr. Tseng addressed the design and analysis of the performance of the storage pads, including the rigidity of the pad, and **[check – and**

add other issues.] Joint Testimony of Robert Youngs and Wen Tseng on Unified Contention Utah L/QQ (inserted into the record after Tr. 5529) ("Youngs/Tseng Dir."). Each of the witnesses filed separate rebuttal testimony. Dr. Youngs filed rebuttal testimony regarding non-vertically propagating waves. Rebuttal Testimony of Robert Y. Youngs on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 10479) ("Youngs Reb."). Dr. Tseng also submitted rebuttal testimony regarding rigidity as the pad, frequency dependency of the soil springs and dampers used in the cask stability analysis and other matters. Rebuttal Testimony of Wen S. Tseng on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 10727) ("Tseng Reb.").

Mr. Bruce E. Ebbeson provided testimony regarding the structural design and analyses related to the seismic performance of the CTB at the PFSF. Testimony of Bruce E. Ebbeson on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 6357) ("Ebbeson Dir."). Mr. Ebbeson also provided rebuttal testimony concerning dynamic interaction between soil cement and the CTB mat foundation, as well as on the rigidity of the CTB mat. Rebuttal Testimony of Bruce E. Ebbeson on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 10790) ("Ebbeson Reb."). Finally, Dr. Krishna P. Singh and Dr. Alan I. Soler provided testimony jointly on the design and performance of the HI-STORM 100 storage casks at the PFSF addressing each of the issues raised by the State regarding cask performance and stability under seismic loading. Testimony of Krishna P. Singh and Alan I. Soler on Unified Contention Utah L/QQ (inserted into the record after Tr. 5750) ("Singh/Soler Dir."). Dr. Soler filed individual rebuttal testimony on Section D, addressing the effects of pad-to-pad interaction and long term settlement on the cask stability analyses conducted for the PFSF. Rebuttal Testimony of Alan I. Soler on Section D of Unified Contention Utah L/QQ (inserted into the record after Tr. 10557) ("Soler Reb.").

Dr. Robert Y. Youngs is a Principal Engineer employed by Geomatrix Consultants Inc., in Oakland, California. He received a Ph.D. and a Masters of Science degree in Geotechnical Engineering from the University of California, and has over 25 years of professional consulting experience, primarily focused in the analysis of seismic hazards. His experience encompasses, among other areas, the characterization of earthquake ground motions and the performance of probabilistic and deterministic analyses to develop seismic design criteria for ground motion and fault displacement. Dr. Youngs has conducted these types of analyses for seven NRC-regulated nuclear power plants located in the Western United States. He has also performed similar studies for nuclear power plants in Canada, Spain, Slovakia, and Bulgaria, and is currently involved in similar studies for nuclear power plants in Switzerland and Slovenia. In addition, he has performed similar studies for existing and proposed Department of Energy ("DOE") nuclear facilities at Hanford, Washington; Idaho National [fill in rest] INEEL, Idaho; Rocky Flats, Colorado; Savannah River, South Carolina; and Yucca Mountain, Nevada.

Dr. Wen Shou Tseng is the President of International Civil Engineering Consultants, Inc. ("ICEC"). He received his Ph.D. and a Masters of Science in Civil Engineering from the University of California at Berkeley. ICEC is a company that provides specialty consulting services in the general areas of civil and structural engineering with special emphasis on earthquake engineering. As President of ICEC, Dr. Tseng is responsible for all aspects of the company operation including technical, administrative, financial, contractual and business development matters. He has conducted research and development, and has performed consulting services in the general areas of civil and structural engineering for more than 30 years. His area of specialization is earthquake engineering with special emphasis on the evaluation of soil-structure interaction effects on structures. Prior to joining ICEC as President, Dr. Tseng was with Bechtel, where he headed the

Special Structures group performing research and development and providing technical consulting services to many nuclear power projects. After forming ICEC, Dr. Tseng has continued to work on nuclear power plant projects on seismic design and analysis issues including, among others soil- structure interaction experimental programs, seismic instrumentation, seismic design and analysis, and related work. Has also worked on N.P.P. at ICEC. He has also published many technical papers and technical and project reports on seismic and soil-structure interaction subjects.

Bruce E. Ebbeson is a Senior Lead Structural Engineer with S&W in Cherry Hill, New Jersey. He received a Masters of Science in Civil Engineering from Tufts University, and has approximately thirty years of experience as a Civil/Structural Engineer, specializing in the structural design and analysis, including seismic analysis, of nuclear facilities. Currently, he is the supervisor of the structural division for S&W's Cherry Hill office and serve as structural engineering consultant on various projects performed by S&W in its Cherry Hill, Boston, Denver and Taiwan offices. His experience has included assignments as Principal Structural Engineer on many nuclear facility projects. Mr. Ebbeson has, among other activities, conducted and supervised the performance of original designs and design modifications for those projects, as well as safety evaluations to meet licensing requirements. He also has performed independent design reviews of nuclear facilities at various stages of licensing and operation.

Dr. Krishna P. Singh is the President and CEO of Holtec International ("Holtec"). He received his Ph.D. and Masters of Science in Mechanical Engineering from the University of Pennsylvania. He over twenty-three years of experience in the design and licensing of nuclear spent fuel systems. Dr. Singh led the design and licensing of spent fuel storage systems for over forty nuclear plants, and for Holtec's HI-STAR 100 System and HI-STORM 100 Storage Cask System ("HI-STORM System"). He is also the inventor of numerous spent fuel storage system components recognized as significant contributions to dry storage technology. Dr. Singh has authored over 500 industry reports and over fifty published papers in the referenced technical literature.

Dr. Alan I. Soler is the Executive Vice President and Vice President of Engineering for Holtec International. Dr. Soler received a Ph.D. in mechanical engineering from the University of Pennsylvania and a Masters of Science in mechanical engineering from the California Institute of Technology. Dr. Soler is the lead structural discipline expert responsible for the design of the HI-STORM System, including supporting analyses. He has either performed or reviewed all HI-STORM System seismic analyses conducted in support of deployment of the HI-STORM System at the PFSF. Prior to his current employment with Holtec International, he was a tenured Professor of Mechanical Engineering and Applied Mechanics at the University of Pennsylvania for over 26 years.

c. ***Section E of Utah Contention L/QQ***

PFS presented the direct testimony of five witnesses on Section E of Utah Contention L/QQ, regarding the seismic exemption granted by the NRC Staff for the PFSF. Dr. Krishna P. Singh, Dr. Alan I. Soler and Dr. Everett L. Redmond, II presented testimony regarding the radiological dose consequences of the seismic exemption, including applicable dose limits for beyond design basis accident events and the lack of radiological consequences if a hypothetical cask tipover or impact would occur. Testimony of Krishna P. Singh and Alan I. Soler, and Everett L. Redmond II on Radiological Dose Consequence Aspects of Basis 2 of Section E of Unified Contention Utah L/QQ (inserted into the record after Tr. 12044) ("Singh/Soler/Redmond Dir."). Mr. Donald Wayne Lewis presented testimony concerning canister transfer operations and the safety classification of the structures, systems and components ("SSCs") relevant to Unified Contention Utah L/QQ. Testimony of Donald Wayne Lewis on Section E of Unified Contention

Utah L/QQ (inserted into the record after Tr. 8968) ("Lewis Dir."). Dr. C. Allin Cornell presented testimony regarding the seismic exemption requested by PFS to use a PSHA methodology based on a 2,000-year return period earthquake as the seismic design basis for the PFSF. Dr. Cornell addressed the appropriateness of using a probabilistic seismic hazard analysis as the basis for designing the PFSF, the sufficiency of a 2,000-year return period design basis earthquake, the seismic related design procedures and criteria contained in NRC guidance documents, and several issues raised by the State regarding the NRC's granting of PFS's exemption request. Testimony of C. Allin Cornell (inserted into the record after Tr. 7856) ("Cornell Dir."). Dr. Cornell also filed rebuttal testimony regarding issues raised by the direct testimony of State witness Walter J. Arabasz. Rebuttal Testimony of C. Allin Cornell to the Testimony of State Witness Dr. Walter J. Arabasz on Section E of Unified Contention Utah L/QQ (inserted into the record after Tr. 12951) ("Cornell Reb.").

Dr. C. Allin Cornell is currently a research professor at Stanford University and an independent engineering consultant. He received a Ph.D. and a Masters of Science in Civil Engineering from Stanford University. As a research professor, he performs research and supervises several Ph.D.-level graduate students in the areas of probabilistic analysis of structural engineering and earthquake engineering. As a consultant, he assists engineering and earth sciences firms, industrial concerns, and government agencies in developing and applying methodologies and standards for probabilistic seismic hazard analysis, engineering safety assessments, natural hazards analyses, and earthquake engineering. Dr. Cornell has extensive professional expertise in earthquake engineering, probabilistic engineering analysis of seismic and other loads on structures, and structural responses to such loads. He has been actively involved in the development of structural design guidelines, codes and standards, including the appropriate level of earthquake design required to achieve a desired level of safety. Dr. Cornell has participated directly, commonly as a senior advisor, in many prominent PSHA studies, including the PSHA for the Diablo Canyon Nuclear Power Plant ("NPP"), the major EPRI Seismic Owners Group PSHA of the Central and Eastern US ("CEUS") NPP sites, the Caltrans-sponsored PSHA studies of all major California bridges, and PSHAs for the INEEL and LLNL DOE national lab sites and the Yucca Mountain site. Dr. Cornell was also a member of the Senior Seismic Hazard Analysis Committee ("SSHAC") (sponsored jointly by NRC, EPRI and DOE) to establish "standards" for conducting PSHAs at nuclear facility sites.

Dr. Cornell was one of the originators of seismic probabilistic risk analysis ("SPRA") for nuclear power plants, beginning with informal advice to his then MIT colleague Norman Rasmussen who directed the first nuclear power plant PRA, WASH 1400. He co-authored, with Nathan Newmark, the first published SPRA paper (presented by invitation at the annual meeting of the American Nuclear Society); this was followed by a second paper (co-authored by several structural and nuclear engineers) based on the first practical application to a specific NPP (Oyster Creek). Dr. Cornell has participated in numerous SPRA studies for nuclear facilities, including the Diablo Canyon NPP, and was a member of the NRC-sponsored Senior Seismic Margins Research Project committee responsible for directing a major project conducted by the LLNL studying the fragility curves of NPP SSCs.

He has been involved in establishing seismic of design standards for nuclear power plants, radiological waste facilities, offshore oil platforms, and buildings. Dr. Cornell has extensive experience in seismic probability risk assessment and margins criteria development.

Donald Wayne Lewis is currently employed by S&W as the Lead Mechanical Engineer for the PFSF project. Mr. Lewis received his Bachelor of Science in Civil Engineering from Montana State University and has 19 years of experience in the nuclear power industry, including 10 years of experience with the design, licensing, construction, and operation of independent spent fuel storage installations.

Dr. Everett L. Redmond, II is a Principal Engineer and Manager of the Nuclear Physics Department with Holtec. He received a Ph.D. and a Masters of Science in Nuclear Engineering from MIT, where he also minored in biology. He is responsible for all shielding, criticality, and confinement analysis work related to Holtec's dry cask storage systems. Dr. Redmond is the author of the shielding analyses performed in support of the general NRC certification of Holtec's HI-STORM 100 Cask System under Docket 72-1014. He also has performed site-specific shielding analyses in support of deployment of the HI-STORM 100 Cask System at the PFSF. Dr. Redmond's professional background and work experience include significant expertise on matters pertaining to the shielding characteristics of the HI-STORM 100 Cask System and the radiation dose associated with the use of the HI-STORM 100 Cask System. His work in those areas has included developing analytical methods and models for conducting shielding analyses and dose calculations, and performing site boundary dose evaluations for ISFSIs.

PFS presented one additional oral rebuttal witness on radiological dose consequences, Mr. John Donnell. He testified regarding the land use conditions surrounding the PFSF site and the feasibility of taking remedial action in the event of a beyond design basis accident.

Mr. Donnell is the Project Director for the Applicant. In this capacity, he is responsible for the execution and integration of the legal and technical activities of the PFSF project. He is knowledgeable about the land use in Skull Valley and PFS operations. He has 21 years of experience in nuclear project management and engineering.

2. **NRC Staff Witnesses**

a. Section C of Utah Contention L/QQ

The NRC Staff testimony on Section C of State Contention L/QQ was presented by Dr. Goodluck I. Ofoegbu. NRC Staff Testimony of Goodluck I. Ofoegbu Concerning Unified Contention Utah L/QQ, Part C (inserted into the record after Tr. 11001) ("Ofoegbu Dir."). Dr. received a Ph.D. in Geological Engineering from University of Toronto. He is employed as a Principal Engineer at the Center for Nuclear Waste Regulatory Analyses ("CNWRA"), which is a division of the Southwest Research Institute ("SWRI"), in San Antonio, Texas. In his position as Principal Engineer at the CNWRA, he serves as Principal Investigator for several projects involving geological engineering. His work includes mechanical analysis of underground excavations, foundations, earthworks, and natural geological processes such as faulting and volcanism.

b. Section D of Utah Contention L/QQ

The NRC Staff Testimony on Section D of Utah Contention L/QQ was presented in two panels. The first panel addressed the dynamic analyses conducted for the design of the storage pads and the CTB. Daniel J. Pomerening presented testimony for the Staff jointly with Dr. NRC Staff Testimony of Daniel J. Pomerening and Goodluck I. Ofoegbu Concerning Unified Contention Utah L/QQ, Part D (Seismic Design and Foundation Stability) (inserted into the record after Tr. 6496) ("Pomerening/ Ofoegbu Dir.").

Daniel J. Pomerening is employed as a Principal Engineer in the Mechanical and Materials Engineering Division of the SWRI. He received a M.E. in Civil Engineering, Structural Engineering and Structural Mechanics from University of Califonia Berkeley. In his position as Principal Engineer at the Mechanical and Materials Engineering Division, he serves as principal investigator for projects associated with the evaluation of structural design and environmental testing of systems and components, with an emphasis on dynamic loading. Among his responsibilities related to the CNWRA, he currently serves as an investigator for several projects involving the technical evaluation of facility operation systems, evaluation of the adequacy of design criteria, evaluation of the structural design of the facility, and review of accident analyses.

Testimony regarding PFS's cask stability analysis, the independent cask stability analyses conducted by the NRC Staff, and other related issues was presented by Dr. Vincent K. Luk and Mr. Jack Guttman. NRC Staff Testimony of Vincent K. Luk and Jack Guttman Concerning Unified Contention Utah L/QQ (Geotechnical Issues) (inserted into the record after Tr. 6760) ("Luk/Guttman Dir.").

Mr. Guttman is employed as Chief of the Technical Review Section, Spent Fuel Project Office ("SFPO"), Office of Nuclear Material Safety and Safeguards ("NMSS") of the NRC. He has a Masters of Science degree in Nuclear Engineering from University of Michigan. As Chief of the Technical Review Section in the SFPO, his responsibilities include direction and supervision of various technical reviews related to the licensing and certification of radioactive material transportation and storage packages, under 10 C.F.R. Parts 71 and 72, respectively, including technical reviews related to independent spent fuel storage installations ("ISFSIs"). Among his other responsibilities, he routinely directs and supervises the evaluation and use of computer code modeling and analytical methodologies in assessing the safety and performance of radioactive material transportation and storage packages.

Vincent K. Luk is employed as a Principal Member of the Technical Staff in the Nuclear Technology Programs Department at Sandia National Laboratories ("SNL"), in Albuquerque, New Mexico. He received a Ph.D. and a Masters of Science in Theoretical and Applied Mechanics from Northwestern University, and currently serves as Leader of the Structural Analysis and Evaluation Team for an NRC Integrated Vulnerability Assessment Project, examining the vulnerability and structural integrity of nuclear power plants subjected to external high-energy impacts. In addition, Dr. Luk serves as the Principal Investigator in an NRC project, establishing criteria and review guidelines in evaluating the seismic behavior of dry cask storage systems; and in examining the dynamic seismic behavior of free-standing dry cask storage systems and soil-structure interaction effects in simulated earthquake events.

c. Section E of Utah Contention L/QQ

The Staff's testimony regarding the seismic exemption issues in Section E consisted of a panel of three witnesses: Dr. John A. Stamatakos, Dr. Rui Chen, and Dr. Martin W. McCann, Jr. NRC Staff Testimony of John A. Stamatakos, Rui Chen and Martin W. McCann, Jr., Concerning Unified Contention Utah L/QQ, Part E (Seismic Exemption) (inserted into the record after Tr. 8051) ("Stamatakos/Chen/McCann Dir."). Dr. Stamatakos also filed individual rebuttal testimony on the seismic exemption issue. NRC Staff Rebuttal Testimony of Dr. John A. Stamatakos Concerning Unified Contention Utah L/QQ, Part E (Seismic Exemption) (inserted into the record after Tr. 12648) ("Stamatakos Reb."). The Staff also presented testimony on the radiological dose consequences of the seismic exemption through Michael D. Waters. NRC Staff Testimony of Michael D. Waters Concerning Unified Contention Utah L/QQ, Part E (inserted into the record after Tr. 12215) ("Waters Dir.")

Dr. John A. Stamatakos is a Principal Scientist at the CNWRA. He received his Ph.D. and Masters of Science in Geology from Lehigh University. In his position as Principal Scientist at the CNWRA, Dr. Stamatakos serves as the Principal Investigator for several projects involving technical evaluation of structural deformation and seismicity, including tectonics and neotectonics research. His work includes field analyses of the structural and tectonic elements of the Basin and Range province in the southwestern

United States, and the evaluation of seismic and faulting hazards at various nuclear facilities.

Dr. Rui Chen received her Ph.D. in Civil and Geological Engineering from the University of Manitoba, Winnipeg, Canada. She is an independent consultant in geological engineering and geosciences. She was employed for five years as a Research Engineer and Senior Research Engineer at the CNWRA, where she was involved in various matters including the technical analysis of mechanical, thermal, and hydrological processes in complex geomechanical and geotechnical engineering systems related to the proposed Yucca Mountain repository; and the evaluation of seismic hazard analyses and seismic design related to proposed spent fuel storage facilities, including the proposed PFS Facility. Since then, she has provided technical assistance and consulting services to the CNWRA at SWRI involving a broad range of problems in underground rock engineering, seismic hazard assessment, and earthquake engineering; including the evaluation of seismic and geotechnical hazards at various nuclear facilities. She also teaches graduate and undergraduate courses in the fields of geotechnical engineering and geosciences in the Department of Civil Engineering and College of Natural Sciences at the California State University at Chico, California.

Dr. Martin W. McCann, Jr. is President of Jack R. Benjamin & Associates, Inc., in Menlo Park, California. He received a Ph.D. in Civil Engineering and a Masters of Science in Structural Engineering from Stanford University. Among his duties at Jack R. Benjamin & Associates, Inc. Dr. McCann he provides consulting services to private industry and government entities, both in the United States and abroad, in the area of risk analysis for critical facilities, development of generic standards and guidelines for use in assessing seismic hazards and in the assessment of seismic hazards at specific sites, with emphasis in the area of PSHA. He has provided technical assistance and consulting services to the CNWRA at SWRI in its review of various PSHAs, including the PSHA for the proposed Yucca Mountain repository and other U.S. Department of Energy's ("DOE") nuclear facilities. In addition, in his position as a Consulting Professor of Civil and Environmental Engineering at Stanford University.

Michael D. Waters is a Health Physicist in the NRC's Spent Fuel Project Office. He received his Masters of Science in Nuclear Engineering Sciences from University of Florida. Mr. Waters is responsible for performing technical reviews of spent nuclear fuel storage casks, ISFSIs and transportation packages, primarily in the areas of shielding, confinement, containment, radiation protection, and criticality. In addition, he continues to be responsible for certain reviews initiated in his former position as a Project Engineer in SFPO, involving management of the safety reviews of applications for these designs and facilities. His safety reviews have included both new ISFSI license applications and amendments to existing licenses.

3. State Witnesses

a. Section C of Utah Contention L/QQ

Testimony for the State on issues relating to Section C of Utah L/QQ was provided by Dr. Steven F. Bartlett individually on the soils characterization issues and in a panel with Dr. James K. Mitchell on soil cement issues. State of Utah Testimony of Dr. Steven F. Bartlett on Unified Contention Utah L/QQ (Soils Characterization) ("Bartlett Soils Dir."); State of Utah Testimony of Dr. Steven F. Bartlett and Dr. James K. Mitchell on Unified Contention Utah L/QQ (Soil Cement) (Bartlett/Mitchell Dir."). Dr. Bartlett also filed surrebuttal addressing Mr. Trudeau's prefiled rebuttal testimony regarding soils characterization. Surrebuttal of Dr. Steven Bartlett to PFS Witness Paul Trudeau's Rebuttal Testimony on Section C of Unified Contention Utah L/QQ (inserted into the record after Tr. 119981) ("Bartlett Soils Reb.").

Dr. Bartlett is an Assistant Professor in the Civil and Environmental Engineering Department of the University of Utah, focusing on geotechnical engineering. He is a licensed professional engineer in the State of Utah. Dr. Bartlett testified regarding dynamic analysis, soils characterization, the use of soil cement and the seismic exemption. Dr. Bartlett's expertise is in soils and structural foundations. Dr. Bartlett is not a structural engineer and has not conducted any dynamic analyses of free standing structures under seismic conditions.

Dr. James K. Mitchell is a University Distinguished Professor Emeritus at Virginia Tech and Professor Emeritus at the University of California at Berkeley. His experience in the field of geotechnical engineering focuses on soil behavior, soil and site improvement, and foundation engineering including the design, testing, and use of soil cement for soil stabilization.

b. Section D of Utah Contention L/QQ

The State presented the testimony of three witnesses in two panels on Section D of Utah Contention L/QQ relating to the dynamic analyses conducted for the PFSF. Dr. Steven F. Bartlett and Dr. Farhang Ostadan presented testimony on PFS's dynamic analyses of the storage pads and the CTB. State of Utah Testimony of Dr. Steven F. Bartlett and Dr. Farhang Ostadan on Unified Contention Utah L/QQ (Dynamic Analyses) (inserted into the record after Tr. 7268) ("Bartlett/Ostadan Dir."). Dr. Bartlett also presented surrebuttal to the testimony of Paul Trudeau on Section D. State of Utah Partial Surrebuttal Testimony of Dr. Steven F. Bartlett to Rebuttal Testimony of Paul J. Trudeau on Unified Contention L/QQ (Dynamic Analyses) (inserted into the record after Tr. 11306) ("Bartlett Section D Reb.").

Dr. Ostadan is a consultant in the field of soil dynamics and geotechnical earthquake engineering. He testified regarding the dynamic analyses conducted for the PFSF. Dr. Ostadan's relevant background and experience are primarily in soil structure interaction issues as they apply to structural foundations.

Dr. Moshin R. Khan and Dr. Farhang Ostadan presented additional testimony regarding the cask stability analyses conducted by PFS. State Testimony of Dr. Mohsin R. Khan and Dr. Farhang Ostadan on Unified Contention Utah L/QQ, Part D (cask stability) (inserted into the record after Tr. 7123) ("Khan/Ostadan Dir.").

Dr. Khan is a consultant and registered professional engineer, with a doctorate in the mechanics of structures. He testified regarding the adequacy of the PFS cask stability analyses. Dr. Khan had never performed any seismic analyses, modeling or testing of free standing structures. His experience with testing and modeling equipment is limited to equipment that is bolted or affixed to structures, such as electrical equipment.

c. Section E of Utah Contention L/QQ

The State presented three witnesses regarding Section E of Utah Contention L/QQ, the PFS seismic exemption. Dr. Bartlett presented testimony regarding the application of DOE seismic classification standards and probabilistic seismic hazard analyses. State of Utah Testimony of Dr. Steven Bartlett on Unified Contention Utah L/QQ, Part E (Lack of Design Conservatism) ("Bartlett Section E Dir."). Dr. Bartlett has only limited experience in the application of such standards or in conducting a probabilistic seismic hazard analysis.

Further testimony on the Staff's rationale for approving the PFS seismic exemption was presented by Dr. Walter J. Arabasz. Dr. Arabasz is a professor of geology and geophysics at the University of Utah. State of Utah Testimony of Dr. Walter J. Arabasz on Unified Contention Utah L/QQ (Seismic Exemption) (inserted into the record after Tr. 9098) ("Arabasz Dir.").

The State's sole witness regarding the radiological dose consequences of the seismic exemption was Dr. Marvin Resnikoff, a physicist. State of Utah's Testimony of Dr. Marvin Resnikoff Regarding Unified Contention Utah L/QQ (Seismic Exemption - Dose Exposure) (inserted into the record after Tr. 12349) ("Resnikoff Dir."). He performed two radiological dose calculations regarding the effects of a worst case beyond design basis, seismically-induced accident at the PFSF. Dr. Resnikoff has no expertise concerning seismology, the dynamic behavior of structures or storage casks, the structural engineering or design of storage casks, or the mechanisms by which damage might occur to a storage cask such as by deformation of the storage cask due to impact, cracking of concrete, thinning of concrete or steel, stretching of steel, loss of hydrogen in cask shielding due to cask heat up, or displacement of a cask lid during tipover. Additionally, he has no experience or expertise in determining the duration of a beyond design basis accident.

II. OVERVIEW OF THE SEISMIC ISSUES

A. Introduction

As the above background discussion illustrates, the seismic contention has undergone significant modifications since it was first advanced by the State at the start of this proceeding. The aspects of the unified contention on which hearings were held[5] make essentially four major claims: (1) that PFS has insufficiently characterized – through bor-

[5] Several other seismic issues once raised by the State have been withdrawn by stipulation of the parties or by unilateral decision of the State. These include, among others, Basis 4 of original Contention L; certain portions of Basis 3; and Sections A and B of Contention L/QQ (which corresponded to Bases 1 and 2 of original Contention L) See Joint Stipulation of Facts and Issues Not in Dispute With Repect to Unified Contention Utah L/QQ, January 31, 2002.

ings, samplings, laboratory analyses and other testing -- the soils beneath the safety-related structures at the PFSF to accurately predict how those soils will behave in the event of a design basis earthquake; (2) that the proposed use of soil-cement and cement-treated soil to improve the soil's subsurface conditions is new and unproven and must be demonstrated by testing prior to licensing of the facility, and that the soil cement is subject to degradation with time; (3) that the dynamic analyses performed by PFS to determine the stability of the casks, storage pads and CTB under seismic loadings are so flawed that the results of the analyses cannot be relied on to demonstrate that these structures and components will be able to withstand a seismic event; and (4) that the Staff's granting of an exemption from NRC regulatory requirements so as to allow PFS to design the facility based on a probabilistic seismic hazards analysis and a 2,000 year return period earthquake was improper and may lead to radiological releases in excess of regulatory limits in the event of such an earthquake.

B. Soils Characterization Issues

Placed at issue in Section C.1 of Utah L/QQ is the sufficiency of PFS's characterization of the soils beneath the safety-related structures at the PFSF – through borings, samplings, laboratory analyses and other testing – to accurately predict how those soils will behave in the event of a design basis earthquake. As set forth in the License Application and as demonstrated on the hearing record, PFS has carried out a comprehensive program of geotechnical investigations and laboratory tests which is presented in Section 2.6 and Appendix 2A of the SAR, as revised through April 2001 (Rev. 22). That section, 219 pages long plus attachments and appendices, describes the various investigations that have been conducted, and includes geologic maps, profiles of the site stratigraphy, and discussions of structural geology, geologic history, and engineering geology.[6] This program sufficiently characterizes the soils beneath the site for purposes of seismic design.

1. Soil Borings in the Pad Emplacement Area

Central to the PFS geotechnical investigations program has been the drilling of boreholes to investigate the soil properties and the performance of cone penetration ("CPT") tests.[7] The State is satisfied with the sufficiency of the borings conducted by PFS for the soils under the Canister Transfer Building ("CTB"). However, it contends that PFS failed to perform a sufficient number of borings for the pad emplacement area because it used an approximate borehole and CPT spacing of about 221 feet instead of the 100 feet recommended in NRC Reg. Guide 1.132.

Reg. Guide 1.132 provides guidance for the spacing of boreholes for investigating the underlying soils of nuclear power plants. The State has provided no convincing explanation why the NRC borehole spacing guidelines for nuclear power plants – should be appropriate for an ISFSI, such as the PFSF, which is a different and altogether less complex facility. In fact, Dr. Bartlett acknowledges that Reg. Guide 1.132 is a guidance document not applicable to ISFSIs. See Findings 4-10, infra.

Moreover, the record shows that the geotechnical characteristics of the PFSF have been well established through the borings and other investigations that have been conducted by PFS, and the State has made no showing that any difference might arise if the

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6 In particular, Figure 2.6-5 of the SAR includes 14 sheets of "foundation profiles" that depict the composition of the PFSF subsoil layers at various locations in the pad emplacement area, and Figures 2.6-20 through 2.6-22 of the SAR present foundation profiles under the CTB. These profiles provide a wealth of detail on the characteristics of the soils at the PFSF site. They cover all safety-related structures and encompass all borings and soil tests made by PSF in the vicinity of those structures.

7 Cone penetration tests are conducted using a device with a conical tip that is driven continuously into the soil, and which provides a record of the soil strength by tracking the stress required to advance the cone through the soil. Tr. 11727-29 (Trudeau).

borings under the storage pad were spaced 100 ft. apart instead of 221 feet. For example, the soils below approximately thirty feet are dense and have significant strength and very low compressibility, so they are of no concern from the geotechnical standpoint, and further investigation of them should not be required. Similarly, it is undisputed that immediately below the thin surface layer of aeolian soils there is a three to ten feet layer of silty clay and clayey silt variously identified as "Upper Lake Bonneville Deposits," "Layer 1B," and "Layer 2." While possessing considerable strength, the Upper Lake Bonneville Deposits are, relatively speaking, the least strong and most compressible soils in the profile and therefore, the parties agree, are the soils of interest for establishing minimum values of undrained soil shear strength. For that reason, the soils in this layer were the focus of the majority of the investigations conducted by PFS. See Findings 10-22, infra.

Further, the soils underlying the PFSF site exhibit significant layering as one proceeds vertically downwards, but are uniform as one moves horizontally from one location to another. The horizontal uniformity of the soils is demonstrated, among other things, by the consistency in the number of blows ("blow count") required to drive the sampler into the soil at various depths across the pad emplacement area; by the fact that the CPT tests performed in 1999 yielded essentially the same value of tip resistance for comparable depths at various locations across the pad emplacement area; and by data on the properties of the soils obtained by PFS consultant Geomatrix in a trench dug by Geomatrix in the pad emplacement area, which showed that the soils in approximately the upper 30 feet of the subsoil are uniform and consistent in the horizontal direction. See Findings 23-29.

Because of the horizontal uniformity of the soils in the pad emplacement area and the well-understood vertical layering of the soils, there is no reason to believe that additional borings or CPT tests with a density of 100 ft., as recommended by Reg. Guide 1.132 for nuclear power plants, would yield any additional or different information from that which is already available. Thus, no deficiency exists in the PFSF geotechnical investigation program in failing to meet the spacing recommendations in Reg. Guide 1.132.

2. Continuous Soil Sampling

The State also claims that PFS has not performed continuous sampling of critical soil layers important to foundation stability for each major structure as recommended by Reg. Guide 1.132. Again, there is no basis for applying to this facility the recommendations in that regulatory guide. In any event, PFS implemented the guidance in Reg. Guide 1.132 by conducting continuous sampling in both the pad emplacement area and the CTB through the use of CPT tests, which are continuous through the soil profile. See Findings 30-32.

The State asserts that the CPT testing conducted by PFS does not meet the guidance in Reg. Guide 1.132 because the continuous measurements taken by the cone penetrometer are not "sampling" since no soil samples are recovered for laboratory testing. However, the purpose of the recommendation in Reg. Guide 1.132 that continuous sampling be conducted is to identify "[r]elatively thin zones of weak or unstable soils [that] may be contained within more competent materials and may affect the engineering characteristics or behavior of the soil or rock." The CPT tests established that no such zones of weak or unstable soils exist at the pad emplacement area or under the CTB. Therefore, the objectives of Reg. Guide 1.132 with respect to continuous soil sampling have been achieved. See Findings 33.

3. Number of Tested Samples

It is undisputed that the main soils characteristic of interest in seismic analysis and design is their undrained shear strength, both with respect to horizontal and vertical loadings. Shear strength is established through laboratory testing. The State challenges the sufficiency of PFS's testing to determine the minimum horizontal shear strength of the soil in the pad emplacement area on the ground that PFS performed laboratory tests of shear strength on only three specimens taken from a single soil sample.

The soil sample used by PFS to measure the minimum horizontal shear strength of the soils in the pad emplacement area was obtained from the Upper Lake Bonneville Deposits layer, which as noted above contains the soils with lowest values of shear strength. The sample also exhibits the highest void ratio of all the samples tested in the pad emplacement area (signifying lowest density and hence lowest strength), and was taken from the quadrant in the pad emplacement area that had been determined to have the lowest soil strength. Accordingly, the manner in which that sample was selected demonstrates that this sample had the lowest shear strength found in the soils in the pad emplacement area, and that the value of undrained horizontal shear strength obtained by PFS from that sample constitutes a conservative lower bound of the strength of the soils at the pad emplacement area. See Findings 34-39.

That the selected sample had the lowest shear strength of the soils in the pad emplacement area was further confirmed by the CPT tests conducted by PFS. These tests involved taking continuous measurements of cone penetrometer tip resistance at 37 locations in the pad emplacement area. The report for those tests provided tables of numerical values of tip resistance versus depth, which recorded the actual measurements of tip resistance and allow a numerical correlation to be drawn between the measured tip resistance and the undrained shear strength of the soil at the various locations in the soil profile. The value of undrained shear strength corresponding to the lowest tabulated value of tip resistance is essentially the same as the value of shear strength measured in the laboratory by PFS. See Findings 40-41. Therefore, the cone penetration test results confirm that the value of minimum shear soil strength determined by PFS in laboratory tests is indeed the minimum value found in the pad emplacement area.

Dr. Bartlett claimed nonetheless that there can be considerable horizontal variability in the shear strength exhibited by the Upper Lake Bonneville soils across the pad emplacement area, and locations may exist at which the shear strength of the soil may be lower than that obtained by PFS. However, the CPT profiles show that the measured cone penetrometer tip resistance is remarkably uniform for a given depth from one location to another. Thus, there is no evidence of the existence of "considerable horizontal variability" in the shear strength of the soils within any given layer. See Finding 42.

In any case, even assuming that such variability exists, there is no evidence that there are locations in the place emplacement area where the soils in fact have lower shear strength than that obtained in the PFS laboratory tests. To the contrary, the sample selection procedure used by PFS and the tabulated results of the CPT testing provide assurance that the minimum shear strength value was found and used by PFS in its analyses.[8]

4. Failure to Perform Certain Tests

The final area of State concern with respect to the PFS geotechnical investigations is the failure to conduct two types of tests: strain-controlled cyclic triaxial tests and triaxial extension tests. The first of these tests is intended to measure certain properties of the soils (shear modulus and damping) versus soil shear strain (i.e., deformation) at high

[8] Even if soils of lower strength were to exist in the pad emplacement area, there are numerous conservatisms incorporated into the PFS analyses and design that more than compensate for the difference between a speculated lower strength and that utilized by PFS in its analyses. See Findings 43.

strain levels under dynamic loading conditions. While PFS did not conduct this type of test, it performed a different type of strain-controlled test, the resonant column test, that provides the same information for the conditions existing at the PFSF site. Therefore, the strain-controlled cyclic triaxial tests the State seeks are unnecessary. See Findings 44-46.

In addition, other tests conducted by PFS determined that the shear strength of the soils at the PFSF site degrades very little under repetitive cycles of dynamic loading, so that the levels of soil strain for the anticipated design basis seismic loadings are low. For that reason, strain-controlled cyclic triaxial tests are unnecessary, since they measure soil properties at high strain levels that will never be reached. See Findings 47-48.

The State also faulted PFS for failing to conduct triaxial extension tests to assess whether the soils are subject to bearing capacity failure due to tension loadings. PFS conducted triaxial compression tests to determine the possibility of soil bearing capacity failure due to compressive loads, but Dr. Bartlett asserted that if significant anistropy exists (that is, dissimilar shear strength of the soils in the horizontal and vertical directions), then the use of triaxial compression tests may overestimate the shear resistance along the potential failure plane. However, the anistropy of the PFSF soils is slight, not significant. This is demonstrated by the fact that the minimum vertical and horizontal shear strengths obtained by PFS in its tri-axial compression test for the pad emplacement area are almost identical. The soil failure mechanism is also a composite of failures along horizontal and vertical surfaces and is adequately represented by either the horizontal or vertical shear strengths. Therefore, the effects of anisotropy are insignificant and do not warrant performing triaxial extension tests. See Findings 49-52.

In addition, the concern behind seeking such tests is the potential for bearing capacity failure of the soils beneath the storage pads. PFS computed the minimum factor of safety against bearing capacity failure of the storage pads using many conservative assumptions. If these conservatisms in the analysis were removed, the minimum factor of safety against bearing capacity failure of the storage pads would be well in excess of 3. Given this large margin against bearing capacity failure, no need exists for triaxial extension tests. See Findings 52.

5. Conclusions re Soils Characterization

The ultimate issue with respect to soils characterization is whether the program implemented by PFS to determine the characteristics of the soils at the PFSF site provides reasonable assurance that the soil conditions are adequate for the proposed foundation loading, as required by 10 CFR 72.102(d). See Findings 56. The soils investigations performed by PFS are very comprehensive, and the results of the various investigations correlate well with each other and show remarkable consistency. In addition, the approach followed by PFS in establishing the strength and other characteristics of the soils is exceptionally conservative; for example, PFS used the least favorable measure of each property (e.g., shear strength) to represent the entirety of the site subsoil. See Findings 57-59. Therefore, the geotechnical site characterization information prepared by PFS and presented in the SAR is adequate to develop the design bases for the PFSF, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c, d) and 72.122(b).

C. Soil Cement and Cement-Treated Soil Issues

Soil cement is a material produced by blending, compacting and curing a mixture of soil, portland cement, other possible admixtures, and water to form a hardened material with far greater strength than that of the soil, and thus is used to increase soil strength. Some soil-cement mixtures are referred to as "cement-treated soils" because they have less cement content. Soil cement is typically expected to be able to pass durability tests that measure the ability of the stabilized soil to retain its properties after long periods of exposure to the elements. Cement-treated soil has less strength than soil-cement and is not expected to pass durability tests. See Findings 62-63.

PFS intends to use cement-treated soil in the area directly underneath the storage pads to create a cohesive material that will be strong enough to resist the sliding forces generated by the design basis earthquake. The cement-treated soil is intended to provide bonding with the bottom of the concrete pad above it and with the clay soils beneath, so as to transfer the horizontal earthquake forces downwards from the pad and into the underlying clay soils. Soil cement is to be used in the area around and between the cask storage pads to support the weight of the transporter vehicle that is used to deliver storage casks to the pad area. Soil cement is also to be placed around the Canister Transfer Building foundation mat to provide additional passive resistance against sliding forces in the event of a design basis earthquake. The design requirements for the soil cement and cement-treated soil at the PFSF have been established and are set forth in the SAR. See Findings 64.

1. "Proof of Design" Issues

The State claimed that the use of soil cement in the manner proposed at the PFSF is unprecedented and unproven, and argued that PFS should demonstrate by testing prior to licensing that it can develop a soil cement mixture that has the properties called for in the design. Soil cement has been used, however, for a wide variety of applications, including soil stabilization, in numerous instances, both in the United States and abroad. In particular, soil cement was used extensively to resist lateral forces and form permanent foundations for the five highway tunnels for I-90 and I-93 that converge at the Fort Point Channel crossing of Boston's Central Artery/Tunnel Project. This is essentially the same use of soil cement that is being proposed for the PFSF. See Findings 65-67. Moreover, there is no regulatory requirement that the suitability of soil cement for its intended use be demonstrated by case precedent. Therefore, the "newness" of the proposed uses of the soil cement at the PFSF raises no licensing issue. Indeed, Dr. Mitchell -- the State's soil cement expert -- attached no significance to the novelty of the application and indicated that new uses for soil cement are being developed all the time. Findings 68.

Further, all parties agreed that the design requirements for the use of soil cement at the PFSF can be met by the use of appropriate soil-cement mixtures. The State soil cement expert testified that he knew of nothing that would preclude PFS from meeting its design objectives for the soil cement program. See Findings 71-72.

All parties also agreed that PFS has developed a suitable program for testing the properties of the soil cement to demonstrate that the soil cement and cement-treated soil meets the design requirements specified by PFS. The parties agreed that this testing program is based on appropriate industry standards and includes the proper tests and suitable test methodology. There is also agreement that the test program, when implemented, will be effective in establishing whether the properties of the soil cement specified in the design have been achieved. The parties further agreed that the program to which PFS has committed is reasonable and should lead to proper soil cement and cement-treated soil installations. See Findings 73-77.

The sole point of disagreement between the Applicant and the Staff, on the one hand, and the State, on the other, was that the State believed that a test program to confirm that the soil cement will have the requisite properties should be completed before licensing, so as to provide "proof of design." The other parties did not think this is either required by NRC regulations or necessary. At the hearing, the State witnesses pointed out that adverse economic consequence could befall PFS if, after licensing, it was determined that the uses of soil cement proposed by PFS were unworkable and believed it would be "prudent" for PFS to demonstrate that the soil cement properties of its design were achieved before licensing. However, the State witnesses knew of no regulatory rule, regulation or regulatory guidance that requires that Applicant proceed with the soil cement testing program in advance of licensing. The Staff testified that, once the design requirements are established and a commitment is made to perform an appropriate testing program to demonstrate compliance with them, an applicant is free to defer testing to the post-licensing phase. See Findings 78-79.

From a licensing standpoint, the significant facts are that the design requirements for the soil cement and the cement-treated soil are well established. Additionally, PFS has committed to developing a soil-cement mix design using standard industry practices, and has further committed to performing a soil cement testing program in accordance with appropriate industry standards. PFS has specified the tests it intends to perform and the acceptance criteria that will be applied to the test results. PFS is also committed to performing field testing during construction to demonstrate that it has produced a soil cement with the required properties. The commitments made by PFS provide reasonable assurance that the soil conditions at the PFSF will be adequate for the foundation loading that will be imparted by the design basis earthquake, and no additional requirements need to be imposed on the Applicant prior to licensing.

2. Impact of Soil Cement Installation on Properties of Native Soils

The State raised concerns about the potential impacts that the installation of soil cement may have on the native soils at the PFSF site. The soil cement and the cement-treated soil to be used at the PFSF will be constructed by removing the topmost aeolian silt layer of soil at the PFSF site, and mixing it with cement in the appropriate proportions. The design requires that there be between a foot and two feet of cement-treated soil under each storage pad. In some spots it may be necessary to insert fill in below one or more of the pads to limit the cement-treated soil thickness to two feet. In such locations, PFS will use compacted native clay as fill.[9]

The State claimed that installing the cement-treated soil may disturb the native Upper Lake Bonneville clays that underlie the storage pads, and cause a reduction in the clays' strength. According to the State, this may occur from exposure of the soil to the elements or through deformation ("remolding") by construction equipment. Findings 83, 87-8.

PFS, however, plans to minimize exposure to the elements by the use of appropriate construction procedures and scheduling. Remolding of the native soils under the storage pads will be addressed by using construction equipment that can be located on either side of the pads and by keeping all other construction equipment off of the exposed subgrade. PFS intends to demonstrate at the start of construction that the techniques to be used by the contractor that will install the soil cement will not have an adverse impact on the strength of the underlying soils. See Findings 84-86, 89.

The State has also posited that the concrete pads and the cement-treated soil underneath them may serve as an impermeable barrier that will trap moisture in the underlying soils and weaken them. This, however, will not happen because the storage casks on top of the pads will provide a source of heat that will be transmitted downwards through the concrete pad and the cement-treated soil, causing any moisture that may be present in the soils beneath the pads to migrate to the surrounding areas due to heat gradient effects. See Findings 90.

[9] The State expressed the concern that the compacted clay may be weaker than undisturbed soil. PFS witnesses testified, however, that PFS intends to compact the fill soil to such a degree that the resulting recompacted clays are at least as strong as the undisturbed clay present at the site.

The State also expressed more general concerns about potential water infiltration into the subsoil at the PFSF site –via cracks in either the cement treated soil or soil cement. However, PFS plans to install berms around the pad emplacement area to direct any surface water away from it and within the pad emplacement area, the site is generally sloped from south to north and from the center of the site to the edges, where there are concrete lined drainage ditches to transport the surface water to the detention pond at the north. Also, any water that enters through a crack in the soil cement will be unlikely to penetrate all the way down to the underlying soils because the soil cement will be constructed in thin lifts, and it is very unlikely that the cracks on each lift will line up exactly with the cracks on other lifts. Moreover, any moisture that infiltrates into the soil will tend to evaporate because of the site's arid conditions and any moisture accumulation and attendant potential reduction in the shear strength of the soil would only be a localized phenomenon, which would not have a significant effect on the strength or bearing capacity of the soils underlying the storage pads or the CTB. Finally, tests performed on the soils at the PFSF site demonstrate that the strength of the soils is only minimally affected by an increase in water content. See Findings 91-95.

3. Cracking and Other Mechanisms Potentially Degrading Soil Cement Performance

The State asserted that the performance of the soil cement and the cement-treated soil will be degraded by cracks that will form in the soil cement over the life of the PFSF facility. The main direct consequence of any cracks that form would be the potential infiltration of moisture into the soil beneath the soil cement that surrounds the pads and the CTB. However, as just discussed, water infiltration -- if it were to occur -- is not expected to have a significant adverse impact on the performance of the soil cement, the cement-treated soil, or the underlying native soils. Finding 97.

Another consequence of crack formation, according to the State, is the potential reduction in the tensile strength of the soil cement. However, PFS does not rely on the tensile strength of the soil cement, so the effect, if any, of such cracking is inconsequential. See Findings 98.

4. Young's Modulus Issue

The State also raised two specific issues with respect to the Young's Modulus of the cement treated soil.[10] The first is whether the design requirements imposed by PFS, i.e., a minimum compressive strength of 40 pounds per square inch ("psi") and a maximum Young's Modulus of 75,000 psi, can be successfully implemented.

Seeking to have a compressive strength in excess of 40 psi while limiting the Young's Modulus to less than 75,000 psi is achievable because having cement-treated soil with relatively low strength and relatively low modulus is consistent with the anticipated performance of soil cement and cement-treated soil and with data reported in the literature. See Findings 117-118.

The second issues is the States claim that the 75,000 psi Young's Modulus that is to be achieved to comply with the requirements of the Holtec cask drop analysis should be a dynamic rather than a static modulus. However, the issues of whether the 75,000 psi Youngs Modulus is static or dynamic is one of semantics and of not concern here. The PFS design has specified the parameters and which the 75,000 psi Young's Modulus is to determine specifically if it is to be determined as to soil strain of 1.93%. While that soil strain corresponds to that obtained in static testing, the important consideration is that the

[10] The Young's Modulus of a material measures the stress to strain ratio, that is, how much the material deforms as a function of the stress applied to it. For purposes of the issues in this proceeding, the vertical Young's Modulus is the parameter of interest.

maximum modulus requirement be shown to be satisfied for the specified soil strain. PFS intends to demonstrate this by appropriate testing.

5. Conclusion re Soil Cement Issues

Implementation of the program that PFS has developed for the testing and construction of soil cement and cement-treated soil will lead to the installation of soil cement and cement-treated soil mixes that will meet the specified design requirements and will give adequate performance for the life of the PFSF. Concerns about the potential impact of soil cement installation on the native soils, and about the potential degradation of soil cement performance with time, are unfounded or inconsequential.

The State's expert has agreed that a Young's Modulus determined at a soil strain of 1.93% would meet the design intent. PFS has made a licensing commitment to demonstrate by testing that the cement-treated soil to be placed under the storage pads satisfies this design specification. No further showing needs to be made at this time.

D. Dynamic Analysis Claims

1. Introduction

The claims in Section D of Contention L/QQ are aimed at the dynamic analyses performed by PFS. The gist of the State's case is that there are errors, improper assumptions, and other deficiencies in the analyses that render them unreliable, and thus PFS "has failed to demonstrate that the structures and their foundations have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake, and does not satisfy 10 CFR § 72.102(c) or (d) or § 72.122(b)(2)."

In addition to the specific claims alleged in Section D of the contention, State witnesses Drs. Ostadan, Khan and Bartlett raised some general concerns in their prefiled direct testimony and at the hearing. Those concerns can be summarized as follows: (1)

The non-linear analyses of the stability of the casks under seismic loadings are extremely sensitive to the choice of input parameters, and since the actual values of those parameters have not been established by shake table testing, the results of the analyses obtained by Holtec cannot be relied upon. (2) The seismic design of the casks and storage pads - which leaves the casks unanchored, allows the pads and the casks to slide in an earthquake, and relies on the use of soil cement and cement-treated soil to improve subsurface conditions - is "unique, unprecedented and unproved." (3) The PFS seismic design lacks sufficient margin to accommodate the potential impact of deficiencies such as those raised by the State. We will discuss these general concerns first and then address the specific deficiencies raised by the State witnesses.

2. Issues Relating to the Cask Stability Analysis

a. Introduction

At the PFSF, spent nuclear fuel will be stored in large storage casks resting on concrete pads. The storage cask system to be used by PFS is the Holtec International HI-STORM 100 Storage Cask System ("HI-STORM System"). The HI-STORM System consists of a massive cylindrical steel and concrete storage cask surrounding a multi-purpose stainless steel canister in which the spent nuclear fuel is sealed. The multi-purpose canister ("MPC") in which the spent fuel is sealed at the shipping nuclear power plants is a rugged cylindrical container stored vertically within the storage cask. It is designed to meet the applicable provisions of Subsection NB of the ASME Code. Finding 130.

The HI-STORM System storage casks will be placed on an array of concrete pads. Each pad will be sized to accommodate a 2 x 4 array of casks and will be capable of supporting eight loaded storage casks. The cask storage pads will be independent structural units constructed of reinforced concrete, each pad being 30 ft wide, 67 ft long and 3 ft thick. At maximum capacity the facility would contain 500 such pads, each supporting eight loaded storage casks. The massiveness, stable configuration and sturdy construction of the HI-STORM System cask and MPC provide assurance that they will be able to withstand very large loadings without release of radioactive material. Finding 131.

b. Holtec's Cask Stability Analyses

i) Description of analyses and their results

Holtec performed seismic analyses for the HI-STORM System to be used at the PFSF using its specially developed computer code, DYNAMO. This code has been used by Holtec to perform the seismic analyses in its Safety Analysis Report for the HI-STORM System which supports the Certificate of Compliance that the NRC has issued for the HI-STORM System under 10 C.F.R. Part 72. Holtec has also performed site-specific seismic analyses using DYNAMO for ISFSIs featuring HI-STORM System storage casks on concrete storage pads at five nuclear power plant sites. In addition, Holtec has extensive experience in using DYNAMO for the seismic analysis of spent fuel racks used to store spent fuel in nuclear power plant spent fuel pools. Findings 137-138.

In order for DYNAMO to be approved by the NRC for use in these licensing analyses, the code had to be validated to demonstrate that it produces acceptable results for the class of problems for which it is used. A series of classical problems having known solutions were modeled using the code and were shown to give results in good agreement with the analytical results. In addition, problems that had no simple analytical solutions were also evaluated and shown to give good agreement with numerical solutions using other industry codes such as ANSYS. Finally, some features of DYNAMO were validated by comparing results from experiments with those obtained through the use of DYNAMO. During the course of various license submittals, DYNAMO was also subjected to additional validation at the request of NRC's reviewers. All of these validations were successful. Finding 139.

Data characterizing the earthquake excitation and the soil response for the PFSF site (soil properties used to characterize the soil springs and dampers, and acceleration time histories) were provided to Holtec as design inputs by Geomatrix. Holtec then computed the values of the spring constants and damping coefficients for use in its analyses using the soil property values supplied by Geomatrix. For the design basis analysis, Holtec modeled various configurations of one to eight casks on the concrete pad using lower bound, best estimate and upper range soil properties. To model the effect of friction between the cask and pad, Holtec used an upper bound coefficient of friction of 0.8 at the cask/pad interface (to emphasize or increase the likelihood for cask tipping) and a lower bound coefficient of friction of 0.2 (to emphasize or increase the likelihood of cask sliding). Findings 141-143.

Nine cases were run for the upper bound coefficient of friction of 0.8, and one case was run for a lower-bound coefficient of friction of 0.2. For the 2,000 year design basis earthquake, the Holtec analyses showed a maximum displacement of the cask on the order of 3 to 4 inches with a corresponding maximum angle of tilt of 1.026 degrees, which provides a factor of safety against cask tipover of 28.6 when compared to the angle of tilt at which a cask would tip over from the moment of its own weight. The case evaluated for a coefficient of friction of 0.2 produced a maximum sliding displacement on the order of 2 inches. Findings 144-145.

In addition to its design basis cask stability analyses using DYNAMO, Holtec performed several beyond design basis cask stability analyses, most of which were for a 10,000-year return period earthquake. For these analyses, Holtec used the VisualNastran

("VN") computer code because DYNAMO is a small deflection program, and can not accurately model the large cask rotations or displacements that could occur in a 10,000-year earthquake. The simulations of the 10,000-year beyond design basis earthquake showed some instances of cask rotations on the order of 10-12 degrees. Even with such large rotations, the casks have a safety of factor on the order of 2 or more when compared to the angle of tilt (29.3 degrees) at which a cask would tip over from the moment of its own weight. Findings 146, 148.

ii) State challenge to appropriateness of the use of DYNAMO

Dr. Khan, on behalf of the State, questioned the suitability of the DYNAMO code for the PFSF 2,000 year design basis earthquake stability analysis, given that DYNAMO is a small deflection program not capable of handling large cask rotations. However, the Holtec analyses show that in the event of a design basis earthquake the casks will only undergo small rotations, well within the code's computational capabilities. Finding 152.

Dr. Khan also claimed that Holtec had not validated its DYNAMO results for the 2,000-year DBE at the PFS site with another structural analysis code such as Visual-Nastran and therefore could not determine whether DYNAMO had provided erroneous results. In fact, Holtec ran one of the nine configurations of the original design basis analysis on VisualNastran. The VisualNastran run predicted small cask displacements similar to the DYNAMO results, thus validating the DYNAMO analyses and showing that the DBE analysis was within the DYNAMO code's capabilities.

iii) Contact stiffness

Dr. Khan also challenged Holtec's choice of contact stiffness for its analyses, particularly the vertical contact stiffness. Vertical contact stiffness represents the amount of force applied at the interface points of contact between two bodies that would be required to have one of the bodies approach or penetrate the other a unit distance. For its cask stability analysis for the 2,000 year return period DBE, Holtec used a vertical contact stiffness of 454 x 10^6 lbs. per inch. Findings 154-155.

Dr. Khan claimed that Holtec's chosen contact stiffness is too high and results in making the vertical frequency of the cask too rigid, thus artificially reducing the vertical displacement because the code will treat the cask as if anchored to the pad. According to Dr. Khan, a contact stiffness should be chosen that corresponds to cask frequencies that fall in the amplified spectral range of the earthquake input spectra. Therefore, he argued that a more appropriate contact stiffness value for unanchored casks would be in the range of 1 x 10^6 lbs/inch to 10 x 10^6 pounds per inch. This conclusion was based on Dr. Khan's belief that it is impossible to choose a contact stiffness based on physical principles because the contact stiffness changes throughout an earthquake event. Therefore, unless one had obtained shake table test measurements of contact stiffness, one must conservatively choose a value that tunes the natural frequency of the cask to the amplified range of the earthquake response spectrum so as to maximize vertical excitation of the cask. Finding 155.

Dr. Khan's approach artificially maximizes the vertical response of the cask by assuming that the natural frequency of the cask oscillating or vibrating on the pad and the earthquake are in resonance. Also, since the amplified spectral range of an earthquake will vary depending on the geology and soils of its location, setting the contact stiffness so as to cause the cask and the earthquake to be in resonance means that the choice of contact stiffness will vary depending on the geographic location of an ISFSI, a result that is contrary to logic and physical reality, because contact stiffness is a physical parameter of the contacting objects and their intrinsic material properties. Findings 151-157.

Drs. Singh and Soler testified that in performing computer analyses it is often wise to choose a lower value of contact stiffness than the actual one to avoid excessive computing time, but one should always avoid using such a low value that the corresponding cask frequencies fall into the amplified spectral range of the earthquake spectra. If that is done (as proposed by Dr. Khan), the results of the analysis will be contaminated by introducing a non-existent excitation of the cask, since the actual physical contact stiffness of the cask-pad interface does not produce cask frequency responses in the amplified spectral range of the earthquake. Finding 158.

Since contact stiffness is a physical parameter of the contacting objects and their intrinsic material properties, it can therefore be derived from nature's physical laws. Holtec computed the vertical contact stiffness of 454×10^6 lbs. per inch for its DYNAMO design basis analysis using a well established, state of the art methodology for calculating the contact stiffness between two objects. When contact stiffness is thus computed, it does not vary from one geographic location to another as the earthquake characteristics change. This is an appropriate result, as opposed to the site variability that would result under the approach espoused by Dr. Khan. See Findings 164-166.

There are also simple mathematical relationships between the natural frequency of the cask under dynamic conditions, the static deflection of the pad caused by the cask resting on its surface, and contact stiffness. According to those relationships, the natural frequency of the cask is a function of the static deflection of the pad caused by the cask resting on its surface, or in other words, the static contact stiffness. See Findings 167-171, 173.

The existence of those simple relationships is highly significant because: (1) it refutes Dr. Khan's assertion that neither static deflection nor contact stiffness derived under static conditions have relevance to the dynamic analyses; (2) it demonstrates that, since the natural frequency of the cask is controlled by the physical characteristics of the cask-pad interface and not the incoming earthquake excitation, it is incorrect to adjust the contact stiffness to tune the natural frequency of the cask to the external earthquake excitation; (3) it allows one to ascertain whether the amplified range of the response spectral curve corresponds to a physically realistic deflection that would be seen in the real world; and (4) it allows one to ascertain what the natural frequency of the cask is. The natural frequency of the HI-STORM 100 cask on the pad of 111 hertz, a frequency far above the range of the earthquake input spectra and the frequencies of interest for seismic earthquake analysis, which are well below 50 hertz. See Findings 174-175. The ability to thus determine the natural frequency of the cask demonstrates the fallacy of artificially choosing a contact stiffness that would cause the cask to be in resonance with the much lower frequencies of the amplified spectral range of the earthquake.

iv) Damping

In its cask stability analysis for the 2,000 year earthquake, Holtec used a 5% value of damping at the cask-pad interface to represent the dissipation of energy that occurs when the cask impacts the concrete pad during an earthquake event. Impact damping is the loss of energy associated with the impact of two bodies, such as that of a ball bouncing on a rigid surface. Findings 183-184.

Impact damping differs from structural (or "material") damping, which is the damping or loss of energy associated with the deformation of structures and materials. Holtec did not take credit for structural or material damping of the cask, canister and their internals in its cask stability analysis. Holtec similarly did not take credit for the damping associated with internal impacts within the cask and canister. Holtec's analysis was thus conservative in that it neglected all damping associated with the cask, except that due to impacts between the cask and the pad. Finding 184.

Dr. Kahn took issue with Holtec's use of 5% damping in its DYNAMO model for the 2,000 year design basis earthquake. Dr. Kahn claimed that the "results" of his cask stability analyses show that use of 5% damping significantly reduces the estimated cask response, and in reality only friction should be the primary energy dissipation mechanism. Finding 186.

Dr. Khan's report specifically refers to the damping that he utilized in his modeling of cask stability as "structural damping." Dr. Khan further confirmed at the hearing that he utilized structural damping associated with the "whole structure" in his analyses of cask stability. Thus, Dr. Khan used structural damping in his analysis while, as noted above, Holtec did not. See Finding 188.

Dr. Khan's assessment of Holtec's modeling was based on the mistaken assumption that Holtec used structural damping in its modeling, which led Dr. Khan to erroneously criticize the appropriateness of the damping values used by Holtec. In reality, the 5% impact damping value used in the DYNAMO cask stability analyses is a conservative choice to represent impact damping between the cask and the pad. See Findings 189-192.

To illustrate the reasonableness of the impact damping values used in Holtec's cask stability analysis, Drs. Singh and Soler provided computer simulations showing the effect of impact damping on a ball or cask dropped from a height of 18 inches using impact damping values of 1%, 5% and 40%. At 1 percent damping, which is the value that Dr. Khan would have Holtec use, the ball or cask would require more than 73 bounces before it came to rest; at 5% the ball or cask would come to rest after approximately 14 bounces and at 40% the ball or cask would come to rest after 2 or 3 bounces. See Finding 193. Clearly, a 1% value of impact damping is unreasonable.

c. *Cask Stability Analyses Performed by Sandia Laboratories for the Nuclear Regulatory Commission Staff*

The NRC commissioned Sandia Laboratories to perform a confirmatory analysis of the behavior of the Holtec cask under the design-basis 2,000-year return period seismic event and under the beyond-design basis 10,000-year return period seismic event. Instead of using soil springs, the Sandia model used a complex, finite element representation of the soil cement/soil foundation and extended the foundation boundary well beyond the pad boundary. The model accurately represented all the components of the cask pad system (storage cask, reinforced concrete storage pad, soil cement, cement-treated soil, and six-layer soil foundation to a depth of 140 feet), took into account a variety of soil property parameters (upper bound, lower bound, and best estimate), used conservative estimates for the Young's modulus of the cement-treated soil, used bounding conditions for three different interfaces (e.g., storage cask and pad, storage pad and cement-treated soil, and cement-treated soil and soil foundation), and used three different sets of time histories (representing the 2,000-year design basis earthquake, the 10,000-year beyond design basis earthquake and the Pacoima Dam record for the 1971 San Fernando earthquake) in order to envelope all potential site conditions. Additionally, the Sandia analysis examined two variants of the model (one without any soil cement or cement treated soil and the other with the dead load weights of fully-loaded neighboring storage pads and the seven additional casks on the target pad) to examine the potential affects of pad-to-pad interaction and the behavior of a cask absent the soil cement. Findings 198-201.

This comprehensive analysis, combined with sensitivity studies for key components of the cask/pad system, such as the soil cement, demonstrated that a cask on a pad at the PFSF ISFSI would not tip over, collide, or uplift under any possible scenario for either the 2,000-year return period ground motions or the Pacoima Dam record. Likewise, for a 10,000-year return period ground motion, no set of conditions resulted in cask tipover or impacts from cask sliding. The maximum sliding displacement of any cask during any 10,000-year event simulation was 15.98 inches, which would still not result in a cask colliding with another. Likewise, the maximum cask rotation under the 10,000-year return period was 1.16 degrees, representing roughly a factor of safety of 25 against overturning. The only uplift (i.e., the cask base being entirely lifted off the surface of the storage pad) that resulted from any assumed set of conditions at the PFSF was a predicted cask uplift during a 10,000-year return period ground motion of 0.26 inches for less than 0.30 seconds. Findings 208-211.

While the State raised questions about some of the input parameters for the model used in the Sandia analyses, these questions were the result of the State's misunderstanding of the finite element modeling methodology employed. For example, the State witness did not understand how the interfaces were modeled and assumed that using a coefficient of friction at the interface modeled the materials as frictional would allow the pads to slide contrary to PFS's design. However, the use of the coefficient of friction at the interface is a well-established method of modeling interfaces between two materials for finite element modeling, and allowed the analyses to envelope all possible conditions in the Sandia analyses. Further, the soil properties, including cohesion, wee incorporated into the finite elements used in the model and the actual results showed displacements well within the elastic range of the materials being modeled. Hence, the concerns of the State witness were not borne out. Both the model and the computer code on which it ran were state-of-the-art and provided independent confirmation of the results of the Holtec analyses using an alternate, but equally well-established methodology for modeling cask stability and seismic response. Findings 212-217.

d. *State's Cask Stability Analysis*

In addition to criticizing the Holtec analysis, Dr. Khan performed an analysis of his own utilizing the computer code SAP 2000 and a model of the cask that he developed. Dr. Khan conducted several analyses using the SAP2000 computer code, in which he sought to show the effect of changing various parameters (contact stiffness, coefficient of friction, and damping) that may bear upon the movement of a HI-STORM System cask on a storage pad during a seismic event. He varied the coefficient of friction as Holtec had done, using coefficient of friction values of 0.20 and 0.80 for the cask-pad interface. For the values of the vertical contact stiffness and damping he chose a wide range of values seeking to show what effect changing of these parameters would have on the movement of the cask on the concrete storage pad. Findings 218-220.

The cask displacements predicted by Dr. Khan's analysis ranged widely from a few inches to many feet. One run showed the cask lifting up of the surface of the pad by more than 2 ft. and moving laterally on the order of 40 ft. Dr. Khan did not claim that the results of this or any other of his runs were a "correct" or "realistic" prediction of what would occur at the PFSF under earthquake conditions, but asserted that the purpose of his runs was to show the wide variability of the results that could occur from choosing different parameters at the cask-pad interface to model movement of the casks on the pad. Similarly, the State's expert Dr. Ostadan, who testified with Dr. Khan, readily acknowledged that he did not believe the results of Dr. Khan's analysis, particularly that the casks would lift 2 ft. up in the air and move 40 feet under earthquake conditions. See Findings 221.

Perhaps the reason Dr. Khan's analysis gave erratic results is that his model used improper parameters. As discussed above, Dr. Khan chose a vertical contact stiffness of 1×10^6 lbs. per inch. Holtec determined that using such a low value of vertical contact stiffness results in a deflection of 3/8 of an inch in the pad simply from having the cask rest on its surface. This is totally unrealistic, since the pressure applied by a fully loaded cask on the reinforced concrete pad is 26 lbs. per square inch, equivalent to a man standing on one foot. Drs. Soler and Singh rightfully pointed out that a model should be able to provide a physically correct answer for all conditions, including the initial static case, as well as under dynamic loading. Dr. Luk agreed. See Findings 164-179, 239.

Additionally, for computer modeling purposes one needs to use a horizontal stiffness for the friction spring at the cask pad interface that must be overcome for sliding to occur. The experts for both parties agreed that horizontal stiffness is a mathematical artifice, required only by the algorithms of the computer numerical analysis in order to arrive at a solution. However, the horizontal stiffness "should be selected high enough to minimize elastic contact" or movement. For his analysis, Dr. Khan choose a contact stiffness of 100,000 lbs/inch. This value was unreasonably low and caused Dr. Khan's model to generate unrealistic predictions for cask sliding, such as a 0.72 inch horizontal displacement of the cask just prior to the actual initiation of cask sliding. Such a prediction is not consistent with reality. Findings 177-178.

Also, as noted above, Dr. Khan used structural damping in his model, and doing so was also incorrect. Structural damping is significant for structures and components that are anchored. The principal mode of damping for a free-standing structure, however, is impact damping, not structural damping, and different physical principles and analytical methods apply to both. This is reflected in the computer modeling. Since Dr. Khan mistakenly regarded structural damping as applicable, he included damping in all the stiffness elements in his model. Dr. Khan acknowledged that had he not done so, friction would be the sole energy dissipation mechanism for cask sliding. Findings 189-192.

Finally, Dr. Khan's use of the SAP2000 program to simulate the motions of the casks at the PFSF was not a good choice. SAP 2000 is designed to be used for structural systems which are primarily linear elastic. It does not appear, however, that SAP2000 can model significant non-linearities such as those involved in large motions of the casks under dynamic loadings. In this respect, Holtec was unable to duplicate the results of Dr. Khan's SAP 2000 model using Visual Nastran, which all parties acknowledge can handle large deflections. The inappropriateness in the use of SAP2000 is evidenced by the results of Dr. Khan's analysis, in particular runs 1 and 3 of his third model, which show casks lifting off the ground by one or two feet and moving laterally 30 to 40 feet. The combination of inappropriate values of vertical and horizontal contact stiffness, an incorrect modeling of damping, and the use of a computer code not suited for the assumed conditions, yielded results that are unreliable. Dr. Khan's analysis is therefore not credible. Findings 227-238.

e. Conclusions re Cask Stability Analyses

The various cask stability analyses conducted by the Applicant, Sandia, and the State illustrate the safety of the PFSF seismic design, the importance of experience in modeling freestanding structures, and the necessity of applying a fundamental understanding of the underlying physics in order to develop models that produce meaningful results. Both Holtec and Sandia constructed models of the cask and storage pad system using well-established modeling methodologies that appropriately depicted the properties of the materials used in fabricating the storage cask and pad system at the PFSF, including realistic site conditions for soils. These two methodologies produced results that were both internally consistent across a variety of parameters and were also consistent with one another. In neither set of analyses was their the potential for cask sliding and collision, tipover or uplift for the 2,000 year design basis ground motions. In developing these models, both teams – Holtec and Sandia – had the resources of extremely experienced individuals who had modeled numerous storage cask stability simulations. Drs. Singh and Soler for Holtec have decades worth of experience in understanding the mechanics and dynamics of storage cask behavior. Dr. Luk at Sandia, with the assistance of a variety of other experts, had been involved in a cask modeling project for several years prior to the PFSF cask stability issues arose, and has conducted a large-scale generic cask stability analysis, as well as several site specific cask stability analyses. Findings 20-241.

By contrast, the only analysis that purports to demonstrate flaws in the Holtec analyses was conducted by Dr. Khan, whose only experience in finite element modeling has been in the modeling of components that are not freestanding, but bolted or otherwise affixed to another surface. He had virtually no prior experience in modeling freestanding structures or in modeling their behavior. This lack of experience is reflected in the anomalous results of his analysis, which were neither internally consistent nor logical, but for some cases produced reasonable appearing results and for others produced results which he himself admitted were not indicative of anything that could really happen. Rather than seeing that as a defect of his model or the computer code he used to run the model, he merely assumed that the Holtec analyses had failed to take into account the effect of varying contact stiffness, even though his contact stiffness values were obviously absurd from a real world standpoint (predicting a cask penetration of the storage pad of three-eighths of an inch) and his model could not replicate results of known classical problems. Findings 242.

During the course of the hearings, Holtec performed numerous analyses that addressed each of the conditions and parameters that the State had claimed were not adequately considered. The results of these confirmatory analyses were uniform and consistent: the storage casks do not tip over, even during a 10,000-year return period ground motion, no matter how the values of contact stiffness, damping or other parameters were changed. This demonstrates the ability of the PFSF cask/pad configuration to withstand design basis (and beyond) seismic loadings without cask tipover. Findings 243.

3. State Claims re Newness of Proposed Seismic Design of Storage Pads and Casks

In their prefiled direct testimony on Section D of Contention L/QQ, State witnesses Drs. Bartlett and Ostadan identified as a primary purpose of their testimony to demonstrate that the PFSF seismic design is "unique, unprecedented and unproved." At the hearing, however, the evidence showed that the features the State witnesses identified as unique are not so. The PFSF is not the first or only away-from-the-reactor spent fuel storage facility, not the first or only facility to deploy unanchored HI-STORM storage casks. The PFSF is not the first or only nuclear facility that deploys unanchored safety-related equipment, and is not the first or only facility to use shallow concrete pads. To the contrary, the use of unanchored casks restring on concrete pads is a conventional design.[11] And, as discussed above, the use of soil cement to provide foundation stability is neither new nor unique to the PFSF. Findings 244-46.

The State witnesses also identified as a "unique feature" of the PFSF design that it utilizes a "controlled sliding" design concept for the HI-STORM Holtec storage casks under which "the casks will be allowed to slide and such sliding will occur in a uniform and controlled manner without collision or tipping." These assertions were shown at the hearing to be incorrect. Dr. Ostadan acknowledged that the storage cask design does not "control" sliding, but merely allows it to occur, and the fact that sliding may occur in a

[11] The State has made reference to 500 casks being stored at the PFS. However, from the analytical standpoint, having 500 pads at a site as opposed to a few makes little or no difference.

"uniform and controlled manner" is not a design requirement, but the result predicted by the cask stability analyses conducted by Holtec. See Findings 247-48.

In addition, the State asserts that PFS uses pad sliding as a mechanism to reduce the seismic loading transmitted to the casks. It is true that, if pad sliding occurs, it has the beneficial effect of reducing the seismic loading to which the cask is subjected. However, to the extent that such effect occurs, it is again a consequence of the earthquake dynamics and not a design feature or mechanism. Thus, there are no "unique" features of the PFSF facility that represent deficiencies, render the design unconservative, or require special scrutiny. See Findings 249-251.

4. State Claims re Alleged Lack of Margin in the Design

Drs. Bartlett and Ostadan also asserted that narrow margins exist in the PFSF design, from which they concluded that what might otherwise be minor deviations or inconsistencies in the analysis might be sufficient to create unintended consequences or to result in failure of the safety-related structures at the PFSF. At the hearing, the State witnesses reaffirmed that their concerns should be viewed against the backdrop of their opinion that the PFS seismic design lacks sufficient margin to accommodate the potential impact of the deficiencies they allege. Dr. Ostadan indicated: "a lot of my comments would not be here if you had a large margin." See Findings 252-254.

This assessment of their own claims by the State witnesses is important because the PFS testimony demonstrates that significant conservatisms are incorporated into the analysis of the foundations of the CTB and the storage pads, and the actual margins against the mechanisms for potential foundation failure are rather large, not small as the State alleges. There is also ample testimony in the record that the designs of the CTB, the casks and the storage pads incorporate such wide margins of safety that even if a failure of the foundations took place there would be no adverse safety consequences. See Findings 255-256.

Since the State has essentially conceded that many of its concerns would be "nit-picks" if there was adequate margin in the design, and given that PFS has demonstrated the existence of significant margins in the designs of the CTB, the storage casks and pads, and their respective foundations, the State needs to make some showing that the specific deficiencies it postulates rise above the level of second or third order effects and would have a significant adverse impact on safety. The State has made no such showing, and in fact its witnesses acknowledged that they conducted no calculations or any other analyses to substantiate and quantify their concerns. Finding 256.

5. Overview of PFS Stability Analyses and State's Concerns

Many of the State's concerns center around two seismic stability analyses prepared by Applicant's consultant Stone & Webster, i.e., Calculation Nos. 05996.02-G(B)-04, Rev. 9, *Stability Analyses of Cask Storage Pads* (July 26, 2001) ("Cask Storage Pad Stability Calc. Rev. 9") ("G(B)-04"), and 05996.02-G(B)-13, Rev. 6, *Stability Analyses of Canister Transfer Building* (July 26, 2001) ("CTB Stability Calc. Rev. 6") ("G(B)-13"). PFS Exhibits UU and VV. In these analyses, PFS sought to evaluate three potential "failure modes" for the structures: sliding stability, overturning stability, and bearing capacity stability. Sliding "failure" occurs if the structure moves horizontally, parallel to the ground. Overturning "failure" occurs if the structure rotates as a rigid body about a horizontal axis. Bearing capacity "failure" takes place if the soils beneath the structure become overloaded in the vertical direction, leading to settlement or rotation of the structure's foundation. The fact that these "failures" occur does not necessarily have in itself adverse safety consequences. Finding 257.

For each type of failure mode, the stability analyses include a "base case" that reflects the design intent with respect to the soils and foundations, and which considers the pertinent combinations of horizontal and vertical seismic loadings. In addition, the stability analyses also include hypothetical, "what if" scenarios, in which stability analyses are conducted for various conditions and combinations of earthquake loadings. Finding 261.

The intent of the seismic stability analyses is to establish what margin of safety or "factor of safety" ("FS") is provided by the design of the structures' foundations against each of these failure modes. It is typical in the industry to use FS = 1.1 as the desired safety factor against each of the three above mentioned failure modes. Use of such a safety factor is, however, not required by NRC regulations, and an applicant may gain regulatory approval by showing that the structures, systems and components perform their safety functions when subjected to seismic loadings, whether or not they meet the factor of safety guidelines. Finding 258.

Failure to meet the factor of safety of 1.1 against one of the postulated failure modes does not mean that the failure mode in question will occur. It is only when the results of the analysis predict a factor of safety of less than 1.0 that the failure mode in question is possible. Even then, however, numerous conservatisms are incorporated into the design of the structures and into the stability analyses mean that even if the calculated factor of safety is less than 1 the structures most likely would not experience the failure mode in question during the seismic event. Finding 259.

Of the three potential foundation failure modes, the one of most concern to the State is that the structure in question will fail to satisfy the 1.1 factor of safety against sliding in the event of a design basis earthquake. However, there is no dispute that in the event the CTB experiences sliding there will be no adverse safety consequences because the building is not connected to any safety-related components such as electrical or piping lines that may be damaged if sliding occurs. See Findings 263-264. Similarly, there will also be no adverse safety consequences to the sliding of the pads because there are no safety-related components connected to them. In fact, if pad sliding occurs, such sliding reduces significantly the seismic loading to which the cask is subjected. See Findings 265. Therefore, pad sliding is beneficial to the stability of the safety-related component of concern, i.e., the storage casks.

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

a. Pad Flexibility

The State alleges in Section D.1.b of Contention L/QQ that PFS's dynamic analyses for the storage pads incorrectly assume that the pads will behave rigidly during the design basis earthquake. The State asserts that this incorrect assumption leads to significant underestimation of the dynamic loading atop the pads, especially in the vertical direction, and overestimation of foundation damping.[12] Findings 266-267.

International Civil Engineering Consultants, Inc. ("ICEC"), the storage pad vendor, performed a detailed calculation for the design of the reinforced concrete pad on which the storage casks will be placed. As part of this calculation, ICEC computed the maximum displacements of the pad in the vertical direction at various nodes in the pad and determined that the maximum deviation of local displacements from rigid body motion for the pad is of the order of approximately 1/8 of an inch. Such a small local displacement would produce only secondary effects on the global dynamic response of the pad/cask system and would not affect the stability of the casks. See Finding 268.

[12] Foundation or radiation damping is the ability of structures to reflect back ("radiate") into the soil a portion of the energy imparted upon the structures by the seismic excitation. A rigid structure is most efficient in radiating energy back into the soil; a flexible one is less efficient, with the efficiency decreasing as the flexibility of the structure increases.

At the hearing, Dr. Ostadan asserted that what is significant is not so much the amplitude of the non-rigid displacements but the relative motion of various points on a pad with respect to each other, so that if there is a "rippling effect" in the displacement of the pad, this effect will tend to decrease the radiation damping available. However, a plot of vertical displacements on a pad as a function of location shows that the displacement along the pad is virtually zero for most of the length of the pad and there is one single, gradual, small vertical displacement of the pad at the point of application of the seismic loading, which slowly decreases as one moves away from the point of application of the seismic force. These results show the absence of "ripples" of the type of concern to Dr. Ostadan, and demonstrate the rigid behavior of the pad under dynamic seismic loadings. See Finding 269.

PFS also performed a numerical evaluation of the effects of pad flexibility on the properties of the foundation. This evaluation demonstrated that the effect of flexibility on the foundation damping properties of the pad is insignificant in the frequency range of importance to the cask response. Likewise, the computer analyses conducted by Sandia Laboratories for the NRC Staff incorporated pad flexibility and yielded very small cask displacements under seismic loadings. Further, Holtec performed at another facility parametric studies that assessed the stability of the casks assuming a rigid versus a flexible pad and determined that the differences in the two cases were minor. These various analyses show that the effects of pad flexibility on the dynamic behavior of the casks in a seismic event is negligible. Finding 270.

b. Frequency dependence of spring and damping values

In Section D.1.e of Contention L/QQ the State alleges that "Applicant's calculation for cask sliding do not address the frequency dependency of the spring and damping values used to model the foundation soils." The State acknowledged that the Holtec cask stability evaluations needed to be non-linear, time-dependent analyses and therefore Holtec could not model the spring and damping as frequency dependent parameters. However, Dr. Ostadan claimed that Holtec should have used a damping value that corresponded to the fundamental frequency of the pad. He was unaware of what frequency should have been used, to what frequency the damping value used by Holtec corresponded, or the extent of the error, if any, in Holtec's assumed value of damping. See Findings 272-273.

Use of the approach proposed by Dr. Ostadan would require knowing the system frequency beforehand; however, due to the non-linear response of the casks caused by sliding and tipping, the predominant frequency of the cask/pad/soil system's response to the seismic input is not unique, but shifts as the casks move on the pad. Finding 274.[13]

Moreover, in its analyses, Holtec selected the soil mass, spring and damping parameters using formulae published in a well-recognized technical treatise. The chosen combination of parameters produces approximate frequency-dependent foundation impedance functions that cover the frequency range important to the cask response. Use of this method, coupled with the use of three sets of soil properties ensures that a sufficiently broad range of frequencies of the cask/pad/soil system is considered. See Finding 276.

At any rate, variations in damping have relatively little impact on the behavior of the casks during a seismic event, for there are sufficient margins in the design to maintain the sliding and tipping of the casks within acceptable levels. This is evidenced by one of the analyses that Holtec performed using VisualNastran, in which the model was set so

[13] As explained by Dr. Tseng, it is not appropriate, as Dr. Ostadan suggested, to look only at the fundamental frequency of the pad's response, since that response is only applicable to the pad/soil system. Rather, one would also have to include the casks as part of a global cask/pad/soil system.

that a small value of damping (1%) was used. The predicted displacements of the cask during a seismic event for that case were analogous to those obtained with higher values of damping. Thus, even assuming that the frequency dependence of the soil spring and damping was insufficiently accounted for in the Holtec analyses, the effect of such underestimation would be negligible and would be accommodated by the large margins provided by the design against overturning and cask-to-cask impact. Findings 277-78.

The Sandia results provide further confirmation for the above conclusions. Sandia's finite element analysis bypasses the need for springs and dampers by including soil properties directly into its analysis. The results for the 2.000 DBE was essentially the same as Holtec's.

c. *Long Term Pad Settlement*

In their prefiled direct testimony, State witnesses Bartlett and Ostadan raised a concern that "the long term settlement of the pads has not been considered in design of the pads." At the hearing, this concern was described as involving, not the design of the pads themselves, but the potential impact of pad settlement on the sliding of the casks on the pads due to the deformation caused by such settlement. Dr. Ostadan indicated that he expects that, over the long range, the middle of the pad will settle more and the edges will settle less, deforming the pad into a concave shape ("dishing effect"). Based on conservative assumptions, the upper-bound estimate of the total long-term static settlement of the pads was computed by S&W to be approximately 1.75 in. However, the long-term static settlement of the pads that can be realistically expected to occur would be only one fourth to one third of the 1.75 inch estimate, or approximately ½ inch. Findings 279-283.

The main consequence posited by Dr. Ostadan of the long term settlement of the pads would be a change in the pattern of cask sliding on the pad because the "dishing" effect would make it somewhat more difficult for a cask to slide at some points of the pad and easier to slide at others. However, due to the great stiffness contrast between the concrete pad and the underlying clayey soils, the long-term settlement of the pads at the PFSF will most likely be uniform across the pad, thereby avoiding or greatly minimizing the dishing effect postulated by Dr. Ostadan. Moreover, even assuming a 0.5 inch differential settlement between the center of a pad and the pad's edges, the average slope measured along the short end of the pad would be only 0.159 degrees. Such a slight slope would have no significant impact on the motion of the casks. Findings 288-89.

Another potential consequence of long term settlement of the pads postulated by the State was a "slight inclination" or tilting of the pads. However, the maximum angle of tilting of the pad resulting from long static term pad settlement would be on the order of only 0.64 degrees. Pad tilting is accounted for in the Holtec analysis and shown to have only secondary effects on the stability of the casks. Findings 290.

The last concern raised by the State with respect to long-term settlement of the pads is that it may lead to the cracking of the soil cement layer adjacent to the pads. However, Dr. Mitchell testified that if the maximum differential settlement between the center of the pad and the soil cement were one half inch, this "would alleviate [his] concern a great deal." Finding 291. So, by the assessment of the State's own expert, the possibility of soil cement cracking as a result of pad settlement is slight and, as discussed earlier, such cracking would be inconsequential.

Witnesses for both Applicant and the Staff testified that the anticipated long term static settlement of the pads does not pose a concern in terms of the dynamic stability of the foundations and constitutes, at most, a maintenance issue. Finding 295. That conclusion is well supported by the record.

To put the long term settlement issue in context, several nuclear power plants have operated with estimated long-term settlements of the foundations of safety-related structures in excess of 2 inches. Outside the nuclear arena, the geotechnical standards set by the U.S. Army Corps of Engineers allow as much as a foot of settlement for reinforced concrete foundations supporting smoke stacks, silos, and towers. Thus, he anticipated long term pad settlement at the PFSF is much less than allowed under industry practice and guidelines. Findings 292-293.

d. Pad-to-Pad Interaction

Section D.1.g of Contention L/QQ reads: "The Applicant has failed to analyze for the potential of pad-to-pad interaction in its sliding analyses for pads spaced approximately five feet apart in the longitudinal direction." Though limited to PFS's pad sliding analysis, the allegations made by the State with respect to pad-to-pad interaction go well beyond the text of the contention, as evidenced by the discussion below.

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

At the hearing, the State witnesses expressed the view that there can be seismically-induced interaction between adjacent pads even if the pads do not slide relative to the underlying soil. This potential interaction was said to be due to weakness, deformability and potential lack in uniformity of the soils beneath the pads, and to differences in the number of casks loaded in adjacent pads. Both of these mechanisms were alleged to cause out-of-phase motion of adjacent pads and the transmission of dynamic loadings of one pad on another across the five-foot soil cement "plug" that separates them. Findings 299, 301.

The soils beneath the pad foundations are essentially uniform across the pad emplacement area and have sufficient strength to withstand the design basis earthquake loadings without experiencing significant deformation (i.e., strain). These soils are only "soft" when compared with the adjacent soil cement layer. Therefore, the postulated interaction due to soil deformation is unlikely to occur. Findings 302-304.

In addition, Holtec conducted an analyses in which it modeled two adjacent pads, five feet apart, one pad fully loaded with eight casks, the other having only a single cask, and included a representation of the soil cement between the pads. Holtec simulated both a situation in which the soil cement between the pads retains its integrity and transmits both tension and compression forces, and another in which the soil cement is assumed to be cracked and thus able to transmit only compression forces. In both cases, the configuration and input parameters were set so that the potential for pad-to-pad forces was maximized. Even though the model was intended to maximize pad-to-pad interactive forces, the maximum estimated force in the soil beneath the pads was less than the minimum required to initiate pad sliding. Also, while both cases predicted some interactions between the pads or between the pads and the soil cement, the forces resulting from those interactions resulted in cask motions of the same order – inches – as had been obtained in prior simulations that had not expressly accounted for pad-to-pad interaction forces. The results of the Holtec simulation indicate that pad-to-pad interaction forces have essentially no impact on the stability of either the pads or the storage casks. Findings 305-307.

The State witnesses also testified that their concern over pad-to-pad interaction would be magnified if the pads actually were to slide. However, the use of cement-treated soil and other conservatisms in PFS's design will provide a large margin against the potential sliding of the pads. Thus, interaction between sliding pads is not a realistic concern. Findings 308-309.

The State suggested that the pad-to-pad interaction forces calculated in the Holtec analyses referenced above might be additive to the maximum forces included in the PFS sliding stability calculation, making the forces acting on the pad exceed the available resisting forces and potentially inducing pad sliding. This interpretation is erroneous because the Holtec model is all-inclusive, since it accounts for both the seismic forces acting directly on the pads and the effects of pad-to-pad interaction.[14] Findings 310.

Dr. Ostadan also theorized that there could be configurations in which interaction loads from various pads could accumulate on a single pad and result in potential sliding of the pad. However, even assuming such were to happen, sliding of the pads is beneficial to the stability of the casks and has no adverse safety consequences. Findings 311.

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

In Section D.1.c.(i) of Contention L/QQ the State raises the concern that "the Applicant has failed to provide a realistic evaluation of the foundation pad motion with cement-treated soil under and around the pads in relation to motion of the casks sliding on the pads" because of failure to account for "unsymmetrical loading that the soil cement would impart on the pads once the pads undergo sliding motion." In their direct testimony, Dr. Ostadan and Dr. Bartlett interpreted this concern as addressing the potential collision between a pad and the adjacent soil cement "plug" in the event there is a crack or gap between the two surfaces, such gap causing out-of-phase motion of the pad and the soil cement. However, the pads will be bonded to the underlying clayey soils by means of the cement-treated soil layer between them; therefore, the pads will not slide. Findings 312-313.

[14] In addition, it would be improper to add the maximum seismic forces acting on the pad and the maximum pad-to-pad interaction forces, since they could act at different points in time and, depending on the direction of the pad motion, could be subtractive rather than additive.

In addition, should there be a sliding of the pads leading to a collision with the soil cement plug across a postulated gap between the two surfaces, the soil cement will tend to crush under the imparted loading because there is a significant difference between the compressive strength and modulus of elasticity of the storage pad and those of the soil/cement. The crushing of the soil cement will reduce the force that can be transmitted from one pad to another. Because of the low magnitude of the force that will be transmitted through the soil-cement plug between the storage pads, the effect on the stability of the casks of a collision between the pad and the soil cement plug will be minor. Finding 314.

Holtec performed a confirmatory analysis in which a single pad fully loaded with eight casks was caused to slide on the underlying soil and collide with a fixed, rigid soil cement frame surrounding the entire pad with a clearance gap of approximately 0.6". The analysis showed that the forces produced by those collisions would be less than the forces that would be imparted by the gradual application of compression of the pad against the soil cement, and there would be a reduction by a factor of two in the displacement of the casks. In short, a collision between the pad and the soil cement "plug" would have no discernible adverse impact and, indeed, could have a beneficial effect on the stability of the casks. Findings 315-316.

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

At the hearing, the State witnesses posited the existence of a new "load path" from the pads to the underlying soil due to pad-to-pad interactive forces. The State witness expressed the concern that having additional forces transmitted from one pad to another could result in an accumulation of loads on a particular location and that, as those loads are transmitted down to the soil, they may exceed the soil's load bearing capacity.

The State witnesses offered no evidence that such load accumulation will take place, its magnitude, or its potential effect on the soil given that the soil has a substantial strength, even at its weakest points. Moreover, to the extent that such loading results in sliding of the pads, which as stated is the State's primary concern insofar as foundation failure is concerned, such sliding will be beneficial, as the analyses by Holtec consistently show both without and with soil cement around the pads. Findings 317.

iv) Significance of issue with respect to cask stability.

The pad-to-pad interaction analyses performed by Holtec utilizing assumptions that favored such interactions predict that some loadings will be imparted on the pads. However, such "worst case" forces are not of sufficient magnitude to affect the stability of the casks. The maximum peak-to-peak cask displacement observed in any of the cases is six inches, and the maximum cask excursion from its starting location is 3.8 inches. Thus, pad-to-pad interaction concerns are inconsequential. Findings 318-319.

e. Calculation of Dynamic Forces for Pad Stability

While not identified as a deficiency in the statement of Contention L/QQ, Dr. Ostadan identified in his prefiled direct testimony as an "overriding concern" with the PFS stability analysis for the storage pads that PFS calculated the inertial force acting on a pad by multiplying the peak ground acceleration times the combined masses of the casks and the pad. Dr. Ostadan testified that use of peak ground acceleration to calculate the inertial forces was incorrect and that PFS should have instead used the response acceleration values generated by Holtec in its cask response calculation. Finding 320.

After the issue was raised, PFS sought to determine what the acceleration for the pad would be using Dr. Ostadan's method and the effect it would have on pad stability. PFS determined that the horizontal response acceleration computed based on Holtec analysis would be .79g instead of the .711g used by PFS in its pad stability analyses. Use of the .79g acceleration instead of the peak ground acceleration employed by PFS would merely result in a slight decrease in the "base case" factor of safety against sliding of the pads from 1.27 to 1.22. So the "overriding concern" of Dr. Ostadan has no practical significance, since even if Dr. Ostadan were correct, a calculated margin of 22% against the potential onset of sliding would still exist. Finding 321.

Another confirmation of the appropriateness of its use of peak ground acceleration in its pad stability analysis is obtained by comparing the factor of safety against sliding of the pads computed by PFS for its base case, 1.27, against the factor of safety that would be obtained using the time history of forces developed by Holtec in its analysis of the pad and casks, which was calculated as 1.25. Thus, there is only a very slight reduction in the minimum factor of safety against sliding when the forces computed by Holtec are used instead of the peak horizontal ground accelerations. Findings 322-324.

There is nothing to indicate that the use by PFS of the peak ground acceleration to compute the dynamic forces for pad stability is erroneous. In addition, the peak seismic acceleration, whatever its value, will be applied only at a single point in time in the entire time history; therefore, any errors in the computation of that acceleration will be absorbed by the fact that the average factor of safety against sliding is approximately 10 throughout the duration of the earthquake. Findings 327.

Moreover, as the undisputed testimony of all parties shows, should sliding of the pads occur it would reduce the loading on the casks and be beneficial to stability of the casks. Therefore, this concern, even if valid, would have no practical impact on the safety of the facility. Finding 328.

f. Factors of Safety in a Sliding Stability Calculation

In their direct testimony, the State witnesses indicated that PFS failed to demonstrate in the "simplified Newmark sliding block analysis" presented in calculation G(B)-04 that a factor of safety of 1.1 has been achieved against sliding of the pads." The Newmark sliding block analysis is employed in a hypothetical case that disregards the cohesive bond between the cement-treated soil and the underlying clay and assumes that the only shear strength available to resist sliding at the interface between the cement-treated soil and the underlying soils is the frictional resistance of the clay. For this hypothetical, highly conservative scenario, pad sliding on the order of a few inches is predicted by the analysis. Findings 329-330.

While the State raises a number of concerns with the methodology used by PFS in its Newmark sliding block analyses, the testimony at the hearing showed that the Newmark sliding block analysis performed by PFS is conservative. Moreover, it goes without saying that the hypothetical case in which there is no cohesion between the cement-treated soil and the underlying clay is not the design base case but one of several "what if" scenarios that are included in the stability analysis. PFS's design basis for the pads relies on the shear strength available at the interfaces between the cask storage pad and the underlying cement-treated soil and between the cement-treated soil and the underlying clayey soils. The design basis of the pads provides a conservatively calculated factor of safety against sliding that exceeds 1.1; therefore, the pads do not slide. Indeed, State witness Dr. Bartlett described the concerns over the PFS Newmark analysis as a "secondary issue." Findings 331-332.

Finally, as pointed out above, sliding of the pads would be beneficial in that it would reduce the seismic loading on the casks. Finding 333.

g. Non-Vertically Propagating Waves

In Sections D.1.a and D.1.d of Contention L/QQ, the State asserts that, in spite of proximity to major active faults, the Applicant's calculations unconservatively assume that only vertically propagating in-phase waves will strike the pads, casks and foundations, and fail to account for horizontal variation of ground motion that will cause additional rocking and torsional motion in the casks, pads and foundations. Finding 334.

PFS performed an analysis in which it computed the angle of incidence on the storage pads at the PFSF of earthquake waves originating from the primary sources of earthquake hazards to the PFSF, the Stansbury and East faults. The analysis utilized the physical laws governing the propagation path of seismic waves from a point source deep under the surface of the earth to a point on the surface. The analysis determined that the angle of incidence of the waves at the PFSF site would be very close to vertical, typically less than 10 degrees. Thus, the proximity of the site to the major active faults does not result in high angles of incidence from vertical for earthquake waves impinging the site, and the assumption of vertically propagating waves is reasonable. Findings 335-336.

PFS calculated the difference in arrival times at opposite edges of a pad for waves having angles of incidence on the order of 10 degrees or less and obtained differences in arrival times on the order of 0.001 to 0.002 seconds. These time differences would only affect motions in the very high frequencies of 50 to 100 Hz , which are far above the dominant frequency range of peak cask response of 1 to 5 Hz calculated by PFS. Therefore, the rocking and torsional motions of the storage pads caused by the small angles of incidence from vertical of the seismic waves arriving at the PFSF site would be insignificant. Findings 337.

Geomatrix quantified the effects of non-vertically propagating waves on a pad's torsional and rocking response using a well-accepted methodology reported in the technical literature. Geomatrix concluded that the calculated angles of incidence of the seismic waves would induce a very small amount of additional torsion on the pads (a maximum of 1 to 3 percent of the amplitude of the direct horizontal translational motion), and an equally small amount of rocking (a maximum on the order of 5 percent of the direct vertical motion amplitude). These results show that the additional rocking and torsional motion of the pad caused by non-vertically propagating waves at the PFSF would be small compared to the motion caused by the vertically propagating waves and would be absorbed by the very large margins in the range of cask movements calculated by Holtec for the design basis earthquake. Findings 338, 341-346.

An independent calculation by the NRC Staff of the maximum bending in the storage pads due to the arrival of non-vertically propagating waves determined that maximum rocking of the storage pad will produce displacements of 1.16 inches and the amount of rotation will be less than 0.1 degrees. Accordingly, the Staff also concluded that the stability of the cask will not be affected by non-vertically propagating seismic waves that may occur at the site. This conclusion is further confirmed by the results of the Sandia analysis which takes into account any synergistic effects of non-vertically propagating waves. Finding 339.

Dr. Ostadan did not disagree that the departure from vertical of the angle of incidence of seismic waves arriving at the PFSF site is small, and that the difference in arrival times of the wave from one end of a pad to another is also small. He referred, however, to the potential effect of a row of ten pads interacting with another adjacent row, and hypothesized that the phase difference in the seismic loadings caused by the difference in arrival times at one row versus another could cause some interaction between the two sets of pads as they slide. Finding 347.

Dr. Ostadan's concern appears to relate only to the case discussed above in which the pads are assumed to slide because of the existence of cohesionless soils. As noted, this is not the design base case, but a hypothetical "what if" situation. Also, as Dr. Ostadan acknowledged, the separation between two contiguous rows of pads is only a few feet, whereas the source is several miles away, under the earth's surface. Thus, the difference in phase due to different arrival times of the seismic excitation to contiguous rows of pads will be small and the interaction between the two rows of pads will most likely be unnoticeable. Findings 348-354.

Based on the above discussion, the effect of earthquake motions on structures and components at the PFSF may be properly represented by the use of vertically propagating earthquake waves, and the effect of non-vertically propagating waves alleged by the State can be disregarded. Finding 355.

h. Cold Bonding

Section D.1.f of Contention L/QQ asserts that PFS "has failed to consider the potential for cold bonding between the cask and the pad and its effects on sliding in its calculations." Dr. Ostadan defined the cold bonding phenomenon as occurring "when two bodies (cask and pad) with such a large load (the cask) are in contact. Some local deformation and redistribution of stresses may occur over many years at the points of contact, which would create a bond in the form of a welding, which increases the resistance to sliding of the cask on the pad." Finding 356.

The average pressure at the interface between the pad and the cask, conservatively assuming that the entire weight of the cask is supported only over a 12" wide annulus around the periphery, is only 40 psi. This pressure is well below the allowable bearing stress of 1785 psi for concrete with a compressive strength of 3000 psi. Such pressure is clearly insufficient to create a bonding between the steel bottom of the cask and the concrete surface of the pad. Findings 357-360.

Assuming there was cold bonding, the State witnesses testified that it would operate to impede the initial sliding motion of the cask under seismic loadings. However, the seismic forces would readily break the bond and the cask would then slide on the pad in accordance with whatever coefficient of friction existed between the cask and the pad. Therefore, assuming the cold bonding phenomenon actually took place, its effect would be very limited both in duration and effect and would be subsumed in the variable coefficients of friction assumed in the Holtec analyses. Any small perturbations in the cask response due to irregular sliding would be within the range of results encompassed by the design basis simulations. Findings 361.

i. Need for Multiple Sets of Time Histories

Section D.1.h of Contention L/QQ alleges as a deficiency the fact that the PFS cask stability calculations use only one set of seismic time histories in the analysis. The State claims that non-linear analyses are sensitive to the phasing of input motion, that more than one set of time histories should be used, and that "fault fling" (i.e., large velocity pulses in the time history) and its variation and effects are not adequately bounded by one set of time histories. Finding 362.

Time histories represent the variation of ground acceleration with time during an earthquake. Applicable NRC guidance (Section 3.7.1 of NUREG-0800 and Section 5 of NUREG 1567) allows the designer a choice between two alternative methods for developing design time histories. One approach is to use multiple sets of time histories that in the aggregate envelop the design response spectra. The second approach is to develop a single set of time histories sized to envelop the design response spectra, as well as a target power spectral density function. Findings 303-364.

PFS elected to use the second approach, and its consultant Geomatrix developed a set of time histories in accordance with the methodology specified in the NRC guidance documents. The time histories developed by Geomatrix also accounted for forward directivity in the design response spectra, and thus incorporated fault fling effects.[15] Findings 365-368.

State witness Dr. Ostadan opined that the industry practice is to require the use of multiple time histories for non-linear analyses. However, Dr. Ostadan's opinion was based on his reading implicitly into Section 3.7.1 of the Standard Review Plan such a requirement, which is clearly not stated in the SRP. Dr. Ostadan's interpretation of the SRP is inconsistent with that of the Staff, and is also inconsistent with the Staff's acceptance of the use of single time histories for non-linear analyses for both free standing spent fuel rasks and casks in numerous proceedings in which Holtec has been involved. Finally, the use of the single time history procedure is appropriate for the PFSF because it is more likely to capture the maximum amplitudes and frequency content of the earthquake excitation, which is the most important contributor to cask stability. [16] Findings 369-371.

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[15] Fault fling describes the enhanced ground motions that occur when the seismic rupture moves at a speed that is near to that of the seismic waves radiating from the fault plane, such that the seismic waves build up into a coherent, strong velocity pulse that arrives in the early portion of the ground shaking.

[16] Even if one were to assume that Dr. Ostadan is right and multiple sets of time histories should have been used, this would only result in a different set of seismic inputs; the margins of safety that exist in the design would accommodate any differences in cask loadings.

7. Specific State Claims Regarding Stability Analysis of the CTB

a. Overview of CTB Design and Seismic Stability Analyses

The CTB is a massive building, conservatively designed to industry codes and standards that provide wide margins of safety. Because of these conservatisms and its physical configuration (short, squat, bottom heavy), there is no concern about potential overturning of the CTB under beyond-design basis earthquake loadings. Nor is there any concern about bearing capacity failure of the building, since the margin of safety provided in the design is 5.5. Finding 372.

Thus, the only failure mechanism that is being raised as potentially occurring with respect to the CTB is sliding. However, as noted earlier, any such sliding would have no safety consequences, since there are no safety-related structures connected to the building that could be adversely affected by the sliding. Finding 373.

b. Failure of Soil Cement Buttress in Seismic Event

In Section 2.c of Contention L/QQ the State asserts that PFS has not supported its assumption that the soil cement buttress surrounding the building will provide the adequate passive resistance to sliding, because PFS has failed to demonstrate that the proposed soil cement buttress will not crack during a seismic event. However, as discussed in Section C above, any cracks that form in the soil cement surrounding the CTB will be thin, vertical, random cracks that do not affect the ability of the soil cement to provide the passive resistance to sliding relied upon in the design. Finding 374.

At most, the building and the soil column under it will displace a small distance to close each crack, and then the full passive resistance of the soil cement to sliding will be restored. Findings 375-77.

The State also asserts that the CTB dynamic stability analysis performed by PFS does not address the possibility that the foundation mat and the surrounding soil cement will experience out-of-phase motion that results in the mat imparting loadings on the soil cement that could cause it to crack. However, the accelerations of the structure and the soil cement are expected to be similar in the vicinity of the structure, and the loadings applied to the soil cement by the CTB mat will not be substantial. The soil cement is strong enough to resist the horizontal loads to be applied by the CTB foundation mat and stiff enough to minimize the movement of the canister transfer building base mat against it. Therefore, soil cement cracking is unlikely to develop through this mechanism. Again, should cracks develop in the soil cement, the building and the soil column under may displace a small distance to close each crack, and then the full passive resistance of the soil cement to sliding will be restored. Findings 378-379.

c. Potential Reduction in Damping

The State claims in Section D.2.b of Contention L/QQ that the soil cement buttress will trap some of the energy that would be dissipated through radiation damping, thus increasing the seismic loads to which the building will be subjected. The theory is that when two building foundations are in proximity, the presence of one foundation will trap the energy that would otherwise be released to the soil by the other foundation. However, the soil cement around the CTB is totally unlike a building foundation and will not trap energy in the manner described in the contention. In addition, energy radiates downward into the soil at the interface between the CTB and the subgrade at the base of the foundation mat. Thus, the presence of a soil-cement cap around the CTB has no effect on this energy-dissipation mechanism, because it is directed downward and not in the horizontal direction. Findings 380-381.

Even if one were to assume that the presence of soil cement around the CTB is equivalent to having another building in the CTB's proximity, under the applicable codes and the general practice in the industry there is no need to consider structure-to-structure interaction in the dynamic analyses. Findings 382-383. The alleged reduction in radiation damping can thus be disregarded.

d. *Mat Rigidity*

Section 2.a of Contention L/QQ questions the assumption in the CTB dynamic stability analyses that the building's foundation mat is rigid. That assumption, however, has been confirmed through a PFS analysis which shows that the maximum variation of vertical displacement along the centerline of the building in the N-S direction is 0.163 inches over the length of 279.5 ft., which represents a less than 0.005% deflection. The maximum variation of vertical displacement in the E-W direction is .333 inches over the length of 240 ft., or about 0.01% deflection. Such small displacements over an area of 67,200 square feet (240 feet times 280 feet) show that the CTB basemat acts like a rigid body under earthquake loadings. Findings 384-385.

As with the storage pads, Dr. Ostadan maintained that the small displacements predicted by the PFS calculation can be significant because the important consideration is not the amplitude of the displacements but how many times it occurs over the length of the structure. However, the CTB mat displacements do not take place over short distances but rather over a distance of about 65 feet, and there is only one such occurrence, at the southern end of the mat; the northern end of the mat shows no displacements. Thus, the CTB basemat is rigid even under Dr. Ostadan's definition. Finding 386.

Assuming the CTB mat to be rigid is appropriate in view of the physical configuration of the mat (five-foot thick reinforced concrete, stiffened by shear walls connected to it), which provides the mat with significant resistance to deformation in the vertical and the horizontal directions. The assumption of mat rigidity is endorsed by Section 3.3.1.6 of industry code ASCE 4-86 and is also consistent with the practice in the nuclear industry, which is to treat foundations for safety-related structures similar in design to the CTB as rigid. Findings 387-388.

e. *Non-vertically Propagating Seismic Waves*

The concern over the effect of non-vertically propagating waves applies both to the CTB and to the pads and has been addressed above with respect to the pads. The same conclusion reached for the pads applies to the CTB: the effects of non-vertically propagating waves on the seismic loadings imparted on the CTB foundations are negligible. In addition, to the extent there are any potential effects from non-vertically propagating waves on the stability of the CTB, PFS has addressed such effects by incorporating into the detailed design of the building a mass eccentricity factor of 5%. This approach is recommended in the Commentary to Section 3.3.1.2(a) of the ASCE 4-86 industry code. By implementing this recommendation in the detailed design of the CTB, PFS avoids any need to account in the seismic analyses of the building for non-vertical propagation of seismic waves. Findings 389-390.

f. *Conclusions on CTB Dynamic Stability Claims*

It is clear from the record that the State has failed to substantiate any of its concerns regarding the dynamic stability of the CTB under seismic conditions. Moreover, the concerns raised by the State, if substantiated, would only have the effect of reducing the margin of safety against sliding, potentially leading to some sliding of the structure in the event of an earthquake. It does not follow, however, that sliding of the building would occur even if its factor of safety against sliding drops below 1.0, because there are substantial conservatisms included in the CTB sliding stability calculation, which provide additional margins of safety against sliding. Findings 391-392. Even if the building were to slide, there would be no adverse safety consequences. Therefore, the deficiencies asserted by the State, if existing, would have no licensing significance.

E. Seismic Exemption

PFS requested an exemption from NRC regulations that would otherwise have required the PFSF, an ISFSI located west of the Rocky Mountains, to be designed based on seismic ground motions obtained using the deterministic procedures and criteria in Appendix A, 10 C.F.R. Part 100. The exemption requested that PFS be allowed to design the PFSF based on a probabilistic seismic hazard analysis (PSHA") and a 2,000-year mean return period ("MRP") design basis earthquake ("DBE"). The exemption request was consistent with the general trend of the Commission to allow the use of PSHA methodology. For example, Appendix A, 10 C.F.R. Part 100 was replaced in 1996 with regulations and guidance documents that allow the use of PSHA methodology for the seismic design of new nuclear power plants. The Commission has recently proposed a rule change to allow the use of PSHA methodology for the seismic design of ISFSIs regulated under 10 C.F.R. Part 72. Findings 393-94.

The initial rulemaking plan for implementing the change from deterministic to probabilistic methods for the seismic design of ISFSIs was set forth in SECY-98-126 (June 4, 1998), which discussed three different options for incorporating PSHA methods into 10 C.F.R. Part 72. The "preferred" option proposed a 1,000-year mean return period design basis earthquake for "Category 1" structures, system and components important to safety ("SSCs") i.e., those whose failure would not result in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a); and a 10,000-year mean return period design basis earthquake for Category 2 SSCs (those whose failure would result in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a)). SECY-01-0178 (September 26, 2001) modified the rulemaking plan to add a new "preferred" option consisting of the use of a single 2,000-year mean return period earthquake as the design basis for all ISFSI SSCs. In a Staff Requirements Memorandum dated November 19, 2001, the Commission approved the modification to the rulemaking plan proposed by SECY-01-0178. The proposed rule was published for comment on July 22, 2002. Findings 395-397.

On April 2, 1999, PFS filed its exemption request seeking authorization to use PSHA methods for determining the seismic design of the PFSF, with a 1,000-year MRP DBE as the PSHA design basis. On August 24, 1999, PFS amended the exemption request to use a 2,000 year MRP DBE for its design basis. The Staff approved PFS's amended exemption request in its Safety Evaluation Report issued in October 2000. The Consolidated SER dated March 2002 sets forth the final statement of the Staff's reasons for granting the exemption. The Applicant fully sets forth in its prefiled direct testimony and at the hearing its own independent justifications for why the granting of its exemption request would not endanger life or property or the common defense or security and would otherwise be in the public interest. Finding 398.

1. Acceptability of Probabilistic Seismic Hazard Analysis Methodology

The State, the Staff and PFS agreed that the use of PSHA methods is appropriate for the seismic design of the PFSF, and may be used in lieu of the deterministic methods currently called for by the Part 72 regulations. PSHA methodology has several advantages over a deterministic approach, including:

- PSHA incorporates the effects of all potential seismic sources, rather than focusing on only the most significant earthquake sources, as does a deterministic safety hazards analysis ("DSHA");

- PSHA considers the range of source-to-site distances, rather than using a fixed site-to-source distance as does a DSHA;
- PSHA considers the range and randomness of earthquake magnitudes;
- PSHA accounts for uncertainty in assessment of seismic hazards, providing a more complete estimate of earthquake hazards than does DSHA.

Thus, PSHA methods, unlike deterministic methods, take into account the entire range of potential seismic events that could affect a site. These data are used to construct a curve of estimated annual probability of exceedance versus level of ground motion. This curve can be used to select the design ground motion at a level corresponding to a pre-specified mean annual probability of exceedance ("MAPE"). The PSHA methodology is well-established and widely accepted, being the prevalent methodology now used for seismic design. In addition to the advantages described above, the probabilistic approach can be used to set design criteria that are consistent among different regions and among different failure consequences, thus allowing a rational and a equitable allocation of safety resources. Findings 403-408.

In short, the use of PSHA methodology by PFS to characterize the seismic hazard at the site and to set the seismic design basis of the PFSF is fully consistent with current NRC policy and practices as well as with broader engineering policy and practice. Finding 409.

2. Acceptability of 2,000-Year Mean Return Period Design-Basis Earthquake

The use of a 2,000-year return period earthquake for the seismic design of the PFSF is supported by the principles of risk-informed seismic design, the conservatisms of the applicable codes and standards, and the overall conservatism embodied in the design of the facility. Finding 411.

a. Risk-Informed Seismic Design

A 2,000-year return period design-basis earthquake is consistent with well-accepted principles of risk-informed seismic design. The first such principle is that there should be a risk-graded approach to seismic safety that allows facilities and structures with lesser failure consequences to have larger mean annual probabilities of failure. Thus, such facilities can be designed to the less severe ground motions associated with a shorter return period earthquake. This principle is reflected in many design codes, including the draft International Standards Organization guidelines for offshore structures, Federal Emergency Management Agency ("FEMA") guidelines for building assessment, and DOE Standard 1020. All parties accepted this principle, and recognized that the public health and safety consequences of a failure of the PFSF ISFSI would be less severe than those of a failure of a nuclear power plant. Indeed, the testimony discussed below regarding dose consequences further demonstrates the minimal public health and safety consequences of a postulated failure at the PFSF ISFSI. The Staff's grant of the exemption request is thus consistent with this first principle of risk-informed seismic design. Findings 411-414.

The second principle of risk-informed seismic design is that the ability of the design basis earthquake to provide the desired level of seismic safety is to be assessed based on two considerations or factors, referred to as the "two-handed approach". The first factor is the MAPE of the DBE. The second factor is the level of conservatism incorporated into the criteria and procedures for the design of the facility. All parties agreed that these two factors, taken together, are necessary to determine the adequacy of the seismic design of the facility and that focusing on just one factor provides an incomplete, inaccurate picture of the seismic design basis for a facility. Finding 415.

The conservatisms which are embedded into design codes, standards and procedures may be referred to as "risk reduction factors." These risk reduction factors express the degree to which the likelihood of failure of a structure, system, or component in a facility is reduced by the conservatisms inherent in the codes, standards and procedures that govern its design. The practical effect of the existence of risk reduction factors is to further reduce the MAPE. For example, a design basis earthquake may be expressed as a MAPE of 5×10^{-4}, but a risk reduction factor of 5 would reduce the MAPE to 1×10^{-4}. Findings 416-420.

Because of the effect of the risk reduction factors, virtually all facilities designed against a given DBE have a mean return period to failure that is longer than that of the earthquake for which they are designed, meaning that they are able to withstand a more severe, i.e., longer return period, earthquake than the DBE. Thus, the actual level of seismic safety achieved by a facility is dependent upon both the DBE and the risk reduction factor inherent in its design codes, standards and procedures. Findings 416.

b. Risk Reduction Factors of PFSF SSCs

Dr. Cornell provided testimony demonstrating that the risk reduction factors applicable to SSCs at the PFSF are at least 5 to 20. These risk reduction factors, coupled with the 2,000 year DBE, would provide a performance goal of 1×10^{-4} or better. Finding 428-431.

Dr. Cornell also testified that 1×10^{-4} is an appropriate performance goal for the PFSF, applying a risk-graded approach to seismic safety. Dr. Cornell's conclusion was based on three considerations. First, the use of a probability of seismic failure or performance goal for the PFSF of 1×10^{-4} is consistent with the risk-graded probabilistic approach that the Commission has adopted. Second, a performance goal of 1×10^{-4} is consistent with DOE policy as represented by DOE-STD-1020, which provides a performance goal of 1×10^{-4} for facilities comparable to the PFSF. Third, a performance goal of 1×10^{-4} provides a lower probability of failure than the performance goals associated with structures critical to public safety, such as bridges and hospitals. The State agreed with Dr. Cornell that the 1×10^{-4} performance goal was appropriate for the PFSF, and that the performance goal would be met if Dr. Cornell's conclusion regarding the magnitude of the risk reduction factors was correct. Findings 424-427, 429.

The State provided little testimony directly contesting Dr. Cornell's conclusions. Rather, State witness Dr. Bartlett expressed concern in three areas: (1) whether it was appropriate to extrapolate the risk reduction factor available in the designs of SSCs for NPPs to the design of SSCs for ISFSIs, since the applicable code and standards may be different for both types of facility; (2) whether such risk reduction factors are in any case available with respect to the foundations of the safety-related structures at the PFSF; and (3) general concerns about the stability of the casks. Findings 432, 435-437.

Specifically, Dr. Bartlett opined that the codes and standards applicable to ISFSIs (and particularly the provisions of the Standard Review Plans ("SRPs") for ISFSIs) may incorporate less conservatism than the corresponding codes and standards applicable to NPPs. Thus, he stated that the SRPs for an ISFSI might not provide the same risk reduction factor of 5 to 20 or greater that would be obtained through the application of the NPP SRP (NUREG-0800). This concern was expressed by Dr. Bartlett to arise from his lack of knowledge of the conservatisms provided by both sets of standards, rather than being an informed opinion on his part. In reality, the design of the SSCs at the PFSF satisfies the codes and standards specified in NUREG-0800 and used generally in the design of NPPS. Finding 432.

With respect to Dr. Bartlett's second concern, the foundations of the CTB and those of the storage pads have the same risk reduction factors of 5 to 20 or greater that exist at other aspects of the PFSF facility. This is demonstrated by the fact that the failure of NPP foundations has been included as a potential failure mechanism in the seismic PRAs that have been conducted for numerous nuclear power plants. The results of those PRAs determined that NPP foundation failure modes, such as overturning, loss of bearing capacity and sliding, were not critical failure conditions, meaning that sufficiently large risk reduction factors were incorporated into their design to avoid their failure during an eqrthquake. The same is true for the PFSF foundations. Finding 436.

Dr. Bartlett also expressed the view that the conservatisms claimed by PFS to be available with respect to the design of the foundations of the CTB and the cask storage pads are insufficient. He claimed that the SRP factor of safety of 1.1 against sliding, overturning, and loss of bearing capacity provided smaller <u>absolute</u> safety margins where the earthquake magnitude was smaller than for the equivalent safe shutdown earthquake ("SSE") for NPPs. However, he conceded that the <u>proportional</u> margins are identical. Findings 437.

Further, in asserting that the margins against foundation failure are slim, Dr. Bartlett did not take into account the additional factors of safety inherent in the conservatism of the PFS seismic analyses. For example, in the calculation of the factor of safety against sliding for the storage pads, PFS included numerous conservatisms, including: using the lower bound, worst case value for the static shear strength of the soils throughout the entire soil profile; not taking into account the increase in strength the soils would draw from cyclic seismic loading; ignoring the passive resistance provided by the soil cement surrounding the pads; and using the peak magnitude of the earthquake ground motions, which will only occur for a brief period of time during the earthquake loadings. Taking credit for these conservatisms would raise the factor of safety against sliding of the pads from 1.27 to over 5 for the design basis case. Findings 438-439.

Similarly large margins exist due to conservatisms in the calculations for both overturning and loss of bearing capacity. Taking credit for just two of the conservatisms in the calculation for the factor of safety against bearing capacity failure (use of load combinations allowed by ASCE 4-86 and taking credit for the dynamic strength of the soils), would increase the factor of safety against loss of bearing capacity to 3.63. The factor of safety against pad overturning is already 5.6, even before taking credit for any of the conservatisms built into that calculation. The CTB foundation calculations likewise contain numerous design conservatisms that, if credited, would provide a factor of safety on the order of 10 against loss of bearing capacity, 5 or more against overturning, and significantly increased margins against sliding. Findings 440-444.

With respect to Dr. Bartlett's concerns about cask stability, the casks also have additional margins of safety that are not taken into consideration in any of the design basis calculations. Beyond–design-basis analyses conducted by Holtec and Sandia demonstrated that the effective risk reduction factor of the HI-STORM 100 Cask System is in excess of 5. The Holtec beyond-design-basis analysis established that a HI-STORM 100 storage cask would not tip over during a 10,000-year return period earthquake and would still retain significant margins against tipover during such an event. The maximum cask rotation for the 10,000-year event was approximately 10 to 12 degrees, leaving a factor of safety against overturning of nearly three. Even under a variety of worst-case scenarios this factor of safety still existed against overturning for a 10,000-year return period event. These conclusions were confirmed by the Sandia analyses which, using state-of-the-art modeling techniques, demonstrated that the cask rotations would be even smaller than Holtec predicted (on the order of 1 degree for a 10,000-year return period event) suggesting even larger margins of safety against tipover. Findings 448-450.

Even if the casks were to tip over, it is uncontested that there would be no breach of the confinement barrier of the canister containing the spent nuclear fuel. Holtec's tipover analysis demonstrates that the effects of the tipover would remain within the HI-STORM 100 Cask System's design limits. But, even more importantly, the cask and canister have substantial additional margins of safety beyond the design limits. The actual g limit for the cladding in the fuel assemblies is at least 63g. Huge margins exist in the design of the MPC canister system, which prevents the release of radioactive materials under loadings up to 300g, as demonstrated in an evaluation of a 25 ft. straight drop of an unprotected MPC on a hard concrete floor. Finding 451.

c. Asserted Need for Fragility Curves

While Dr. Bartlett asserted in his prefiled testimony that the failure to develop fragility curves (showing probability of failure as function of seismic loading) was a deficiency of PFSF's beyond-design-basis analysis, a fragility curve is not needed to confirm that a particular component has a risk reduction factor larger than some specified level, or can meet a specified seismic performance level. These determinations can be made without the aid of a fragility curve, through analyses of the desired performance goal level that shows that a particular goal has been met, as was done by Holtec's beyond design basis accident analysis. At the hearing, Dr. Bartlett acknowledged that it was not necessary to develop fragility curves for the SSCs at the PFSF in order to determine whether the specified performance goal was met. Findings 452.

3. NRC Staff-State Disputes on Adequacy of Staff Review of 2,000-year MRP DBE Exemption

The State challenged the process by which the Staff reviewed the PFS seismic exemption request. The heart of the dispute between the State and the Staff has to do with how the Staff evaluated the Applicant's PSHA and determined it to be conservative.

However, the recognized conservatism of the NRC's seismic design requirements, discussed above, and the comparatively low risk of the PFSF was integral in the Staff's approval of the exemption. Additionally, the Staff identified numerous conservatisms in PFS's PSHA that characterized the seismic hazard for the PFSF. Based on its independent evaluation and analysis of the hazard, the Staff concluded that various conservatisms in the Applicant's PSHA may have led to an overly conservative hazard result. The Staff judged that the over conservatism in PFS's estimation of the hazard could be 50% or more.

The State did not challenge the adequacy of the PSHA, but did challenge various aspects of the Staff's conclusion regarding the seismicity of the PFSF site, and in particular disagreed with Staff testimony that the appropriate benchmark NPP SSE against which to judge the acceptability of the PFS sites was an earthquake with a MRF of 5,000 years as opposed to 10,000 years.

None of the State's concerns about the Staff's seismic characterization of the PFSF site undercut the validity of the seismic exemption granted to PFS. Regardless of how one categorizes the seismicity of the PFSF site, the PSHA performed by PFS is conservative. If the PFSF is not a high-seismicity site, as testified by the Staff the real hazard curves should not be as high as those produced by Geomatrix, and consequently the PFSF has been designed to a significantly higher return period than the 2,000 year return period ground motions obtained from the Geomatrix hazard curves. If that is the case, there is a very large additional margin in the design, beyond all other conservatisms discussed so far. On the other hand, if the hazard curve produced by Geomatrix accurately reflects the conditions at the PFSF, the PFS design is conservative for the reasons discussed earlier. Thus, the State's dispute with the Staff over how the Staff evaluated the exemption request and PFS's seismic analyses does not undermine the adequacy of the

PFS exemption request and supporting documents, and does not negate the fact that the grant of the exemption adequately protects public health and safety. Findings 454-471.

4. Specific Issues Raised in Subparts of Section E Other Than Radiolodigical Dose Consequences

The State listed seven bases underlying its concerns over the seismic exemption granted to the PFS. In Basis 1, the State challenged the Staff's grant of the exemption because it did not conform to SECY-98-126. There is, however, no requirement that the Staff follow a rulemaking plan when ruling on an exemption request. Also, the rulemaking plan was changed in SECY-01-0178, making the use of a 2,000-year mean return period earthquake as the design basis for all ISFSI SSCs the preferred option for incorporating PSHA methodology into 10 C.F.R. Part 72. The PFSF exemption request conforms to the new preferred methodology. Findings 473-474.

Basis 2 is discussed separately in Section 5 below.

The issue raised in Basis 3 is the appropriate NPP SSE benchmark for applying the risk graded approach to the PFSF. However, the State's witness Dr. Arabasz mooted this issue, by agreeing that a MAPE of 10^{-4} is the appropriate reference earthquake for a nuclear plant at the PFSF site. This is the same benchmark used by PFS. As discussed above, the Staff testimony suggests that a 5,000-year earthquake should be the NPP SSE benchmark. However, we do not need to decide that issue because we believe that the sufficiency of the 2,000-year DBE for the PFSF is established, per Dr. Cornell's analysis, when judged against a 10,000-year NPP SSE benchmark. Findings 477-479.

In Basis 4, the State challenged the exemption granted to PFS on the grounds that the NRC Staff inappropriately relied on DOE-STD-1020-94, because the NRC Staff did not adopt this Standard in SECY-98-126. Thus, the State argued that the Staff erred in adopting only one (MAPE of the DBE) of the two "hands" required to establish seismic safety. However, implicit in the SRP is the adoption of a two-handed approach, which takes into account conservatisms built into the SRP-dictated design procedures and criteria. Moreover, PFS's analysis which fully embraces the two-handed approach, adequately addresses the State's concern raised in Basis 4. There was no reason put forward by any of the parties for disagreeing with the underlying methodology in DOE-STD-1020-94, which was viewed as methodologically sound and appropriate for setting standards for seismic design and as a model of explicit, graded, risk-consistent seismic criteria.[17] Findings 480.

In Basis 5, the State challenged the grant of the PFSF exemption claiming that the NRC Staff's reliance on the 1998 exemption granted to DOE for the Idaho National Engineering and Environmental Laboratory ("INEEL") ISFSI for the Three Mile Island, Unit 2 ("TMI-2") facility fuel is misplaced because the grant of the exemption there was based on circumstances not present with respect to the PFS ISFSI. The State appears to have read too much into the Staff's reference to the TMI-2 ISFSI, which was not relied on as a controlling precedent in granting the seismic exemption, but was merely referenced to illustrate that ISFSIs have been granted exemptions from seismic design criteria using well-accepted risk-graded principles. Further, the PFS exemption is fully supported by the record developed in this proceeding and is not dependent on the Staff's INEEL determination. Findings 482-483

In Basis 6, the State raised two claims. First, it asserted that the 2,000-year mean return period for the PFS facility does not ensure an adequate level of conservatism because design ground motion levels for certain new Utah building construction under the

[17] While the DOE has recently raised the DBE for category PC3 structures (the category in which ISFSIs would fall were they DOE facilities) from 2,000 years to 2,500 years, the level of conservatism in the applicable design procedures and criteria was reduced such that the performance goal for PC3 structures remains unchanged at 1×10^{-4}. Findings 481.

International Building Code 2000 ("IBC-2000") and highway bridges will employ DBEs of 2,500, which will be more stringent. However, the comparison between the two sets of codes based only on the MRP DBE is erroneous, as the State's own experts agree. The design procedures and criteria of the IBC-2000 are much less conservative than those of the SRP in several regards, including having less stringent design procedures and acceptance criteria than those found in the NRC SRPs. Design codes such as UBC and IBC-2000 yield a risk reduction ratio of only about 2, compared to the risk reduction ration of 5 to 20 or more typical of NRC SRPs. Thus, a facility designed to the NRC SRPs is going to be designed 2.5 to 10 times or more conservatively than one design under the other building codes. Moreover, all PFSF important-to-safety SSCs have risk reduction factors sufficient to provide a probability of failure of 10^{-4} or lower, i.e., at least two times lower than essential facilities designed to the IBC-2000. Additionally, a number of key important-to-safety SSCs in the PFSF have great robustness and/or fractional operating periods that reduce their probabilities of failure even further. Thus, the PFSF, even though designed using a lower MRP DBE than the starting point for determining the seismic ground motions under the IBC-2000 or UBC model building codes, would be able to withstand significantly stronger ground motions than a structure designed to the ostensibly higher MRP DBE. Findings 484-488.

In Basis 6, the State also claims that the 2,000-year mean return period for the PFS facility does not ensure an adequate level of conservatism because the return period was chosen based on the twenty-year initial licensing period rather than a potential thirty to forty-year operating period. All parties agree that public safety hazards are usually measured in frequency of occurrence (whether as measured in annual probabilities or, for example, probabilities per 50-year period). 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. The Commission uses risk annual frequency measured on annual basis for assessing NPP risks, and choosing life-time risk metrics would create inconsistencies and raise practical issues, e.g., how the seismic design basis should be determined to take into account potential re-licensing. Findings 489-495.

5. Radiological Dose Consequences Issues

a. *Applicable Regulatory Standards for Radiological Consequence Dose Limits*

All parties agreed that 10 C.F.R. § 72.106(b) governs the radiation dose limits for an accident involving a cask tipover during a seismic event, rather than the normal operation dose limits of 10 C.F.R. § 72.104(a). Late in the hearing, however, the State asserted that the important issue was the duration of the accident, because radiation doses should be calculated for the entire duration of an accident. 10 C.F.R § 72.106(b) does not define any accident duration, and no regulatory guidance is directly on point. Despite raising the issue, the State offered no testimony on how the duration of an accident should be defined or how long an accident might be deemed to last. As discussed below, however, the question of duration is moot because (1) there would be no tipover of the storage casks during a DBE, or even during a beyond-design-basis seismic event, and (2) even if such a tipover were to occur and one assumed (as the State did) an unrealistic, worst case scenario, the dose limit of 5 rem in 10 C.F.R. § 72.106(b) would never be reached. Findings 496-502.

b. *Evaluation of Potential Damage from Hypothetical Tipover Event*

As discussed above, the HI-STORM 100 storage casks will not overturn, uplift or slide and impact one another during a design basis seismic event. Thus, there is no credible mechanism by which any damage could occur to a HI-STORM 100 storage cask at the PFSF. The State's witness on radiological dose consequences, Dr. Resnikoff, stated that he had no independent basis for predicting that a cask tipover, uplift, or sliding and collision would occur. Findings 524-528.

Moreover, even if a tipover, uplift, or sliding and collision event were to occur, it would have no adverse radiological dose consequences. No State witness has provided testimony asserting that a cask impact from uplift, sliding and collision, or tipover due to a postulated cask tipover event at the PFSF would cause: (1) flattening or other damage to the storage cask, (2) cracking of the steel or concrete, (3) thinning of the steel shell or radial concrete shield, or (4) displacement of the cask lid. Neither has any State witness quantified the effects of any of those mechanisms. Findings 503-511.

On the other hand, results of the PFS analysis of a hypothetical, non-mechanistic cask tipover event show that all stresses on the storage cask remain well within the allowable values of the HI-STORM 100 Certificate of Compliance, assuring integrity of the multi-purpose canister confinement boundary. Because of the design of the HI-STORM 100 storage cask, it is physically impossible for the radiation-absorbing concrete mass in the cask to be lost or reduced in the event of an accident. Localized damage to the concrete and outer shell of the storage cask would reduce the roundness of the storage cask only in the immediate area of the impact and would occur at the top of the storage cask, where shielding is thicker. Such a local deformation would not significantly affect the shielding performance of the storage cask, since the same mass of steel and concrete would still be present. Findings 505-507.

Dr. Resnikoff did not know whether a cask impact due to a beyond design basis seismic accident at the PFSF would cause flattening or other damage to the storage cask, whether or how much cracking of the steel or concrete would occur, or whether or how much thinning of the steel would occur. Dr. Resnikoff testified that the State's concerns relating to damage to the cask, including all the mechanisms postulated in his testimony (e.g., deformation of the cask, flattening or thinning of the concrete, stretching or thinning of the steel, cracking of the cement, and cask lid displacement) were theoretical concerns, and that he did not have expertise to determine whether or to what extent they would occur. Dr. Resnikoff also acknowledged that he does not have either experience or expertise in measuring or quantifying concrete cracking, determining the strength of steel or concrete, calculating the initial angular velocity of a cask during tipover, or measuring or quantifying thinning or flattening of the steel in the cask shell due to impact. Therefore, he would not be qualified to make any of those assessments in any case. Findings 527-528.

Nevertheless, Dr. Resnikoff expressed three concerns in his prefiled testimony with the Holtec cask tipover analysis: (1) the assumption of an initial angular velocity of zero, (2) the related concern that deceleration at the top of the storage cask would exceed 45g, and (3) that the PFS had failed to adequately account for the dynamic impulse resulting from displacement of the cask lid upon impact in a tipover event.[18] Finding 529.

i) Initial Angular Velocity

Dr. Resnikoff postulated that the Holtec analysis of cask tipover was inadequate because the initial angular velocity of a falling cask may be greater than zero. He testified that he had no independent knowledge of the matter, but asked the State's other ex-

[18] During the hearing, Dr. Resnikoff expressed the additional concern that a cask deformation may occur at some place on the storage cask other than the top, due to a cask impacting an already prone cask. Dr. Resnikoff testified that he had no background or experience in cask stability analyses, had not conducted cask stability analyses for the PFSF, and had no knowledge of the behavior of the storage casks from a structural engineering perspective. He had never modeled or reviewed a simulation of a storage cask drop outside of reviewing the Holtec analyses for this case. He had no knowledge of how to evaluate whether a cask lid displacement would occur during tipover. Resnikoff further testified that he did not know whether it is physically possible for one cask to fall on top of another prone cask, that he had no detailed knowledge of the behavior of the casks during a seismic event, and that he had no knowledge of how the casks might interact from a structural engineering standpoint.

perts what the angular velocity of a falling cask would be and their opinion was that the initial angular velocity could be greater than zero. Testimony from Holtec, however, showed that the behavior of the cask is characterized by tilting from the vertical resulting in a plane of precession for a certain duration in the course of the earthquake event, resulting in an oscillatory rocking motion with limited return to the vertical position until the rocking finally ends when the earthquake subsides. Thus, if the earthquake ground motions were assumed to be increased (beyond the 10,000-year MRP earthquake) to the point at which a cask would tip over, the initiating angular velocity propelling the cask towards the ground would be quite small or zero. Findings 530-533.

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

The State's concern regarding the possibility of the top of the canister decelerating at a rate larger than the 45g design limit is premised upon the initial angular velocity being significantly greater than zero. As discussed above, the casks will not tip over and if ground motions were to be increased to a level that may cause tip over, their initial angular velocity would be quite small or zero. Moreover, the 45g design limit does not represent the structural limit of the storage cask, the MPC or the fuel assemblies. Each of these components has substantial additional design margins. The fuel rod cladding can sustain at least a 63g deceleration before damage is likely to occur. Containment of the fuel within the MPC would not be lost at deceleration forces at least as high as 300g, as evidenced by the results of a Holtec MPC drop analysis. Thus, the deceleration limit imposed on the top of the canister in the HI-STORM 100 FSAR is not relevant to radiological dose consequences for a hypothetical, beyond design basis cask tipover event at the PFSF. Decelerations would have to exceed 63g before there was a concern regarding the possible effect of such a deceleration on the fuel assemblies contained in the MPC. Additionally, large margins of safety built are into the design of the the MPC such that it could withstand much larger decelerations before the confinement function of the MPC would be compromised. Findings 534-537.

iii) Cask lid displacement

Dr. Resnikoff expressed a concern that a tipover could cause additional dynamic impulses to the structure of the cask due to a 4.9 inch displacement of the cask lid. This testimony misinterpreted the results of the HI-STORM cask tipover analysis in several respects. First, it assumed that the reported displacement was displacement of the cask lid relative to the cask body, when, in fact, the cask lid and the cask body move together, not relative to one another, so that the 4.9 inches of displacement applies to both the cask lid and the cask body. Second, Dr. Resnikoff mistakenly assumed that the dynamic force due to the displacement of the cask lid and cask body would not be adequately taken into account in the Holtec analysis, when in fact any such dynamic forces due to the impact of the cask lid or body are included in the modeled behavior. Third, the effect on the canister welds of any such forces are considered in the tipover model and no deleterious effects to the welds occur during a hypothetical tipover event. Finally, to the extent that damage to a cask could hypothetically be caused by a tipover, the analysis of the effects on the storage cask demonstrate that any small, localized deformations will occur at or within one foot of the top of the cask, where radiation dose consequences are the least significant. Findings 538-539.

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

Dr. Resnikoff noted four differences between the Holtec CoC and site-specific conditions at the PFSF that allegedly result in a failure of PFS to accurately quantify the consequences of a design basis earthquake at the PFSF: (1) differences in ground motions, (2) differences in occupancy time, (3) failure to address the thirty-three hour corrective action time limit in the event of a 100% blockage of air inlet ducts; and (4) failure to calculate the dose consequences due to the tipover of an entire field of storage casks. However, as Dr. Resnikoff admitted, where site-specific analyses have been conducted, those site-specific analyses render any corresponding analyses in the CoC immaterial. Findings 512-514.

i) Design Basis Ground Motion

Because PFS conducted a site-specific cask stability analysis based on the ground motions present at the PFSF site, all parties agreed that any difference between those ground motions and those fro the the cask stability analysis undertaken in the CoC are immaterial. Finding 515.

ii) Occupancy Time

The PFS site-specific analysis for radiation dose levels uses a 2,000 hours/year occupancy time for calculating normal operating doses, whereas the HI-STORM CoC uses 8,760 hours/year occupancy time to calculate the normal operating dose. 10 C.F.R. § 72.104(a) provides limits to the annual dose equivalent to any real individual who is located beyond the controlled area, not to a hypothetical person beyond the controlled area. Occupancy time for normal operating conditions is thus determined using a real person standard, which takes into account the site-specific circumstances at a facility. This approach is consistent with regulatory guidance. Due to land use patterns surrounding the PFSF site, the use of a 2,000 hours per year occupancy time is conservative. Similar site-specific factors would be used to estimate doses arising from postulated accident conditions. Findings 516-519.

iii) Relevance of Thirty-Three Hour Time Period for Corrective Action of Complete Air Inlet Blockage Under HI-STORM 100 CoC

The thermal analysis used to support the HI-STORM 100 CoC provides that in the event of a 100% blockage of the air inlet ducts, the short term temperature limit of the concrete would be expected to be reached in thirty-three hours. The thirty-three hour period for correcting 100% air duct blockage was based on the requirement that the casks be visually inspected every twenty-four hours, allowing an additional eight hours for corrective action to be taken. The thermal analysis that was used in the HI-STORM 100 CoC makes the unrealistic, conservative assumption that no heat transfer to surrounding air will occur when the ducts are blocked. In effect, the calculation presumes that the cask not only has its air inlet ducts completely blocked, but that it is shrouded so as to prevent any heat transfer. Only under those unrealistic conditions would the short-term temperature limit of the concrete be reached in thirty-three hours.

In reality, 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. Moreover, exceedance of the short-term temperature limit of the concrete would not significantly affect public health and safety, because it would neither reduce shielding effectiveness nor affect containment of the spent fuel within the storage cask. Dr. Resnikoff admitted that he was not concerned about the public health and safety implications of a thirty-three hour cask tipover event, but rather was concerned about a tipover event lasting for years. Findings 520-523.

iv) Multiple Cask Tipover

PFSF conducted a non-mechanistic cask tipover analysis, examining the radiological dose consequences resulting from the hypothetical tipover of a single cask. The results of the analysis, as described above, demonstrated that such a tipover had no effect on MPC confinement and could cause only localized damage to the radial concrete shield and outer steel shell of the cask, with no increase in the radiological doses at the site boundary. Dr. Resnikoff questioned whether this adequately accounted for dose consequences if one assumed that multiple casks tipped over at the PFSF. 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. Holtec also determined that the radiation doses resulting from a multiple cask tipover would be essentially the same as those for the casks standing upright during normal operations, for which an annual dose exposure of 5.85 mrem was calculated. There is, therefore an almost three orders of magnitude margin of safety in the multiple cask tipover scenario with respect to the 5 rem accident limit. Findings 508-510.

Further, the Holtec dose calculations used numerous conservative assumptions regarding the fuel contained in the storage casks, including: (1) all 4,000 casks contained fuel with a burnup of 40,000 MWD/MTU and a cooling time of 10 years, (2) assuming that the fuel assemblies inside the casks have the highest gamma and neutron dose radiation source term, and (3) assuming that the fuel has been subject to a single irradiation cycle. In each of these instances, the assumption is either physically impossible (e.g., all fuel at the PFSF cannot be of the same burnup level) or at least unrealistic (e.g., all fuel has been subject to just one irradiation cycle). These conservatisms would reduce the calculated doses by more than half. Findings 512.

Thus, even under such an unrealistic multiple-cask tipover scenario, the 5 rem radiological dose limit of 10 C.F.R. § 72.106(b) would never be reached, even in the absence of corrective measures. Such measures, such as the construction of an earthen berm to provide shielding at the site boundary, would be relatively easy to take. Finding 553.

v) State Estimation of Radiological Dose Consequences of a Worst Case, Beyond Design Basis Accident at the PFSF

Dr. Resnikoff's prefiled testimony contained two radiation dose calculations: an estimation of the gamma dose coming out of the bottom of eighty tipped-over storage casks, with their bottoms facing the OCA boundary; and an estimation of the neutron dose from a cask based on the amount of "water evaporated" from the concrete shielding. Both calculations contained numerous errors (nine separate errors in total), which Dr. Resnikoff needed to correct at different points both before and during the course of his oral testimony.

Each of these errors resulted in an overestimation of radiation dose consequences for Dr. Resnikoff's unrealistic, worst case tipover scenario. Findings 540-548. Once those errors are corrected, the result is that the radiological consequences of a seismically induced, beyond-design-basis cask tipover event never reach the five rem accident dose limit, even if no remedial action is taken. Finding 548.

III. FINDINGS OF FACT

A. Section C of Contention Utah L/QQ

1. Soils Characterization Issues

1. The Commission's requirements governing the characterization of subsurface soils for an ISFSI are set forth in 10 C.F.R. Part 72. In general, 10 C.F.R. § 72.90 requires an evaluation of site characteristics that may directly affect the safety or environmental impact of the proposed facility. Specific requirements for the characterization of the subsurface soils are defined in 10 C.F.R. § 72.102. See

NRC Staff Testimony of Goodluck I. Ofoegbu Concerning Unified Contention Utah L/QQ, Part C (inserted into the record after Tr. 11001) [hereinafter "Ofoegbu Dir."] at A5.

2. 10 C.F.R. § 72.102(c) states: "Sites other than bedrock sites must be evaluated for their liquefaction potential or other soil instability due to vibratory ground motion." Additionally, 10 C.F.R. § 72.102(d) states: "Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading." See Ofoegbu Dir. at A5.

3. Section C.1 of Contention Utah L/QQ alleges that:

> ***The Applicant has not performed the recommended spacing of borings for the pad emplacement area as outlined in NRC Reg. Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants, Appendix C."*** PFS Exh. 237 at 5 (page 2 of the contention).[19]

4. Notwithstanding the implicit assumption in Section C.1 of the contention, NRC Reg. Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants" is not a binding regulatory requirement (and not even a guidance document) for ISFSIs, but only a guidance document issued by the NRC Staff with respect to soils investigations for the foundations of nuclear power plants. The applicable regulatory guidance document for Part 72 facilities, which is NUREG-1567, does not provide any guidelines on the number or placement of borings for foundation analyses. Trudeau/Wissa Dir. at A20.

5. Nuclear power generation facilities have larger and more heavily loaded foundations than those of the structures at the PFSF. Joint Testimony of Paul J. Trudeau

[19] The text of Contention L/QQ, as stipulated by the parties, was introduced into evidence as PFS Exh. 237.

and Anwar E. Z. Wissa on Section C Of Unified Contention Utah L/QQ (inserted into the record after Tr. 10834 and/or Tr. 11724) [hereinafter "Trudeau/Wissa Dir."] at A20; Rebuttal Testimony of Paul J. Trudeau to Testimony of State of Utah Witness Dr. Stephen F. Bartlett on Section C of Unified Contention Utah L/QQ (Soils Characterization) (inserted into the record after Tr. 11954) [hereinafter "Trudeau Soils Reb."] at A4.

6. Nuclear power plants also have several categories of interconnected safety-related systems and components, such as buried piping and electrical power and control systems, which are sensitive to movements of the ground and the enclosing structures. By contrast, ISFSIs have no such interconnected systems. Trudeau/Wissa Dir. at A20; Trudeau Soils Reb. at A4.

7. For the above-cited reasons, the guidance in Reg. Guide 1.132 is not directly applicable to ISFSIs, such as the PFSF. Trudeau Soils Reb. at A4; Trudeau/Wissa Dir. at A20. In fact, Dr. Bartlett acknowledged that Reg. Guide 1.132 is guidance and not applicable to ISFSIs. Surrebuttal of Dr. Steven Bartlett to PFS Witness Paul Trudeau's Rebuttal Testimony on Section C of Unified Contention L/QQ (inserted into the record after Tr. 11982) [hereinafter "Bartlett Soils Surrebuttal"] at R4.

8. Even if the guidance in Reg. Guide 1.132 were to apply, the document indicates that its recommendations "should be tempered with professional judgment. Alternative and special investigative procedures that have been derived in a professional manner will be considered equally applicable for conducting foundation investigations." PFS Exh. 234, at 1.132-1.

9. Central to the PFS geotechnical investigations program has been the drilling of boreholes to investigate the soil properties. The State is satisfied with the suffi-

ciency of the borings conducted by PFS for the soils under the Canister Transfer Building ("CTB"). However, it contends that PFS failed to perform a sufficient number of borings for the pad emplacement area because it used an approximate spacing of about 221 feet instead of the 100 feet recommended in NRC Reg. Guide 1.132. State of Utah Testimony of Dr. Steven F. Bartlett on Unified Contention Utah L/QQ (Soils Characterization) ("Bartlett Soils Dir.") at A17.

10. PFS elected to follow the guidance in Reg. Guidance 1.132 with respect to the borings in the Canister Transfer Building ("CTB") because that building is somewhat analogous to a nuclear power plant structure. For the storage pads, however, PFS exercised professional judgment and developed a subsurface investigation program that combined the drilling of boreholes with other activities to the extent warranted by site conditions and the size, loading, and isolation of the storage pads. Trudeau Soils Reb. at A4. Dr. Bartlett acknowledged that the 100 foot spacing called for in the Regulatory Guide was not a hard and fast rule, and that it was appropriate to exercise judgment in deciding how many borings should be conducted. Bartlett Soils Surrebuttal at R4; Tr. 11854-55 (Bartlett).

11. The initial geotechnical investigations at the PFSF site were performed in late 1996. The results of those initial investigations were reflected in the initial version (Revision 0) of the Safety Analysis Report ("SAR") for the PFSF, which was filed in June 1997. Trudeau/Wissa Dir. at A11.

12. PFS performed an initial set of borings in 1996 in the pad emplacement area, following a uniform grid-like pattern, with the borings spaced approximately 600 feet apart and covering the entire area. Trudeau/Wissa Dir. at A11. Such a grid was subject to supplementation with additional borings, should anomalous or ir-

regular conditions be encountered; however, no such conditions were identified. Trudeau Soils Reb. at A4.

13. The initial set of borings served to establish that the soil properties were reasonably uniform across the pad emplacement area of the PFSF site. Trudeau Soils Reb. at A4.

14. As the initial borings were made, standard penetration tests were performed that provided estimates of soil strength and compressibility and allowed visual inspection of samples and index property testing of the samples in the laboratory. The "blow count" values required to drive the standard split-spoon sampler into the soil at various depths were consistent across the pad emplacement area, confirming that the subsoil characteristics are uniform and consistent across the pad emplacement area. Trudeau Soils Reb. at A4. Based on these initial results, PFS confirmed that it was sufficient to drill boreholes in a uniform grid across the entire pad emplacement area, so that all sections of the area were covered. Id.

15. After the initial borings, PFS performed considerable additional soil investigations, including borings in the CTB area and a series of cone penetration test soundings to better assess soil strength and compressibility. Cone penetration tests are conducted using a device with an instrumented conical tip that is pushed into the soil and which provides an essentially continuous record of the soil strength by tracking the force required to advance the cone through the soil. Tr. 11727-29 (Trudeau). The device also has an instrumented sleeve that advances as the cone tip moves downward and measures the force required to overcome the friction acting on the sleeve and move the sleeve into the ground. Id.

16. In 1999, PFS drilled and sampled 12 additional borings in the CTB area and performed 39 cone penetration tests (16 of which included measurements of pressure

and shear wave velocities in addition to the penetration resistance data), and 18 dilatometer soundings. Trudeau/Wissa Dir. at A11.

17. The cone penetration tests performed in 1999 yielded essentially the same value of tip resistance for comparable depths at various locations across the pad emplacement area, indicating again that the stratigraphy across the site is uniform. Trudeau Soils Reb. at A4.

18. The results of the geotechnical investigations conducted by PFS are presented in Section 2.6 and Appendix 2A of the SAR, as revised through April 2001 (Rev. 22). That section, 219 pages long plus attachments and appendices, presents a comprehensive description of the various investigations that have been conducted. It includes geologic maps, profiles of the site stratigraphy, and discussions of structural geology, geologic history, and engineering geology. Trudeau/Wissa Dir. at A11.

19. Figure 2.6-5 of the SAR includes 14 sheets of "foundation profiles" that depict the composition of the PFSF subsoil layers at various locations in the pad emplacement area, and Figures 2.6-20 through 2.6-22 of the SAR present foundation profiles under the CTB. These profiles provide a wealth of geotechnical information and cover all safety-related structures and encompass all borings made by PSF in the vicinity of those structures. Trudeau/Wissa Dir. at A11; PFS Exh. 233, 233A.

20. The locations of the borings made to study subsurface conditions at the PFSF site are summarized in three location plans (Figures 2.6-2, 2.6-18, and 2.6-19 of the SAR), which permit correlating the locations of the borings with those of the cone penetration tests and the geological samplings performed by Geomatrix. Trudeau/Wissa Dir. at A11; PFS Exh. 235.

21. The composition of the soils at the PFS site has been well established through the investigations performed by PFS. Tr. 11835 (Bartlett). It is undisputed that the soils below thirty feet or so are dense and have significant strength and very low compressibility, so they are of no concern from the geotechnical standpoint. Tr. 11832-33 (Bartlett). This underlying layer is identified in the Staff's Safety Evaluation Report ("SER") as "Layer 2". Ofoegbu Dir. at A8.

22. The top 30 feet or so of subsoil are relatively compressible soils identified in the SER as "Layer 1." Ofoegbu Dir. at A8. Layer 1 in turn consists of several sub-layers of soils. At the top surface, there is a thin layer of aeolian silt. Below that, there is a three to ten feet layer of a silty clay and clayey silt variously described as "Layer 1B", "Upper Lake Bonneville Deposits" or "Layer 2", and which shall be referred to herein as Layer 1B. Trudeau/Wissa Dir. at A14; Ofoegbu Dir. at A8; Tr. 11733-35, 11748-49 (Trudeau). All parties agree that the Upper Lake Bonneville Deposits are, relatively speaking, the least strong and the most compressible soils in the profile. See, e.g., Tr. 11749 (Trudeau); Tr. 11788-90 (Ofoegbu); Tr. 11834-35 (Bartlett). Beneath Layer 1B is a ten-foot layer referred to as Lower Lake Bonneville Deposits or "Layer 1C," which is siltier, less clayey and stronger than the Upper Lake Bonneville Deposits. Tr. 11748 (Trudeau); Tr. 11836 (Bartlett). Underneath Layer 1C is a three to five foot layer of silty clay and clay silt layer, similar to but stronger than the layer 1B material. Tr. 11748-49 (Trudeau).

23. A determination was made after the initial tests that the soil properties at the PFSF site are reasonably uniform in the horizontal direction (that is, across the various site locations). Trudeau/Wissa Dir. at A11, A12; Tr. 11772 (Trudeau); Tr. 11784-

85 (Ofoegbu). Layer 1B is particularly uniform across the site. PFS Exh 233, 233a; Tr. 11816 (Ofoegbu); Tr. 11884-85 (Bartlett).

24. The horizontal consistency of the materials at the site was further demonstrated by the cone penetration test data, which show that the upper soil layers have fairly uniform properties across the pad emplacement area and beneath the CTB. Trudeau/Wissa Dir. at A12.

25. A trench, approximately 200 feet long and 30 feet deep, was dug by PFS consultant Geomatrix Consultants, Inc. ("Geomatrix") near the center of the pad emplacement area. Data obtained from that trench confirmed that the soils in approximately the upper 30 feet of the subsoil are fairly uniform and consistent in the horizontal direction across the site. The site investigations conducted by Geomatrix for PFS are described in the Geomatrix report "Fault Evaluation Study & Seismic Hazard Assessment, February 1999." Trudeau/Wissa Dir. at A12.

26. Drawings known as "geological plates" were prepared by Geomatrix based on its site investigations. Data from the geological plates correlate well with the data on subsurface conditions presented in the foundation profiles developed by PFS. Comparison of the Geomatrix plates with the foundation profiles in SAR Fig. 2.6-5 demonstrates that the nature, location, and thickness of the various layers of the profile are essentially the same, thus corroborating the foundation profile data. Trudeau/Wissa Dir. at A12.

27. The PFSF boring program determined that the pad emplacement area subsurface conditions are uniform, so that they conform to the general guidance in Reg. Guide 1.132 (PFS Exh. 234), which states at p. 1.132-3:

> Subsurface conditions may be considered favorable or uniform if the geologic and stratigraphic features

> to be defined can be correlated from one boring or sounding location to the next with relatively smooth variations in the thicknesses or properties of the geologic units. An occasional anomaly or a limited number of unexpected lateral variations may occur. Uniform conditions permit the maximum spacing of borings for adequate definition of the subsurface conditions at the site. (Footnote omitted).

Trudeau/Wissa Dir. at A20. Because of the uniform site conditions, there is no need for a denser set of borings. Id. at A11, A20. There is no reason to believe that a denser set of borings would have yielded any different results from those that PFS obtained. Id.

28. It is therefore appropriate to characterize the PFSF site as "uniform" and thus, if the guidance in Reg. Guide 1.132 is to be followed, a maximum spacing of borings is sufficient for the adequate characterization of the subsurface conditions. Trudeau/Wissa Dir. at A20.

29. The soils investigations performed at the PFSF are thus sufficient to properly characterize the site from the geotechnical standpoint and demonstrate that the soil conditions at the PFSF site are adequate for the proposed foundation loadings. Trudeau/Wissa Dir. at A11; Ofoegbu Dir. at A10.

30. Section C.2.a of Contention Utah L/QQ alleges that:

> ***The Applicant's sampling and analysis are inadequate to characterize the site and do not demonstrate that the soil conditions are adequate to resist the foundation loadings from the design basis earthquake in that:***
>
> ***a. The Applicant has not performed continuous sampling of critical soil layers important to foundation stability for each major structure as recommended by Reg. Guide 1.132 Part C6, Sampling.*** PFS Exh. 237 at 2 (5).

31. For the same reasons discussed above, the guidance in Reg. Guide 1.132 with respect to the taking of continuous soil samples is inapplicable to an ISFSI. Trudeau/Wissa Dir. at A21; Trudeau Soils Reb. at A7. Even though the recommendations in the guide are not applicable to the PFSF, the sampling conducted by PFS in the pad emplacement area through the use of cone penetration tests, which are continuous through the upper 30 feet of the soil profile, was consistent with the guide's recommendations. (The soils below the upper 30 feet are much stronger and less compressible than those above, consequently continuous sampling of the deeper soils was not required.) Trudeau Soils Reb. at A6; Ofoegbu Dir. at A9; Tr. 11729, 11773 (Trudeau); Tr. 1864 (Bartlett).

32. PFS also conducted cone penetration tests and performed continuous sampling of soils within the upper 30 feet of the soil profile in borings that were drilled in the CTB area. Trudeau Soils Reb. at A8. Thus, the PFS sampling program was consistent with the guidance of Reg. Guide 1.132 with respect to continuous sampling of the critical soil layers in the areas of importance to safety at the PFSF site. Trudeau Soils Reb. at A7; Ofoegbu Dir. at A12.

33. The State asserts that the sampling program conducted by PFS does not meet the guidance in Reg. Guide 1.132 because the continuous measurements taken by the cone penetrometer are not "sampling" since no soil samples are recovered for laboratory testing. Tr. 11868 (Bartlett). There is, however, no basis for making such a distinction. The purpose of the recommendation in Reg. Guide 1.132 that continuous sampling be conducted is to identify "[r]elatively thin zones of weak or unstable soils [that] may be contained within more competent materials and may affect the engineering characteristics or behavior of the soil or rock." PFS Exh. 234 at 1.132-5. The soils characterizations conducted by PFS, both through

the drilling of boreholes and the performance of cone penetration tests, established that no such zones of weak or unstable soils exist at the pad emplacement area or under the CTB. Trudeau Soils Reb. at A8. Therefore, the objectives of Reg. Guide 1.132 with respect to continuous soil sampling have been achieved. Trudeau Soils Reb. at A8; Ofoegbu Dir. at A12.

34. Section C.2.b of Contention Utah L/QQ states:

> ***The Applicant's sampling and analysis are inadequate to characterize the site and do not demonstrate that the soil conditions are adequate to resist the foundation loadings from the design basis earthquake in that:***
>
> * * * *
>
> ***b. The Applicant's design of the foundation systems is based on an insufficient number of tested samples, and on a laboratory shear strength testing program that does not include strain-controlled cyclic triaxial tests and triaxial extension tests.*** (PFS Exh. 237 at 2(5))

35. In subsection C.2.b of Contention Utah L/QQ, the State raises two separate issues: whether the number of soil samples tested by PFS in the laboratory is sufficient to properly characterize the soils and whether the laboratory soils tests performed by PFS should have included cyclic triaxial tests and triaxial extension tests.

36. It is undisputed that the main characteristic of soils from the seismic standpoint is their undrained shear strength, both with respect to horizontal and vertical loadings. Tr. 11839-40, 11933 (Bartlett). Shear strength is easily established through laboratory testing. Tr. 11840, 11933-34 (Bartlett).

37. The State does not object to the manner in which PFS conducted its laboratory tests for determining the soil shear strength parameters. Tr. 11840-41 (Bartlett). However, the State asserts that PFS's determination of the minimum shear strength of the soil is inadequate due to undersampling, because PFS estimated

the minimum horizontal shear strength of the soils in the pad emplacement area by performing laboratory tests on three specimens taken from a single soil sample. Tr. 11934-35 (Bartlett).

38. The soil sample used by PFS to measure the minimum horizontal strength of the soils in the pad emplacement area was obtained from the weakest portion of the weakest layer (Layer 1B) of the soil profile. This sample also exhibits the highest void ratio of all the samples tested in the pad emplacement area (signifying lowest density and hence lowest strength), and it was taken from the quadrant in the pad emplacement area that had been determined to have the lowest soil strength. Tr. 11767-71 (Trudeau); Trudeau Soils Reb. at A9.

39. PFS contractor ConeTec., Inc. ("ConeTec") took continuous measurements of cone penetrometer tip resistance at 37 locations in the pad emplacement area of the PFSF site. The results of those measures are presented graphically in the foundation profiles prepared by PFS, such as PFS Exh. 233A. ConeTec also provided tables of numerical values of tip resistance versus depth, which recorded the actual measurements of tip resistance and allow a numerical correlation to be drawn between the measured tip resistance and the undrained shear strength of the soil at the various locations in the soil profile. Tr. Tr. 11772-73, 11955-62 (Trudeau); Tr. 11789-91, 11817-18 (Ofoegbu). The value of undrained shear strength that can be derived from the cone penetration test result for the lowest tabulated value of tip resistance corresponds almost exactly to the value of shear strength measured in the laboratory by PFS for the sample it selected for that purpose. PFS Exh. 238; Tr. 11960-11962 (Trudeau). Therefore, the cone penetration test results confirm that the value of minimum shear soil strength determined by PFS

in laboratory tests is indeed the minimum value of undrained shear strength found in the pad emplacement area.

40. The undrained shear strength measurements obtained from the cone penetration tests performed by ConeTec in the area from which a soil sample was taken for strength determination in the laboratory are consistent with the values obtained in the laboratory tests. PFS Exh. 238; Tr. 11955-11962 (Trudeau). Thus, cone penetration measurements provide independent confirmation of the validity of the laboratory test results.

41. Because of the uniformity of the soils in the horizontal direction, the manner in which the test sample used for determining the minimum value of undrained shear strength was selected, and the confirmation provided by the cone penetration test measurements, it is reasonable to conclude that the value of undrained horizontal shear strength used by PFS represents that of the weakest soils found at the pad emplacement area at the PFSF site. Tr. 11772 (Trudeau).

42. Dr. Bartlett asserts that there can be considerable horizontal variability in the shear strength exhibited by the Upper Lake Bonneville soils across the pad emplacement area and that there may be some location at which the shear strength may be considerably below than the 2100 pounds per square foot ("2.1 ksf") obtained by PFS in its laboratory tests. See, e.g., Bartlett Soils Dir. at A26. The only evidence provided by Dr. Bartlett in support of his position are some tracings he made with markers of the cone penetration tip resistance plots, taken off enlarged photocopies of the data plotted in the foundation profiles in SAR Fig. 2.6-5. State Exh. 99; Tr. 11893-99 (Bartlett). Those tracings, however, are too crude to have evidentiary value. Trudeau Soils Reb. at A11; PFS Exh. 236. Resorting to manual plots like Dr. Bartlett's is unnecessary because a report pro-

vided by ConeTec includes tabulations of the actual values of cone penetration tip resistance measured in the tests. See PFS Exh. 238. In fact, the foundation profiles show that the measured cone penetrometer tip resistance varies as one moves downwards (even within a given soil layer) but is remarkably uniform for a given depth from one location to another. PFS Exh. 233A; Trudeau Soils Reb. at A11.

43. Assuming (despite the considerable evidence to the contrary) that such variability existed, there is no basis for asserting that the value used by PFS is not the minimum shear strength of the soils in the pad emplacement area. Even if soils of lower strength were to exist in the pad emplacement area, the conservatisms incorporated into the PFS analyses and design (discussed below) would more than compensate for the difference in that hypothetical lower strength and that utilized by PFS in its analyses.

44. The second part of Section C.2.b of Contention L/QQ asserts that the laboratory soils tests performed by PFS should have included strain-controlled cyclic triaxial tests and triaxial extension tests. The strain-controlled cyclic triaxial tests are intended to measure the properties of the soils (shear modulus and damping) versus shear strain at high strain levels under dynamic loading conditions. Trudeau/Wissa Dir. at A24. Triaxial extension tests are used to assess the bearing capacity of soils by causing them to fail under tension. Trudeau/Wissa Dir. at A27.

45. While PFS did not conduct strain-controlled cyclic triaxial tests, it performed resonant column tests, which are a form of strain-controlled cyclic triaxial tests. Indeed, they are the only form of strain-controlled testing that is recommended in Appendix B, "Laboratory Test Methods for Soil and Rock," to US NRC Regulatory Guide 1.138, "Laboratory Investigations of Soils for Engineering Analysis

and Design of Nuclear Power Plants" for use in developing curves of shear moduli and damping versus shear strain. Trudeau/Wissa Dir. at A26. The resonant column test results can be readily extrapolated to establish the behavior of site soils at higher strains than those covered by the tests, so that all the strains potentially of interest are covered. Id. Therefore, strain-controlled cyclic triaxial tests to measure shear moduli and damping at higher levels of strain than those measured in the resonant column tests are not required. Id.; Tr. 11736-39, 11759-62 (Trudeau).

46. The site response analyses performed by Geomatrix established that the layer of soil exhibiting greatest effective shear strain is Layer 1B. For the soils in that layer, the effective shear strains under design basis seismic loadings are within the range of strains measured directly in the resonant column tests. For that reason, additional strain-controlled cyclic triaxial tests are unnecessary. Trudeau/Wissa Dir. at A26; Tr. 11736 (Trudeau).

47. In addition, PFS conducted stress-controlled cyclic triaxial tests to determine the collapse potential of soil. The results of the tests did not show any degradation of the shear strength of the samples throughout 500 cycles of loading at extremely high cyclic stress ratios. The resulting cyclic strains were very small, indicating an essentially elastic response throughout the tests and demonstrating that there is no strength degradation for these soils due to even higher levels of cyclic stress than those experienced during a design basis earthquake. Thus, strain-controlled cyclic triaxial tests are unnecessary. Trudeau Soils Reb. at A12; Tr. 11791-93 (Ofoegbu).

48. The State expert, Dr. Bartlett, agreed that if one can be assured that there is no marked decrease in shear strength at high levels of strain, the concern about char-

acterizing the dynamic properties of the soil at high strain levels is of no consequence. Tr. 11992-93 (Bartlett). He characterized the testing that PFS conducted with respect to this issue at a "C-minus" level, meaning that knowledge in this area could be improved, but the failure to conduct the strain-controlled triaxial tests was not a fundamental flaw in PFS's program. Id.

49. As noted above, the State also faults PFS for failing to conduct triaxial extension tests to assess the bearing capacity of soils by causing them to fail in tension. The purpose of such tests would be to determine the degree of anistropy in the foundation soils. State of Utah Testimony of Dr. Steven F. Bartlett on Unified Contention Utah L/QQ (Soils Characterization) ("Bartlett Soils Dir.") at A32. Dr. Bartlett asserts that if significant anistropy is present, then the use of triaxial compression tests may overestimate the shear resistance along the potential failure plane. Id.

50. The vertical shear strength obtained by PFS in its triaxial compression tests for the pad emplacement area is 2.2 ksf, and the horizontal shear strength as obtained in the direct shear tests is 2.1 ksf, so the degree of anisotropy exhibited by the PFSF site soils is slight, if any. Tr. 11973 (Trudeau); Tr. 12021 (Ofoegbu).

51. The soil failure mechanism is a composite of failures along horizontal and vertical surfaces and is adequately represented by either the horizontal or vertical shear strengths determined by laboratory test results and field measurements. Staff Exh. ZZ; Tr. 12017-21 (Ofoegbu). Therefore, the effects of anisotropy are insignificant.

52. Dr. Bartlett asserts that performing triaxial extension tests is necessary to properly assess the bearing capacity of the soils beneath the storage pads. Bartlett Soils Dir. at A32. However, such tests typically are not performed to assess the bearing

capacity of foundations, nor are they mentioned in Appendix B, "Laboratory Test Methods for Soil and Rock," of Reg. Guide 1.138. Trudeau/Wissa Dir. at A27. Moreover, the minimum factor of safety against bearing capacity failure of the storage pads was computed by PFS using many conservative assumptions, including among others declining to use, as is customary, the average shear strength of the soil through a depth of 30 ft. below the base of the pads to determine the bearing capacity. Ofoegbu Dir. at A8. If this and other conservatisms in the analysis were removed, the calculated minimum factor of safety against bearing capacity failure of the storage pads would be well in excess of 3. Trudeau Soils Reb. at A9. Therefore, the concerns about soil anisotropy are inconsequential and the asserted need for triaxial extension tests does not exist.

53. Section C.3.a of Contention Utah L/QQ alleges that:

> ***The Applicant has not adequately described the stress-strain behavior of the native foundation soils under the range of cyclic strains imposed by the design basis earthquake.*** (PFS Exh. 237 at 2(5)).

In this contention, the State asserts that the Applicant has relied on simple pseudo-static analyses to calculate the factor of safety against sliding and bearing capacity of the foundations for the pads and CTB and that such simple analyses do not consider the magnitude of the cyclic strains imposed by the earthquake and the effects that these cyclic strains have on the soil's shear strength properties and potential interaction with adjacent structures. Bartlett Soils Dir. at A33.

54. To the extent that this issue raises soil-structure interaction concerns, those are addressed in the discussion of Section D below. To the extent that the State claims that PFS has not performed strain-controlled, cyclic triaxial testing at large strains to show that the shear modulus and damping values used in development

the design basis ground motion are appropriate, this portion of Section C of Contention Utah L/QQ is duplicative of the concerns raised in Section C.2.b. As discussed above, the shear strains imposed on the specimens during the resonant column tests that PFS performed were greater than the actual shear strains that the soils will experience during the design basis earthquake. Trudeau/Wissa Dir. at A28. Therefore, the potential strains that the soils may experience are properly assessed by the tests performed by PFS. Ofoegbu Dir. at A18.

55. The State raised, in the prefiled direct testimony of Dr. Bartlett on soils, a concern that PFS may have used unconservative estimates of the undrained shear strength in the dynamic bearing capacity analyses of the CTB because PFS computed the undrained shear strength test on samples obtained more than 1,000 feet away from the CTB. Bartlett Soils Dir. at A29. In rebuttal, PFS witness Paul Trudeau explained that the undrained strength used in the bearing capacity analyses of the CTB was actually developed based on the summary plot of all of the triaxial tests that were performed on samples of soils obtained from the PFSF site, and the value chosen for the bearing capacity analyses was a reasonable lower-bound of those values. Trudeau Soils Reb. at A14. In addition, the margin of safety against bearing capacity failure of the CTB is 5.5. Therefore, even if an error existed because the shear strength of the soils under the building was less than the value selected by PFS, the very large margin available against bearing capacity failure of the CTB should render the State's concern inconsequential. Id. Dr. Bartlett agreed that Mr. Trudeau's explanation satisfactorily addressed his concern. Bartlett Soils Surrebuttal at R14.

2. Conclusion re Soils Characterization Issues

56. The ultimate issue with respect to soils characterization is whether the program implemented by PFS to determine the strength and other characteristics of the soils at the PFSF site provides reasonable assurance that the soil conditions are adequate for the proposed foundation loading, as required by 10 CFR 72.102(d). See Ofoegbu Dir. at A8.

57. The record shows that the soils investigations performed by PFS are comprehensive and encompass a variety of activities performed over the course of several years by PFS, ConeTec, Geomatrix and other parties. Trudeau/Wissa Dir. at A11-A12; Tr. 11725-33, 11763-65 (Trudeau). The results of these investigations correlate well with each other and show remarkable consistency in their prediction of the properties of the soils beneath the foundations of the storage pads and the CTB. Id.

58. The record also shows that the approach followed by PFS in establishing the strength and other characteristics of the soils is exceptionally conservative. Trudeau Soils Reb. at A3; Tr. 11965-68 (Trudeau). For that reason, even if the concerns raised by the State with respect to the determination of the minimum value of soil shear strength were well taken, which we do not believe to be the case, the conservatisms built into the methodology used by PFS in determining the soils properties and the factors of safety against soil failure would be sufficient to assure that the soils conditions are adequate to meet the anticipated foundation loadings. Trudeau Soils Reb. at A11.

59. We therefore agree with the conclusion reached by the Staff in its Consolidated Safety Evaluation Report ("SER") for the PFSF that "the geotechnical site characterization information presented in the SAR is adequate for use in other sections

of the SAR to develop the design bases for the Facility and perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c, d) and 72.122(b)." Staff Exh. C at 2-57. As Staff witness Ofoegbu testified, ". . . there is abundant reasonable assurance that the work that the site characterization demonstrates, that the information used for design represents the properties of the soils that would affect the behavior of the structures, systems and components important to safety at the site." Tr. 11805 (Ofoegbu).

3. Soil Cement Issues

60. Three of the remaining subsections of Section C of Contention Utah L/QQ raise concerns regarding PFS's proposed use of soil cement to provide additional strength to the soils beneath the foundations at the PFSF. The first of these subsections, Section C.3.b, alleges that:

> ***The Applicant has not shown by case history precedent or by site-specific testing and dynamic analyses that the cement-treated soil will be able to resist earthquake loadings for the CTB and storage pad foundations as required by 10 CFR § 72.102(d).*** (PFS Exh. 237 at 2(5)).

61. This subsection raises two separate, somewhat related claims: (1) First, that the use of soil cement in the manner proposed at the PFSF is unprecedented, and thus unproven, and (2) that PFS should demonstrate by testing prior to licensing that it can develop a soil cement mixture that has the desired properties called for in the design. State of Utah Testimony of Dr. Steven F. Bartlett and Dr. James K. Mitchell on Unified Contention Utah L/QQ (Soil Cement) (inserted into the record after Tr. 11033) [hereinafter ("Bartlett/Mitchell Dir.")] at A8 and A9.

62. Soil cement is a material produced by blending, compacting and curing a mixture of soil, portland cement, other possible admixtures, and water to form a hardened

material with specific engineering properties. Trudeau/Wissa Dir. at A29. Soil cement typically has far greater strength than that of the soil that is its main constituent, and thus is used to increase soil strength. Id.

63. Some soil-cement mixtures are referred to as "cement-treated soils." Referring to a particular mixture as a "soil cement" or as a "cement-treated soil" is a function of the durability of the mixture of soil, portland cement, and/or other admixtures that has been formulated. Mixtures with greater degrees of stabilization and/or durability are generally referred to as soil cement, as opposed to cement-treated soil. Soil cement is typically expected to be able to pass durability tests that measure the ability of the stabilized soil to retain its properties after long periods of exposure to the elements. Trudeau/Wissa Dir. at A32. Cement-treated soil has less strength than soil-cement and is not expected to pass durability tests. Id.

64. PFS intends to use soil cement and cement-treated soil in three different ways: (1) In the area directly underneath the concrete pads upon which the storage casks rest, cement-treated soil is to be used as a cohesive material that will be strong enough to resist the sliding forces generated by the design basis earthquake. The cement-treated soil will provide bonding with the bottom of the concrete pad above it and with the clay soils beneath, so as to transfer the horizontal earthquake forces downwards from the pad and into the underlying clay soils. (2) Soil cement is also to be used in the area around and between the cask storage pads. There, the function of the soil cement is to support the weight of the transporter vehicle that is used to deliver storage casks to the pad area. Soil cement was chosen for this application so that the soil materials would not need to be wasted and replaced with structural fill. (3) Finally, soil cement is to be placed around the Canister Transfer Building foundation mat, extending outward from the mat a dis-

tance equal to the associated mat dimension, to provide additional passive resistance against sliding forces in the event of a design basis earthquake. Trudeau/Wissa Dir. at A34.

65. Soil cement is often used for soil stabilization purposes, that is, to improve the compressive strength of the soil so that it becomes more rigid and less compressible, and to increase its resistance to sliding by virtue of its cohesive properties. Tr. 10843-44 (Wissa). At the PFSF, the design relies on the compressive strength of the soil cement to provide passive resistance to sliding of the canister transfer building, and it relies on the cohesive strength of the cement-treated soil underneath the pads to essentially bond the pads to the underlying stiff clays. Tr. 10841-42, 10845 (Trudeau). While the soil cement "frame" surrounding the storage pads provides passive resistance against sliding of the pads, PFS conservatively does not take credit for such resistance. Tr. 11965-67 (Trudeau).

66. Soil cement has been used for soil stabilization in numerous instances, both in the United States and abroad. Trudeau/Wissa Dir. at A52; Trudeau/Wissa Reb. at A1-A3; Ofoegbu Dir. at A22. For example, soil cement has been used to provide foundation strength for an office building in Tampa, Florida, a dam spillway foundation mat in Fort Worth, Texas, a number of coal handling and storage facilities throughout the United States, a nuclear power station in Koeberg, South Africa, and variety of other applications including highways. PFS Exh. HHH at Sections 2.5 and 2.6; PFS Exh. JJJ at 2.6-113, 1.6-114; Tr. 10971-72, 10974 (Trudeau). In that respect, since all uses of soil cement rely on the same mechanical properties, all prior uses of soil cement can be said to constitute precedents for its use at the PFSF. Tr. 11263 (Mitchell). The number of applications

for soil cement and the confidence in its use by the technical community continues to grow over time. Tr. 11190-94 (Mitchell).

67. In particular, soil cement was used extensively to resist lateral forces and form permanent foundations for the five highway tunnels for I-90 and I-93 that converge at the Fort Point Channel crossing of Boston's Central Artery/Tunnel Project. This is essentially the same use of soil cement that is being proposed for the PFSF. Trudeau/Wissa Reb. at A1; Tr. 10846-47 (Wissa).

68. Even if there were no precedent for the use of soil cement in the manner PFS proposes, there is no regulatory requirement that the suitability of soil cement for its intended use be demonstrated by case history precedent. Ofoegbu Dir. at A 20. And, as the State's soil cement expert acknowledged, there is no significance to an application being new; new applications for soil cement are being developed all the time, and there is nothing inherently wrong with the application that PFS proposes to make of soil cement at the PFSF. Tr. 11054, 11187 (Mitchell).

69. Soil cement and cement-treated soil are better materials to use for purposes of increasing soil stability than the granular material (e.g., structural fill) that would typically be used as an alternative because such granular material derives its strength from friction and, under seismic uplift forces, the frictional force is decreased. The cohesive strength of soil cement, on the other hand, is not affected by the reduction in normal forces that is caused by the seismically induced uplift forces. Tr. 10839-40 (Trudeau). Soil cement is also a better material than structural fill to use in areas such as the CTB, where it was desired to increase the soil's passive resistance. Tr. 10848-49 (Trudeau).

70. The second issue propounded by the State with respect to the use of soil cement at the PFSF is the asserted need to perform the necessary testing in advance of li-

censing the facility to demonstrate that the design concept can be successfully implemented. Bartlett/Mitchell Dir. at A8.

71. The design requirements for the soil cement and cement treated soil at the PFSF are as follows: (1) The cement-treated soil underlying the pads should have a minimum unconfined compressive strength of 40 pounds per square inch (psi). The cement-treated soil is required to have a thickness no greater than 2 feet and a modulus of elasticity or Young's modulus (that is, a vertical stress to strain ratio) less than or equal to 75,000 psi. (The ability of the cement-treated soil to satisfy the Young's Modulus limit is discussed below in connection with the Section C.3.e claims.) (2) The soil cement to be placed around and between the cask storage pads is to have a thickness of 28 inches (3 feet height of the pads, minus the top 8 inches, which will be filled with compacted aggregate). The soil cement adjacent to the pads should have a minimum unconfined compressive strength of at least 250 psi, in order to meet the durability (wet/dry and freeze/thaw cycle) requirements, since it will be exposed to the detrimental effects of frost. (3) The soil cement to be placed around the CTB will have a thickness of 5 feet (plus 8 inches to be filled with aggregate). It also is expected to have a minimum unconfined compressive strength of at least 250 psi, in order meet the durability requirements (wet/dry and freeze/thaw), since its upper half will be within the frost zone, and to provide the required passive resistance to sliding. Trudeau/Wissa Dir. at A37.

72. All parties agree that these design requirements can be met by the use of appropriate soil-cement mixtures. Trudeau/Wissa Dir. at A48; Tr. 10935-36, 10992 (Trudeau); Tr. 11008, 11018-19, 11021, 11023-27 (Ofoegbu); Tr. 11088 (Mitchell); Exh. 228 at 41, 53-54, 90-91, 173-76. Indeed, the State soil cement

expert testified that he knew of nothing that would preclude PFS from meeting its design objectives for the soil cement program. Tr. 11211-12 (Mitchell).

73. All parties also agree that PFS has developed a suitable program for testing the properties of the soil cement, which is embodied in the "Engineering Services Scope of Work for Laboratory Testing of Soil-Cement Mixes," PFS Exh. GGG. Trudeau/Wissa Dir. at A38-44; Tr. 11089-93, 11103-04 (Mitchell). The program will be effective in establishing whether the properties of the soil cement specified in the design have been achieved. Tr. 11266 (Mitchell).

74. The parties also agree that the program is based on appropriate industry standards, including the American Concrete Institute "State-of-the-Art Report" on Soil Cement (PFS Exh HHH), and that it includes the proper tests and suitable test methodology. Trudeau/Wissa Dir. at A45; Ofoegbu Dir. at A22; Tr. 11061 (Mitchell).

75. Finally, the parties agree that the program to which PFS has committed in the SAR (PFS Exh. JJJ at 2.6-118, 2.6-119) is reasonable and should lead to proper soil cement and cement-treated soil installations. Trudeau/Wissa Dir. at A48; Ofoegbu Dir. at A22; Tr. 11088-89, 11103-04 (Mitchell). The program – including the construction procedures it calls for – is based on well-accepted, standard practices set forth in manuals issued by organizations such as the U.S. Army Corps of Engineers and the Portland Cement Association. Tr. 10973-74 (Trudeau, Wissa).

76. There is also a fair degree of flexibility in establishing the acceptance criteria for the soil cement and cement-treated soil, as well as the tolerances that the specified material content must meet. Tr. 10945-47, 10964-65 (Wissa); Tr. 11179-81 (Mitchell). If, however, a soil cement installation failed to meet design requirements, it is most likely that it would be reworked or replaced rather than attempt-

ing to demonstrate its acceptability through analyses. Tr. 10938-40 (Trudeau); Tr. 10965-67 (Wissa).

77. PFS witnesses also testified that appropriate measures will be taken during construction to ensure that the required quality of installation and the requisite properties of the soil cement are achieved and any non-conformances are corrected. The work would be subject to oversight of both the contractor and the owner, would be subject to NRC approval, and would be required to conform with NRC requirements. Tr. 10968-69 (Wissa); Tr. 10992-93 (Trudeau).

78. Thus, the main point of disagreement between the Applicant and the Staff, on the one hand, and the State, on the other, is that the State believes that the test program to confirm that the soil cement will have the requisite properties should be completed before licensing of the facility, whereas the other parties do not think this is either required by NRC regulations or necessary. Compare Bartlett/Mitchell Dir. at A8 with Trudeau/Wissa Dir. at A53, A54 and Tr. 11017 (Ofoegbu).

79. The State pointed out the potentially adverse economic consequence that could befall PFS if, after licensing, it was determined that the use of soil cement in the manner proposed by PFS was for some reason unworkable. Tr. 11096-11100, 11104-07 (Mitchell). However, the State witnesses pointed to no regulatory rule, regulation or regulatory guidance that requires that Applicant proceed with the soil cement testing program in advance of licensing, and the Staff witness testified that, once the design requirements are established and a commitment is made to perform an appropriate testing program to demonstrate compliance with them, an applicant is free to defer testing to the post-licensing phase. Tr. 11017-18 (Ofoegbu).

80. The testimony at the hearing shows that the design requirements for the soil cement and the cement-treated soil are well established. Additionally, the PFS witnesses testified that PFS has committed to developing a soil-cement mix design using standard industry practices, and has further committed to performing a soil cement testing program in accordance with appropriate industry standards. Thus, PFS has specified the tests it intends to perform and the acceptance criteria that will be applied to the test results. As stated in the SAR, PFS is also committed to performing field testing during construction to demonstrate that it has, indeed, achieved in the field the bond strengths that are required. Trudeau/Wissa Dir. at A54. Such tests will include obtaining core samples through the pad and the underlying layers of interest, taking them to the laboratory, performing shear tests at the interfaces, and demonstrating that the shear strength along those interfaces exceeds that of the underlying clay. This will confirm that a good bond has been achieved and that the shear strength available along those interfaces exceeds the shear strength used in the sliding stability analyses. Trudeau/Wissa Dir. at A55; Tr. 10900 (Trudeau); Tr. 10963, 10971, 10981-82 (Wissa).

81. The commitments made by PFS are sufficient to provide reasonable assurance that the soil conditions at the PFSF will be adequate for the foundation loading that will be imparted by the design basis earthquake, and no additional requirements need to be imposed on the Applicant prior to licensing.

82. Section C.3.c of Contention Utah L/QQ alleges that:

> ***The Applicant has not considered the impact to the native soil caused by construction and placement of the cement-treated soil, nor has the Applicant analyzed the impact to settlement, strength and adhesion properties caused by placement of the cement-treated soil.*** (PFS Exh. 237 at 2-3(5-6).)

83. As explained in the State's direct testimony, the concern expressed in this contention is that the installation of the cement-treated soil may disturb the native Upper Lake Bonneville clays that underlie the storage pads and cause a reduction in the clays' shear strength. Bartlett/Mitchell Dir. at A31. Another potential mechanism postulated by the State that may affect the native soils is having traffic and heavy construction equipment disturbing the upper crust of the clays. Id.

84. The soil cement and the cement-treated soil to be used at the PFSF will be constructed by removing the topmost layer of soil at the PFSF site, which is a layer of aeolian silt, and mixing it with cement in the appropriate proportions, as construction proceeds across the site. Trudeau/Wissa Dir. at A34. Soil cement manufacture will likely involve mixing the soil and the cement at a processing plant onsite to ensure high quality. Tr. 10890-91 (Wissa); Tr. 10906 (Trudeau).

85. The design requires that there be a minimum of one foot and a maximum of two feet of cement-treated soil under each storage pad. There may be an area in the southeastern corner of the pad emplacement area where the eolian silt extends deep enough that, after its removal, it may be necessary to fill in below one or more of the pads to limit the cement-treated soil thickness to two feet. Tr. 10898 (Trudeau); Trudeau/Wissa Reb. at A10. In any location where this happens, PFS expects to place compacted native soils.

86. The State expressed the concern that the recompaction process could result in weakened soils. Bartlett/Mitchell Dir. at A31; Tr. 11162-65, 11184 (Bartlett). However, PFS expects to control the compaction of the clayey soils based on Modified Proctor densities, which require a high degree of compactive effort and will result in a stronger compacted clay so that the resulting recompacted clays will be at least as strong as the undisturbed clay present at the site. Id. Such

compaction efforts should prove effective for the types of soil present at the PFSF site. Tr. 11243-44 (Trudeau). The results of the compaction effort can be verified by testing. Tr. 11165-66 (Mitchell), and PFS intends to make such a verification. Tr. 10899-10900 (Trudeau).

87. Another concern expressed by the State is that the process of placing soil cement can disturb the underlying soils, particularly the soils in the vicinity of the area where construction is taking place. There are two main mechanisms by which the underlying soils may be disturbed during the placement of soil cement: exposure to the elements and deformation ("remolding") by construction equipment. Exposure to the elements will be minimized through the use of proper construction procedures and scheduling. Those procedures will require that soil excavation not take place until the first lift of soil cement is ready to be placed. That first lift of soil cement can be pushed out onto the surface of the subgrade with low ground pressure equipment that will not have an adverse impact on the underlying clay. Once in place, the first lift of soil cement will shelter the underlying soil from rain. Trudeau/Wissa Dir. at A55. If there is a heavy rainfall during construction, one of several available options will be utilized to remove excess moisture from the soil. Id.

88. The main area of concern with respect to remolding of the native soils is with respect to the cask storage pads, for which the cohesive strength of the clay under the cement-treated soil is required to provide sliding resistance. However, the pads are only about 30 feet wide. There is construction equipment that can be located on either side of the pads at the placement locations and reach out to make a cut to the final subgrade surface, if necessary. All other construction equipment

can be kept off of the exposed subgrade. Through these means, the subgrade can be sufficiently protected during the soil cement installation. Id.

89. The measures proposed by PFS can effectively protect the soils from any adverse effects from disturbance due to construction activities. Tr. 11162 (Mitchell). In addition, the clays are in fact not prone to deformation due to compaction because they are stiff, partially saturated clays lying more than 100 feet above the water table. Tr. 10899 (Trudeau). PFS also intends to demonstrate at the start of construction that the techniques to be used by the contractor that will install the soil cement will not have an adverse impact on the strength of the soils. Trudeau/Wissa Dir. at A55.

90. The State has also expressed a concern that the concrete pads and the cement-treated soil to be placed underneath them at the site may serve as an impermeable barrier that will trap moisture in the underlying soils and weaken them. Bartlett/Mitchell Dir. at A31. This, however, will not happen because the storage casks on top of the pads will provide a source of heat that will be transmitted downwards through the concrete pad and the cement-treated soil. Therefore, the area beneath the pads on which casks rest will be warmer than surrounding areas, causing moisture to migrate away from the cement-treated layer beneath the pads to the surrounding areas due to heat gradient effects. Trudeau/Wissa Dir. at A57; Tr. 11012 (Ofoegbu); Tr. 11118-19, 11150 (Mitchell). In addition, there is no mechanism for moisture to migrate towards the upper layer of the soil, given the great depth to the groundwater table at the site and the semiarid conditions in Skull Valley. Trudeau/Wissa Dir. at A56; Ofoegbu Dir. at A24.

91. More general concerns also expressed by the State about potential water infiltration into the subsoil at the PFSF site -- whether beneath the storage pads, under

the soil cement "frame" that surrounds the pads, or under the soil cement that forms the buttress around the CTB see, e.g., Tr. 11146-48 (Mitchell) -- are inconsequential. First, the mechanisms postulated by the State witness under which such infiltration could happen are improbable (e.g., continuous top-to-bottom cracking of the three-foot thick reinforced concrete, see Tr. 11134-37 (Bartlett); dropping of a cask on the pad, causing a crack that is not subsequently repaired, see Tr. 11130-34 (Mitchell); water accumulating in the permeable "bathtub" created by the eight inches of aggregate that will be placed on top of the soil cement and then filtering down through shrinkage cracks, see Tr. 11138-39 (Mitchell); snowfall accumulating on top of the aggregate, see Tr. 11141-42 (Bartlett); and separation between the CTB and the soil cement layer adjacent to it due to differential settlement, see Tr. 11153-57 (Bartlett). Some of these mechanisms are virtually impossible. For example, PFS will install berms around the pad emplacement area to direct any surface water away from the pad emplacement area; within the pad emplacement area, the site is generally sloped from south to north and from the center of the site to the edges where there are concrete-lined drainage ditches to transport the surface water to the detention pond at the north. Tr. 11233-34 (Trudeau). Accordingly, there is no potential for significant presence of standing water in the pad emplacement area following snow melt, run-off, thunderstorms, or any other mechanism. Tr. 11234 (Trudeau).

92. Second, any water that enters through a crack in the soil cement will be unlikely to penetrate all the way down to the underlying soils because the soil cement will be constructed in thin lifts, all of which will cure at different times. While each of the lifts may have its own shrinkage cracks, it is very unlikely that the cracks on each lift will line up exactly with the cracks on other lifts. Tr. 11234-35 (Tru-

deau); Tr. 11196 (Mitchell). In addition, the adhesive material used to provide bonding between successive lifts serve as a barrier against crack propagation. Tr. 11197-98 (Mitchell). And, in the area of the soil cement frame around the pads, there is a continuous layer of cement-treated soil that extends out beyond the pads that would prevent the downward passage of water. Tr. 11236-37 (Trudeau).

93. Third, if water enters the soils beneath the soil cement through cracks, it just as easily can evaporate through them during dry periods. Tr. 11196 (Mitchell). Total precipitation at the PFSF site is on the average nine inches a year. Tr. 11139-42 (Bartlett). The site thus has an arid climate, which would facilitate the evaporation of any accumulated water. Tr. 11236 (Trudeau).

94. Fourth, in addition to the unlikelihood of water infiltration through the mechanisms postulated by the State, tests performed on the soils at the PFSF site demonstrate that the strength of the soils is only minimally affected by an increase in water content. PFS Exh. 230; Ofoegbu Dir. at A22.

95. Finally, any moisture accumulation and attendant potential reduction in the shear strength of the soil would only be a localized phenomenon, which would not have a significant effect on the strength or bearing capacity of the soils underlying the storage pads or the CTB. Ofoegbu Dir. at A22; Tr. 11152-53 (Mitchell); Tr. 11157-58 (Bartlett).

96. Section C.3.d of Contention Utah L/QQ alleges that:

> ***The Applicant has not shown that its proposal to use cement-treated soil will perform as intended – i.e., provide dynamic stability to the foundation system – and the Applicant has not adequately addressed the following possible mechanisms that may crack or degrade the function of the cement-treated soil over the life of the facility:***

i) *shrinkage and cracking that normally occurs from drying, curing and moisture content changes.*

ii) *potential cracking due to vehicle loads.*

iii) *potential cracking resulting from a significant number of freeze-thaw cycles at the Applicant's site.*

iv) *potential interference with cement hydration resulting from the presence of salts and sulfates in the native soils.*

v) *cracking and separation of the cement-treated soil from the foundations resulting from differential immediate and long-term settlement.* (PFS Exh. 237 at 3(6)).

97. In this aspect of the soil cement contention, the State asserts that cracks in the soil cement and the cement-treated soil may form through a number of mechanisms. These mechanisms are said to have the potential for degrading the performance of the soil cement or cement-treated soil over the life of the PFSF facility. At the outset, however, we note that the main consequence of crack formation is the potential infiltration of moisture into the soil beneath the soil cement that surrounds the pads and the CTB. Tr. 11147-48 (Mitchell). However, as discussed above, water infiltration -- if occurring -- is not expected to have a significant adverse impact on the performance in an earthquake of the soil cement or cement-treated soil, and the underlying soils, or on the behavior of safety-related structures at the PFSF under seismic loadings.

98. Another concern raised by the State is the potential reduction in the tensile strength of the soil cement due to crack formation. Bartlett/Mitchell Dir. at A22; Tr. 11208-09 (Bartlett). However, any such loss would only occur in the cracked area, and would not constitute a total loss of tensile strength unless the crack went through the entire cross-section of the soil cement. Tr. 11300-01 (Trudeau). This is unlikely to occur. Tr. 11110-11 (Mitchell). In any event, PFS does not rely on

the tensile strength of the soil cement, so the effect, if any, of such cracking is inconsequential. Trudeau/Wissa Dir. at A60; Tr. 10933, 11296-97 (Trudeau).

99. The first mechanism for crack formation raised by the State witnesses in their testimony (although not mentioned in the text of the contention) is delamination or debonding along a soil cement lift interface or an interface with the concrete pad or the native soil during a seismic event. Bartlett/Mitchell Dir. at A23. Delamination or debonding, however, are very unlikely to develop because PFS has identified, described, and intends to use methods for achieving proper bonding between the different soil cement lifts and between the soil cement and the concrete pad and the native soil. The SAR (PFS Exh. JJJ at 2.6-114 – 2.6-117) sets forth the methodology that PFS will use to ensure that adequate bonds exist at all interfaces and preclude delamination or debonding. As indicated there, cement surface treatments, which consist of placing small amounts of cement on the interface between lifts as each lift is applied, are extremely effective in creating a bond along the interface that exceeds the shear strength of the soil cement itself. Thus, if the results of the interface strength tests that PFS is committed to performing demonstrate that such surface treatments are warranted, PFS will institute them as part of its construction procedures. Trudeau/Wissa Reb. at A6; Trudeau/Wissa Dir. at A53; PFS Exh. JJJ at 2.6-114- 117; Tr. 10910-13 (Wissa); Tr. 11103 (Mitchell).

100. Tests will also be carried out during construction to verify that proper bonding between the various surfaces has been achieved and that any shear failure that occurs will develop through the principal material (e. g., the underlying native soil) and not by failure of the bond at an interface. PFS Exh. JJJ at 2.6-114- 117; Tr. 10849, 10900, 10938 (Trudeau); Tr. 10910, 10962-63, 10971-72, 10980-82

(Wissa). The State's soil cement expert agreed that the approach that PFS intends to use is appropriate. Tr. 11129 (Mitchell).

101. Another crack formation mechanism cited by the State is shrinkage cracking during soil cement curing and drying. Bartlett/Mitchell Dir. at A23. Shrinkage cracking is a normal phenomenon in soil cement and cement-treated soil. Trudeau/Wissa Dir. at A58; Tr. 11111-12 (Mitchell). Shrinkage cracking has been extensively investigated over the years and shown to not generally affect the performance of cement-stabilized soils. Trudeau/Wissa Dir. at A58. Steps can be taken during the curing and placement process to minimize the amount of shrinkage and the potential for crack formation. Id.; Tr. 11122-23, 11196-97 (Mitchell).

102. Shrinkage and curing cracks are less likely to occur in cement-treated soil because there is not much cement in the mix and because there is a three foot reinforced concrete mat over the top of the cement-treated soil that prevents direct exposure of the cement-treated soil to the atmosphere. Tr. 11117-18 (Mitchell). On the other hand, the heat that is released by the storage casks will drive moisture away from the cement-treated soil and could stimulate shrinkage crack formation. Tr. 11118-19 (Mitchell). This effect will be significantly limited, however, by the fact that the cement-treated soil underlying the storage pad will be well cured before any cask will be set on the pad and any heat transfer begins to take place. Tr. 11238 (Wissa). Also, any additional shrinkage due to the heat transfer effect would take the form of enlargement of existing cracks, as opposed to development of new ones. Tr. 11239 (Wissa).

103. The cracks that form in soil cement and cement-treated soil due to shrinkage and curing are very narrow (fractions of an inch wide), occur at random locations, and are vertically propagating. Trudeau/Wissa Dir. at A59; Tr. 10850-51 (Trudeau);

Tr. 11239 (Wissa); Tr. 11107-08 (Mitchell); Rebuttal Testimony of Paul J. Trudeau on Section D of Unified Contention Utah L/QQ ("Trudeau Section D Reb.") at A10. Such cracking does not impair the compressive strength of the soil cement or the cement-treated soil and will not be continuous through successive lifts and interfaces. Trudeau/Wissa Dir. at A59; Tr. 11234-35 (Trudeau).

104. With respect to the passive resistance of soil cement, which is relied upon for providing resistance to sliding of the CTB, such resistance is not diminished by the presence of vertical cracks. In order for the sliding resistance to be reduced, all the cracks would have to be lined up parallel to the edge of the foundation. However, such an alignment is highly unlikely because of the random orientation of the cracks, such that some cracks line up parallel to the foundation, some perpendicular to it, and some are lined up at some angle. Tr. 10850 (Trudeau); Ofoegbu Dir. at A28, Trudeau/Wissa Dir. at A59.

105. The presence of shrinkage and curing cracks will not affect the horizontal resistance to sliding that the soil cement is capable of providing. Tr. 10933 (Trudeau). The aggregate width of the cracks will be small, and the potential effect of such cracks (even if continuous through an entire top-to-bottom cross-section of soil cement) will be to require some horizontal displacement in order for the soil to reach full passive resistance. Trudeau/Wissa Dir. at A59; Ofoegbu Dir. at A28, Trudeau Section D Reb. at A10. Such a horizontal movement of the CTB is of no consequence because there are no safety-related connections between the CTB and the surrounding yard area. Id.; Ofoegbu Dir. at A28. In addition, PFS has the opportunity to seal these cracks in the soil cement surrounding the CTB, where the soil cement is relied upon to provide passive resistance, prior to placement of the layer of compacted aggregate in the area. Id.

106. Another crack formation mechanism cited by the State is settlement cracking resulting from differential settlement at the perimeter of the pads and CTB mat foundation. Bartlett/Mitchell Dir. at A23. At the hearing, State witness Bartlett testified that such cracks may not develop as a result of differential settlement, but instead there could be the formation of a shear plane accompanied at most by a small opening or gap. Tr. 11322-25 (Bartlett).

107. The long-term settlement of the pads is estimated as 1.75" in PFS calculations, but is reduced to 0.5" if the conservatisms in the analysis are removed. Trudeau/Wissa Reb. at A7. Such a small settlement will not result in any significant cracking in the soil cement. Id.; Tr. 11125 (Mitchell).

108. Any settlement that occurs will be less pronounced at the periphery of the pads than towards the center, thus reducing the potential for formation of cracks at the interface between the pads and the soil cement. Trudeau/Wissa Reb. at A7. The same situation will exist with respect to any CTB mat settlement. Id.

109. Furthermore, the soil cement to be placed adjacent to the CTB mat is likely to be installed after most of the building has been constructed, thus minimizing the potential for differential settlements to occur at that interface between soil cement and the building mat, because much of the building settlement would have already occurred by the time the soil cement is installed. Id.

110. The State also cites frost penetration and expansion as a potential crack formation mechanism. Bartlett/Mitchell Dir. at A23. However, it is undisputed that if soil cement passes the durability tests that are part of the test program to qualify the soil-cement mix, it is unlikely that freeze-induced cracks will develop in it. Trudeau/Wissa Dir. at A62; Tr. 11142 (Mitchell). Indeed, soil cement has been used under a variety of environmental conditions more severe than those expected to be

present at the PFSF and has continued to perform satisfactorily over the course of many years. Trudeau/Wissa Dir. at A62; Trudeau/Wissa Reb. at A8.

111. With respect to the cement-treated soil under the cask storage pads, the top of the layer of cement-treated soil will be six inches below the frost level for the site; thus, it will not be exposed to freeze-thaw cycles. Trudeau/Wissa Dir. at A62. In addition, when storage casks are present, the cement-treated soil will be warmed by the heat released from the storage casks. Id.

112. The last potential crack-formation mechanism posited by the State is cracking or overstressing due to vehicle loads (e.g., cask transport vehicle). Bartlett/Mitchell Dir. at A23; Tr. 11143-44 (Mitchell). The concern is that the static and dynamic loads imposed by the heavy transport vehicles may exceed the tensile capacity of the pavement structure formed by the aggregate and the underlying soil cement. Tr. 11143-44 (Mitchell).

113. The State's soil cement expert testified that the concern about cracks being induced due to the effects of vehicle loads was something that required attention but he did not consider it to be a real problem. Tr. 11142-43 (Mitchell). This is borne out by calculations performed by PFS, which show that soil cement meeting the 250 psi design specification has a compressive strength that is three to four times greater than the loading that is applied at the surface of the soil cement by a moving, fully-loaded cask transporter. Trudeau/Wissa Dir. at A61; Tr. 11237-38 (Trudeau). Thus, the soil cement provides a firm foundation for the transporter to travel, and it will not be subject to cracking due to transporter-imparted loads. Trudeau/Wissa Dir. at A61; Tr. 11237 (Trudeau).

4. Conclusion re Soil Cement Issues

114. Based on the evidence on the record, we conclude that the program that the Applicant has developed for the testing and construction of soil cement and cement-treated soil, if properly implemented, will lead to the installation of soil cement and cement-treated soil mixes that will meet the specified design requirements and will give adequate performance for the life of the PFSF.

5. Young's Modulus Issue

115. In Section C.3.e of Contention L/QQ, the State alleges that:

> ***The Applicant has unconservatively underestimated the dynamic Young's modulus of the cement-treated soil when subjected to impact during a cask drop or tipover accident scenario. This significantly underestimates the impact forces and may invalidate the conclusions of the Applicant's Cask Drop/Tipover analyses.*** (PFS Exh. 237 at 3(6).)

116. This contention raises two separate issues: first, whether the design requirements imposed by PFS for the cement-treated soil, i.e., a minimum compressive strength of 40 pounds per square inch ("psi") and a maximum Young's modulus of 75,000 psi, can be successfully implemented (see Bartlett/Mitchell Dir. at A23); and second, whether the stability analysis of the storage casks performed by PFS should have treated the Young's modulus as a dynamic modulus. Id. While most of the testimony on the second issue was presented in connection with Section D of the contention, our discussion here will deal with both issues.

117. All parties agree that seeking to have a compressive strength in excess of 40 psi while limiting the Young's modulus to less than 75,000 psi is reasonably achievable because having cement-treated soil with relatively low strength and relatively low modulus is consistent with the anticipated performance of soil cement and

cement-treated soil and with data reported in the literature. Tr. 10915 (Wissa); Tr. 11023-25 (Ofoegbu); Tr. 11159-60 (Mitchell).

118. Questions were raised at the hearing as to how to assure that the Young's modulus of the cement-treated soil was kept to no more than 75,000 psi over the life of the PFSF facility given that the cement-treated soil continues to cure – and thus to strengthen – over time. Tr. 11216-17, 11220-22 (Mitchell). However, the greatest increase in Young's modulus with time occurs during the first 28 days of curing. Tr. 11227, 11229-30 (Mitchell); Tr. 11251-52 (Wissa). For that reason, satisfaction of the requirement of a maximum Young's modulus of 75,000 psi is to be measured after 28 days of curing to be consistent with the criterion used for the Young's modulus of the concrete storage pad. Tr. 11253 (Trudeau).

119. The State witnesses expressed a concern that the 75,000 psi Young's modulus that is to be achieved to comply with the requirements of the Holtec cask drop analysis should be a dynamic rather than a static modulus. See, e.g., Bartlett/Ostadan Section D Dir. at A24. However, at the hearing Dr. Ostadan agreed that the Holtec design intent could be satisfied by formulating a test program that established that the modulus of elasticity of the cement-treated soil did not exceed 75,000 psi while the strain level in the soil directly beneath the cement-treated soil was 1.93 percent. Tr. 7426-27 (Ostadan). Such strain levels are properly characterized as large strains. A test at those strain levels would determine the large strain Young's modulus of the cement-treated soil, and would be consistent with an evaluation conducted for the NRC (NUREG/CR-6608, "Summary and Evaluation of Low Velocity Impact Testing of Solid Steel Billets Onto Concrete Pad", February 1998). That evaluation used "static" or large-strain Young's modulus relationships and showed good correlations between those relationships and the re-

sults of actual drop tests of instrumented steel billets on concrete pads on top of soil. Singh/Soler Dir. at A55; Tr. 6003-04 (Soler).

6. Conclusion re Young's Modulus Issue

120. The State differs with the Applicant and the Staff on whether the test that will determine whether the cement-treated soil beneath the pads meets the 75,000 psi Young's modulus limit should be a "static" or a "dynamic" test. This difference, however, is only one of semantics. PFS has established as a design requirement that, whatever test is used, it must determine the Young's modulus applicable for strains in the order of approximately 2%. (This requirement is not arbitrary, since the modulus – strain relationship and other aspects of the analyses were established through NRC-sponsored, full-scale cask drop tests. State Exh. 197, Att. 3.) Therefore, the issue is one of test methodology and, as such, needs not be addressed in this proceeding. What is important is that PFS has made a licensing commitment to demonstrate by testing that the cement-treated soil to be placed under the storage pads satisfies the design requirements, and that PFS and the NRC are of the opinion that those requirements can be met. Tr. 6646 (Ofoegbu). No further showing needs to be made in this proceeding.

B. Section D of Contention Utah L/QQ

121. Section D of Contention Utah L/QQ raises a number of claims relating to the seismic design of the PFSF and the stability of the foundations of the storage pads and the CTB. The contention reads (PFS Exh. 237):

D. <u>Seismic Design and Foundation Stability</u>.

The Applicant, in its numerous design changes and revisions to the calculations, has failed to demonstrate that the structures and their foundations have adequate factors of safety to sustain the dynamic loading from the proposed de-

sign basis earthquake, and does not satisfy 10 CFR § 72.102(c) or (d) or § 72.122(b)(2) in the following respects:

1. Seismic Analysis of the Storage Pads, Casks, and Their Foundation Soils

The Applicant has not demonstrated adequate factors of safety against overturning and sliding stability of the storage pads and their foundation system for the design basis earthquake (DBE) as outlined by NUREG - 75/087, Section 3.8.5, "Foundation," Section II.5, Structural Acceptance Criteria, because of the following errors and unconservative assumptions made by the Applicant in determining the dynamic loading to the pads and foundations:

a. *In spite of proximity to major active faults, the Applicant's calculations unconservatively assume that only vertically propagating in-phase waves will strike the pads, casks and foundations, and fail to account for horizontal variation of ground motion that will cause additional rocking and torsional motion in the casks, pads and foundations.*

b. *The Applicant's calculations incorrectly assume that the pads will behave rigidly during the design basis earthquake. The assumption of rigidity leads to:*

i) *Significant underestimation of the dynamic loading atop the pads, especially in the vertical direction.*

ii) *Overestimation of foundation damping.*

c. *The Applicant has failed to provide a realistic evaluation of the foundation pad motion with cement-treated soil under and around the pads in relation to motion of the casks sliding on the pads in that Applicant's evaluation ignores:*

i) *the effect of soil-cement around the pads and the unsymmetrical loading that the soil-cement would impart on the pads once the pads undergo sliding motion,*

ii) *the flexibility of the pads under DBE loading, and*

iii) *the variation of the coefficient of sliding friction between the bottom of the casks and the top of the pads due local deformation of the pad at the contact points with the cask.*

d. *The Applicant has failed to consider lateral variations in the phase of ground motions and their effects on the stability of the pads and casks.*

e. *The Applicant's calculations for cask sliding do not address the frequency dependency of the spring and damping values used to model the foundation soils.*

f. The Applicant has failed to consider the potential for cold bonding between the cask and the pad and its effects on sliding in its calculations.

g. The Applicant has failed to analyze for the potential of pad-to-pad interaction in its sliding analyses for pads spaced approximately five feet apart in the longitudinal direction.

h: In an attempt to demonstrate cask stability, the Applicant's calculations use only one set of time histories in its non-linear analysis. This is inadequate because:

i) Nonlinear analyses are sensitive to the phasing of input motion and more than one set of time histories should be used.

ii) Fault fling (i.e., large velocity pulses in the time history) and its variation and effects are not adequately bounded by one set of time histories.

i. Because of the above errors, omissions and unsupported assumptions, the Applicant has failed to demonstrate the stability of the free standing casks under design basis ground motions. Thus, the Applicant's analyses do not support the Applicant's conclusions that excessive sliding and collision will not occur or that the casks will not tip over. 10 CFR § 72.122(b)(2) and NUREG 1536 at 3-6.

2. Seismic Analysis of the Canister Transfer Building and its Foundation

The Applicant has not demonstrated adequate factors of safety against overturning and sliding stability of the CTB and its foundation system for the design basis earthquake as outlined by NUREG - 75/087, Section 3.8.5, "Foundation," Section II.5, Structural Acceptance Criteria, because of the following errors and unconservative assumptions made by the Applicant in determining the dynamic loadings to the CTB and its mat foundation:

a. *The Applicant's calculations incorrectly assume that the CTB mat foundation will behave rigidly during the DBE. The assumption of rigidity leads to:*

   i) *Significant underestimation of the dynamic loading to the mat foundation.*

   ii) *Overestimation of foundation damping.*

b. *The Applicant's calculations ignore the presence of a much stiffer, cement-treated soil cap around the CTB. This soil cap impacts:*

   i) *Soil impedance parameters.*

   ii) *Kinematic motion of the foundation of the CTB.*

c. *The Applicant's calculations are deficient because they ignore the out-of-phase motion of the CTB and the cement-treated soil cap, which potentially can lead to the development of cracking and separation of the cap around the building perimeter.*

d. *The Applicant's calculations unconservatively assume that only vertically propagating in-phase waves will strike the CTB and its foundations, and fail to account for horizontal variation of ground motion that will cause additional rocking and torsional motion of the CTB and its foundations.*

122. As asserted in the text of the contention, the State alleges that PFS has failed to satisfy the requirements of 10 CFR § 72.102(c) and (d) and 10 CFR § 72.122(b)(2). With respect to foundation stability, 10 C.F.R. § 72.102(c) requires that an applicant must evaluate the site for its liquefaction potential or other soil instability due to vibratory ground motion. 10 C.F.R. § 72.102(d) requires that site-specific investigations and laboratory analyses show that soil conditions are adequate for the proposed foundation loading. See NRC Staff Testimony of

Daniel J. Pomerening and Goodluck I. Ofoegbu Concerning Unified Contention Utah L/QQ, Part D (Seismic Design and Foundation Stability) (inserted into the record after Tr. 6496) [hereinafter "Pomerening/Ofoegbu Dir."] at A6.

123. In addition, 10 C.F.R. § 72.122(b)(2) requires, in part, that SSCs important to safety be designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions. The design bases for these SSCs must reflect: (i) appropriate consideration of the most severe of the natural phenomena reported for the site and surrounding area, with appropriate margins to take into account the limitations of the data and the period of time in which the data have accumulated, and (ii) appropriate combinations of the effects of normal and accident conditions and the effects of natural phenomena. In addition, 10 C.F.R. §72.122(b)(2) specifies that an ISFSI should be designed to prevent massive collapse of building structures or the dropping of heavy objects as a result of building structural failure on the spent fuel or onto SSCs important to safety. Id.

1. Organization of Findings of Fact on Section D of Contention Utah L/QQ

124. The claims raised by the State in Section D of Contention L/QQ are for the most part directed against the analyses performed by PFS to demonstrate that the safety-related structures at the PFSF and their foundations have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake. We will first set forth some general issues that generally apply to the overall seismic analysis and design of the PFSF. The discussion of these issues will be followed by findings with respect to the State's claims regarding the seismic analysis of the storage pads, casks and their foundations, and then by findings

with respect to the seismic analysis of the Canister Transfer Building and its foundation.

2. General Issues Relating to the Design and Seismic Analyses for the Storage Pads, Casks and Foundation Soils

125. Following is a discussion of the background and issues related to the design of the casks and the pads on which they rest, and to the stability of the casks during a seismic event. Ultimately, the integrity of the multi-purpose canister ("MPC") contained within the casks is the most crucial concern. The MPC is classified as a Category A structure, system, and component ("SSC"). PFS SAR, Table 3.4-1. The Category A classification means that it is critical to the safe operation of the ISFSI. Lewis Dir. at A13. The storage cask itself is a category B SSC, meaning that it has a major impact on safety and its failure could indirectly result in a condition adversely affecting public health and safety. Lewis Dir. at A14 and A15. The storage pads are classified as category C SSCs, meaning that they have a minor impact on safety such that their failure would not be likely to create a condition adversely affecting public health and safety. Id.

a. Description of the PFS Cask-Storage Pad System

126. At the PFSF, the spent nuclear fuel will be stored in large storage casks resting on concrete pads. The storage cask system to be used by PFS is the Holtec International HI-STORM 100 Storage Cask System ("HI-STORM System").

127. The HI-STORM System consists of a massive cylindrical steel and concrete storage cask surrounding a multi-purpose stainless steel canister in which the spent nuclear fuel is sealed. Each cask is approximately 20 feet tall (239.5 inches) and approximately 11 feet in diameter (132.5 inches). When loaded with a spent fuel canister, the casks will weigh approximately 180 tons. The steel and concrete cy-

lindrical walls of the cask form a heavy steel weldment, consisting of an inner and outer steel shell within which shielding concrete is installed. These walls are approximately 30 inches thick. The MPC in which the spent fuel is sealed is stored vertically within the storage cask. Singh/Soler Dir. at A17.

128. The storage cask has four air inlets at the bottom and four air outlets at the top to allow air to circulate naturally through the annular cavity to cool the MPC. The inner shell of the storage cask has channels attached to its interior surface to guide the MPC during insertion and removal. These channels would also provide a flexible medium to absorb impact loads under postulated, non-mechanistic tip-over events, while allowing cooling air to freely circulate through the cask. Singh/Soler Dir. at A17.

129. The HI-STORM System storage cask is designed as a ASME Section III, Class 3, Subsection NF cylindrical structure. The outer steel shell (which is ¾-inch thick) and the inner steel shell (which is 1¼ -inch thick) are both welded to a 2 inch thick steel baseplate, and are joined by four full-length inter-shell radial steel support plates, each ¾-inch thick and welded to the inner and outer shells. The concrete shielding is placed within this steel weldment. The cask provides an internal cylindrical cavity, 191½ inches in height and 73½ inches in diameter, for housing the MPC. The top steel closure plate is also a steel weldment with confined concrete. Finally, a steel pedestal with enclosed concrete is provided for shielding, missile penetration, canister drop, and cooling flow considerations. As stated earlier, steel channels are located on the interior surface of the inner shell which act to minimize the loadings that would be imparted to the MPC in a postulated, hypothetical cask tip-over scenario. Singh/Soler Dir. at A21. The circular gap between the channels and the MPC varies from 0.75 to 4.75 inches diametrically.

Thus, the effective radial gap between the MPC and the channels which retains the MPC in place over most of its axial extent is ⅜ of an inch. Tr. 5863-64, 6104-05 (Singh)..

130. The multi-purpose canister is the component in which the spent fuel is placed. The spent fuel is loaded into the MPC at a nuclear power plant site, after which, the MPC is filled with an inert gas (helium) and welded shut for storage at the plant site or ready for transport off-site. The MPC consists of (i) the stainless steel enclosure vessel and (ii) the fuel basket. The enclosure vessel is a cylindrical container with flat ends designed to meet the applicable provisions of Subsection NB of the ASME Code. The fuel basket is a stainless steel, continuously welded, stiff honeycomb structure that is designed to meet Subsection NG of the ASME Code, as applicable, and serves to position the fuel in the MPC enclosure vessel. Singh/Soler Dir. at A22.

131. As described in Section 4.2.1.5.2 of the PFSF SAR, the HI-STORM System storage casks will be placed on a regular array of concrete pads arranged to provide a lateral (edge to edge) spacing of 35 feet between adjacent pads in the East-West direction and 5 feet longitudinal spacing in the North-South direction. Each pad will be sized to accommodate a 2 x 4 array of casks with a 15 ft pitch (the distance between the casks center points) in the width (or East-West) direction and 16 ft in the length (North-South) direction. As described in Section 4.2.3.1 of the PFSF SAR, the cask storage pads will be independent structural units constructed of reinforced concrete, each pad being 30 ft wide, 67 ft long and 3 ft thick. Each pad will be capable of supporting eight loaded storage casks. At maximum capacity the facility would contain 500 such pads, each supporting eight loaded storage casks. Singh/Soler Dir. at A18; Youngs/Tseng Dir. at A26-27. A graphical rep-

resentation of the cask storage arrangement is shown on State Exh. 175 (Figure 4.2-7 of the PFSF SAR) and PFS Exh. 84 and Staff Exh. X (Figure 1.2-1 of the PFSF SAR).

b. Holtec's Cask Analyses

i) Overview of Holtec's Cask Analyses

132. Holtec performed seismic analyses for the HI-STORM System to be used at the PFSF using the general design parameters for the HI-STORM System together with the site-specific earthquake ground motions for the PFSF site and other relevant site-specific parameters. The analyses showed that under design basis earthquake conditions for the PFSF, the loaded HI-STORM System casks have large safety margins against overturning or sliding. In no case did the analyses predict that there would be any cask tip-over or any cask-to-cask impacts. Singh/Soler Dir. at A35.

133. Under the design basis earthquake, the Holtec model showed a maximum displacement of the cask on the order of 3 to 4 inches. The maximum angle of tilt indicated by the analysis for the 2,000-year design basis earthquake for an upper bound coefficient of friction of 0.8 is 1.026 degrees. Singh/Soler Dir. at A36. This can be compared to the angle of tilt of 29.3 degrees at which a cask would tip if slightly disturbed, due to the moment of its own weight (i.e., the orientation at which the center of gravity of the cask is directly over the edge of the cask). This provides a safety factor against cask tipover for the PFSF design basis earthquake of 29.3/1.026, or 28.6. Singh/Soler Dir. at A36.

134. Holtec also performed an analysis of the performance of a loaded HI-STORM storage cask subject to accelerations from a postulated, beyond-design basis,

10,000-year return period earthquake for the PFSF site. The earthquake had a vertical peak ground acceleration ("PGA") of 1.33g and horizontal PGAs of 1.25g and 1.23g. Singh/Soler Dir. at A39. The loaded cask exhibited larger rotations relative to the pad (approximately 10.89 degrees from the vertical) than in the earlier analyses using the design basis earthquake levels, but the results of this analysis still showed the existence of significant margins against tip-over. Singh/Soler Dir. at A39. Using the same definition of safety factor against cask overturning as before, the safety factor against overturning for the 10,000-year return period earthquake was 2.69 (29.3/10.89). Singh/Soler Dir. at A39.

135. Holtec performed a series of additional beyond design basis analyses under a variety of assumptions. Those using the 10,000-year return period earthquake showed maximum rotations on the order of 10 to 12 degrees, confirming the large margins of safety against cask tipover stated above. Tr. 5774-76, 5787-88 (Soler).

136. Holtec also evaluated the results of a hypothetical cask tip-over event with the attendant impact of the cask on the pad. This tip-over analysis showed that the maximum fuel deceleration is below 45g, which is a licensing limit set by the NRC Staff. Singh/Soler Dir. at A46. As discussed below, staying within the 45 g limit ensures that, in reality, a very large safety margin exists against canister breach and potential releases of radioactivity. Therefore, even assuming that a cask were to tip over, the cask tipover analyses conducted by Holtec show that no breach of the cask or release of radioactivity from the cask would occur. See discussion of State Section E claims, infra.

ii) Holtec's Design Basis Cask Stability Analyses

137. To perform its design basis analyses, Holtec used its specially developed computer code known as DYNAMO. Singh/Soler Dir. at A26. This code has been

validated and has been reviewed and accepted by the NRC for the licensing of freestanding spent fuel storage systems. Singh/Soler Dir. at A28, A30. It has been used by Holtec to perform the seismic analyses in its Safety Analysis Report for the HI-STORM System which supports the Certificate of Compliance ("CoC") that the NRC has issued for the HI-STORM 100 Cask Storage System under 10 C.F.R. Part 72. Singh/Soler Dir. at A27. Holtec has also performed site-specific seismic analyses using DYNAMO for the HI-STORM System for spent fuel systems for Pacific Gas & Electric (Diablo Canyon), Exelon (Dresden), Energy Northwest (Columbia Generating Station), Entergy Nuclear Northeast (J.A. Fitzpatrick) and Tennessee Valley Authority (Sequoyah). Singh/Soler Dir. at A27. All of these analyses have been for storage casks on concrete storage pads.

138. In addition, Holtec has extensive experience in using DYNAMO for the seismic analysis of spent fuel racks used to store spent fuel inside nuclear power plants. Singh/Soler at A28. The spent fuel racks are large free-standing rectangular structures of honeycomb construction that sit in the spent fuel pool. These racks are square or rectangular, are supported by four or more stubby legs, and rest on the spent fuel pool floor slab. During a seismic event, the racks may slide, tip, and rotate with respect to the spent fuel pool in a manner similar to the potential motions of a storage cask on a concrete storage pad. The same non-linear phenomena (sliding and tip-over) are modeled with the additional feature that fluid coupling between racks, and between racks and walls, is also considered. Holtec has employed its wet storage seismic simulation methodology using DYNAMO at numerous nuclear sites (more than 40), both in the U.S. and abroad, and in all instances the use of DYNAMO has been accepted by the regulatory authority. Singh/Soler at A28.

139. In order for DYNAMO to be approved by the NRC for use in licensing analyses, the code had to be validated to demonstrate that it produces acceptable results for the class of problems for which it is used in accordance with ASME NQA-2a-1990, Part 2.7 ("Quality Assurance Requirements of Computer Software for Nuclear Facility Applications"). A series of classical problems having known solutions were modeled using the code and were shown to give results in good agreement with the analytical results. The problems were chosen to demonstrate all of the features that are built into DYNAMO. In addition, problems that had no simple analytical solutions were also evaluated and shown to give good agreement with numerical solutions using other industry codes such as ANSYS. Finally, some features of DYNAMO were validated by comparing results from experiments designed to be capable of simulation using DYNAMO. During the course of license submittals, DYNAMO was subjected to additional validation at the request of NRC's reviewers. In every case, the DYNAMO code proved capable of providing acceptable solutions to the problem. Thus, DYNAMO has been extensively benchmarked to confirm its adequacy as a non-linear dynamics code. Singh/Soler Dir. at A30, A133-A134.

140. In performing the seismic cask stability analysis for the PFSF, Holtec modeled the casks as free-standing structures on the concrete storage pads with compression-only contact and friction elements modeling the interfaces between casks and the pad. The casks, along with their loaded internals, were modeled as rigid bodies. Singh/Soler Dir. at A31. The concrete storage pad was modeled as a rigid rectangular slab, and the effect of the soil/soil cement foundation was modeled by springs and dampers to characterize the soil resistance in deflection and rotation. Singh/Soler Dir. at A31. Data characterizing the earthquake excitation (accelera-

tion time histories) and the soil response (soil properties used to characterize the soil springs and dampers) were provided to Holtec as design inputs by Geomatrix Consultants, Inc. ("Geomatrix"). Singh/Soler Dir. at A31.

141. Specifically, Geomatrix provided Holtec with sets of "Best Estimate," "Lower Range," and "Upper Range" soil properties for the soil under the pad, including the effect of soil cement, as applicable. Holtec then computed the values of the spring constants and damping coefficients for use in its analyses using the soil property values supplied by Geomatrix. This was done in accordance with the formulas provided in ASCE Standard 4-86, "Seismic Analysis of Safety Related Nuclear Structures and Commentary," Tables 3300-1 and 2, and Figure 3300-3. Singh/Soler Dir. at A32. These formulas are derived from a well-recognized technical treatise, Newmark, N. M., and Rosenblueth, E., <u>Fundamentals of Earthquake Engineering</u>, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1971. Tseng Reb. at A8.

142. Geomatrix also supplied Holtec with the ground motions for the 2,000-year return period design basis seismic event in the form of three acceleration time histories entitled "Fault Normal", Fault Parallel", and "Vertical". These seismic ground motions were developed to match 5%-damped response spectra having the following zero period acceleration ("ZPA"), also known as the Peak Ground Acceleration ("PGA") values:

Fault Normal – 0.711 g
Fault Parallel – 0.711 g
Vertical – 0.695 g.

The actual time histories used in the dynamic analyses, developed in accordance with Section 3.7.1 of the NRC Staff's Standard Review Plan (NUREG-0800), had the following peak acceleration amplitudes:

Fault Normal – 0.73 g
Fault Parallel – 0.71 g
Vertical – 0.73g.

In the design basis analysis, Holtec applied these acceleration time histories at the base of the soil springs with the spring constants and damping values computed as described above. Singh/Soler Dir. at A32.

143. For the design basis analysis, Holtec modeled various configurations of one (1) to eight (8) casks on the concrete pad using the lower bound, best estimate and upper range soil properties. To model the effect of friction between the cask and pad, Holtec used an upper bound coefficient of friction of 0.8 at the cask/pad interface (to emphasize or increase the likelihood for cask tipping) and a lower bound coefficient of friction of 0.2 (to emphasize or increase the likelihood of cask sliding). Singh/Soler Dir. at A34. To model the compression contact at the cask/pad interface, Holtec used a vertical contact stiffness of 454,000,000 lb./inch (Singh/Soler Dir. at A138), and to model the loss of energy that would occur should the cask lift up and impact down on the pad Holtec used an impact damping value of 5%. Singh/Soler Dir. at A65. The vertical contact stiffness and impact damping were modeled using springs and dampers at the cask/pad interface with the appropriate values.

144. Nine cases were run for the upper bound coefficient of friction of 0.8, and one case was run for a lower-bound coefficient of friction of 0.2 for the configuration that gave the limiting results using upper bound coefficient of friction of 0.8. The reason only one case was run at the 0.2 coefficient of friction was that previous

cask stability analyses that Holtec had performed for the PFSF for different earthquakes showed that the bounding solution for cask displacement (as measured at the top of the casks) was for a coefficient of friction of 0.8. Singh/Soler Dir. at A34.

145. As stated above, for the 2,000 year design basis earthquake, the Holtec analysis using the upper bound coefficient of friction of 0.80 showed a maximum displacement of the cask on the order of 3 to 4 inches with a corresponding maximum angle of tilt of 1.026 degrees, which provides a factor of safety in the angle of tilt of 28.6 when compared to the angle of tilt at which a cask would tipover from the moment of its own weight. The case evaluated for a coefficient of friction of 0.2 produced a maximum sliding displacement on the order of 2 inches. Singh/Soler Dir. at A36.

iii) Holtec's Beyond Design Basis Cask Stability Analyses

146. In addition to its design basis cask stability analyses using DYNAMO, Holtec undertook various beyond design basis cask stability analyses. For these analyses, Holtec used the VisualNastran ("VN") computer code because the beyond-design basis analyses that Holtec conducted were mostly for the 10,000-year earthquake level. DYNAMO is a small deflection program, which means that it cannot accurately model large cask rotations or displacement, whereas VN is capable of modeling large rotations of the cask that could occur under the 10,000-year earthquake event. Singh/Soler Dir. at A113.

147. Holtec ran various cask configurations under different assumptions to evaluate the response of the casks to a 10,000-year return period earthquake with a vertical PGA of 1.33g and horizontal PGAs of 1.25g and 1.23g and to respond to specific

issues raised by the State and its witnesses. The results of these analysis are set forth in "PFSF Beyond Design Basis Scoping Analysis" (Holtec Report No. 2022854), PFS Exh. 86C, and supporting exhibit, PFS Exh. 86D, and in "Additional Cask Analysis for the PFSF" (Holtec Report No. 2022878), PFS Exh. 225, and supporting exhibits PFS Exh. 225A, and 225D. In addition, using VN, Holtec produced visual simulations from the analyses which are contained in the "movies" collected in PFS Exhibits OO and 225B.

148. The results of the analyses show that even under the beyond design basis 10,000 year earthquake and using worst case assumptions the casks did not tip over. The simulations of the 10,000 year beyond design basis earthquake showed some instances of large cask rotations, on the order of 10-12 degrees. Tr. 5774-76, 5787-88 (Soler). Even with such large rotations, the casks still have a safety of factor in excess of 2 when compared to the angle of tilt (29.3 degrees) at which a cask would tip over from the moment of its own weight. Singh/Soler Dir. at A36. The assumptions and results of these additional cask stability analyses performed by Holtec will be elaborated on below in the context of the various issues raised by the State.

c. *State Challenges to the Holtec Cask Stability Methodology*

149. The State raises several challenges to the general methodology underlying Holtec's cask stability analyses. These are:

- use of the DYNAMO program, because it cannot predict a cask tipover;
- failure to account for model sensitivity to contact stiffness parameter; and

- failure to use an appropriate value for damping.

i) Use of Small Deflection Program

150. Dr. Khan, on behalf of the State, questions the validity of Holtec's use of the DYNAMO code for the PFSF 2,000-year design basis earthquake stability analysis because DYNAMO is a small deflection program and, consequently, is not capable of handling large cask rotations. Dr. Khan claims that Holtec "has not validated its DYNAMO results for the 2,000-year DBE at the PFS site with another structural analysis code such as VisualNastran or ANSYS" and cannot therefore determine whether DYNAMO has provided erroneous results. Khan/Ostadan Dir. at A11 & A26.

151. The record does not support Dr. Khan's criticisms. First, the results of the Holtec design-basis earthquake analyses show that in the event of a design basis earthquake the casks will undergo small, not large, rotations. As stated above, the maximum rotation of the casks from the nine configurations evaluated by Holtec for the design basis earthquake was 1.026 degrees. Dr. Soler testified that he would consider large rotations somewhere on the order of twenty degrees or less. See Tr. 6101-02 (Soler). Therefore, the rotations obtained through the use of DYNAMO are well within the code's capabilities. Dr. Soler himself has extensive experience in the running of the DYNAMO code and is well aware of its small deflection limitations, and yet he was comfortable with using DYNAMO for the design basis earthquake; on the other hand, he decided to use Visual-Nastran for evaluating cask stability for the 10,000 year beyond design basis earthquake because of the potential for large cask rotations in that case. Singh/Soler Dir. at A113.

152. Second, contrary to the Dr. Khan's claim, Holtec has validated its DYNAMO results for the 2,000-year DBE at the PFS site against another structural analysis code, that is, VisualNastran. As part of its beyond design basis scoping analysis Holtec ran one of the nine configurations of the original design basis analysis using VisualNastran. The VisualNastran run of the design basis earthquake predicted cask displacements on the order of several inches, similar to the DYNAMO results, thus showing that the DYNAMO runs were within the capabilities of that code. Singh/Soler Dir. at A118.

153. Third, as discussed above, DYNAMO has been extensively benchmarked, validated and accepted by the NRC, and it has been shown to the NRC's satisfaction to provide valid predictions. Singh/Soler Dir. at A30, A133-A134.

ii) Choice of Contact Stiffness

(a) Summary of Competing Claims

154. The focus of the State's challenge to the validity of Holtec's dynamic cask stability analysis is Holtec's choice of contact stiffness, particularly the choice for vertical contact stiffness. Khan/Ostadan Dir. at A28. Vertical contact stiffness represents the amount of force applied at the interface points of contact between two bodies that would be required to have one of the bodies approach or penetrate the other a unit distance. Singh/Soler Dir. at A136. The parameter is measured in the pounds of force required to cause one body to approach the second body by one inch. For example, assume that you have a pad of undefined material on which a HI-STORM System cask weighing 360,000 lbs is placed which causes a deformation or deflection of the pad of 0.01 inches, the contact stiffness would be 360,000 lbs./0.01 inches or 36 x 10^6 lbs. per inch. Singh/Soler Dir. at A136; see also Tr. 6045-46 (Soler).

155. For its cask stability analysis for the 2,000 year DBE, Holtec used a vertical contact stiffness of 454×10^6 lbs. per inch. Singh/Soler Dir. at A138, A156; see also PFS Exh. 226. Dr. Khan claims that this choice of contact stiffness is too high and results in making the vertical frequency of the cask toolarge, thus artificially reducing the vertical displacement because the code will treat the cask as if anchored to the pad. Khan/Ostadan Dir. at A28, A31. According to Dr. Khan, the contact stiffness should be chosen to correspond to cask frequencies that fall in the amplified spectral range of the earthquake input spectra. Khan/Ostadan Dir. at A31-A32. Therefore, he concludes that "a more appropriate contact stiffness value for unanchored casks could be in the range of 1×10^6 lbs/inch to 10×10^6 pounds per inch" (instead of the 454×10^6 lbs. per inch used by Holtec in the 2,000-year DBE analysis) because "[t]his range of stiffness values would correspond to cask frequencies that fall in the amplified spectral range of the input spectra." Khan/Ostadan Dir. at A32.

156. In effect, Dr. Khan would choose the contact stiffness so that the natural vertical frequency of the cask on the pad was in resonance with the amplified spectral range of the earthquake. Several consequences flow from following the approach suggested by Dr. Khan. First, such an approach artificially maximizes the vertical response of the cask by assuming that the natural frequency of the cask and the earthquake are in resonance. Second, because the amplified spectral range of an earthquake will vary depending on the geology and soils of its location, setting the contact stiffness to artificially cause the cask and the earthquake to be in resonance means that the choice of contact stiffness will vary depending on the geographic location of an ISFSI and the assumed earthquake excitation. See Tr. 7215-16 (Khan); see also Tr. 9617-18 (Soler).

157. Dr. Khan derives his position from a belief that it is virtually impossible to choose a contact stiffness based on physical principles because the contact stiffness changes throughout a dynamic earthquake event. Khan/Ostadan Dir. at A31. However, according to Drs. Singh and Soler, contact stiffness is a physical property of the cask-pad interface that can be determined from the physical characteristics of the cask and the pad, and as such it would not change depending on geographic location or earthquake excitation. Tr. 9617-19 (Soler).

158. Drs. Singh and Soler testified that often, in computer modeling, one will chose a value of contact stiffness that is lower than the actual physical contact stiffness to avoid excessive computing time, but one should always avoid using such a low value that the corresponding cask frequencies fall into the amplified spectral range of the earthquake spectra. See Tr. 9641-45 (Soler). If that was done (as proposed by Dr. Khan), the results of the analysis would be contaminated by introducing an artificial excitation of the cask which does not exist in fact, since the actual physical contact stiffness of the cask-pad interface does not produce cask frequencies in the amplified spectral range of the earthquake. Therefore, choosing a contact stiffness that would cause resonance of the cask with the earthquake should be avoided because it would produce unrealistic results that would not be expected to occur under earthquake conditions. Tr 9633-45 (Soler). See Tr. 9635-38 (Singh);Tr. 9633-34 (Soler); see also Tr. 9617 (Singh).

159. Drs. Singh and Soler also state that a correct computer model should be able to predict accurately both dynamic and static conditions (Singh/Soler Dir. at A155) and that choosing a contact stiffness of 1×10^6 lbs/inch, as suggested by Dr. Khan, would result in a deformation of 3/8 of an inch of the reinforced concrete

pad under static conditions, which is an obviously incorrect result that defies reality. Singh/Soler Dir. at A143.

160. Dr. Khan does not dispute that a deflection of 3/8 of an inch of the concrete pad from having a HI-STORM 100 cask just sit on the pad surface is contrary to physical fact, but takes exception to the concept that a model should be able to accurately predict both static and dynamic conditions. Tr. 7218-19, 7213-15 (Khan).

(b) Resolution of Claims

161. We are presented here with a situation in which experts from opposing parties provide conflicting technical testimony. As will be seen below, based on the extensive experience of Drs. Singh and Soler; Dr. Khan's lack of experience in this area; the support for Drs. Singh and Soler's position from authoritative guidance such as ANSYS Training Manual; the validation of Holtec's DYNAMO model against known classical solutions; the agreement of Dr. Luk; the additional analyses performed by Drs. Singh and Soler; and the logic of their position, we agree with the interpretation of Drs. Singh and Dr. Soler on the proper application of vertical contact stiffness and decline to follow Dr. Khan's suggested approach.

i) Applicable Professional Experience

162. Dr. Singh and Dr. Soler have extensive professional experience in conducting cask stability analyses. Dr. Singh has a Ph.D. in Mechanical Engineering, which he received from the University of Pennsylvania in 1972. He has extensive experience in the design and licensing of nuclear spent fuel systems which extends back to 1979. Over the past twenty-three years, Dr. Singh has personally led the design and licensing of spent fuel storage systems for over forty nuclear plants,

and for Holtec's HI-STAR 100 System and HI-STORM 100 Storage Cask System ("HI-STORM System"). Dr. Alan I. Soler is responsible for all corporate engineering activities at Holtec International, including overseeing the analyses performed to establish the stability of the HI-STORM System under postulated seismic events. Dr. Soler has either performed or reviewed all HI-STORM System seismic analyses conducted in support of deployment of the HI-STORM System at the PFSF. Likewise he has either performed or reviewed dozens of seismic analyses for freestanding storage casks and storage racks over many years. Together, they have approximately 40 years of experience in dynamic analyses of spent fuel racks, storage and transportation casks. Singh/Soler A3, A10-A13, A27.

163. In contrast, as discussed below, Dr. Khan has virtually no experience in analyzing the dynamic stability of large free standing objects, such as the casks. He acknowledged that in his professional career he had never previously selected or calculated a "contact stiffness" value for purposes of analyzing the sliding or tipping of a free-standing object. He also failed to cite or refer to any authoritative source to support his position. In other words, he provided the Board solely with his professional views in an area in which he himself acknowledged his lack of direct experience. These circumstances lead us to give little weight to his opinions.

ii) Contact Stiffness is an Intrinsic Property of the Materials of the Cask and the Pad

164. Contact stiffness is a physical parameter of the contacting objects and their intrinsic material properties. Tr. 9618-22 (Singh/Soler); see also Tr. 7242-43 (Khan). The contact stiffness at the interface of two objects can therefore be derived from nature's physical laws, as shown by Heinrich Hertz in 1881. Tr. 9118 (Singh).

Holtec computed the vertical contact stiffness of 454 x 10^6 lbs. per inch for its DYNAMO design basis analysis using a well established methodology developed by Timenshenko and Goodier for calculating the contact stiffness between two objects. Tr. 9622-23 (Singh); PFS Exh. 226.[20] The approach used by Holtec is in accordance with guidance from the ANSYS Training Manual, which states that the "Hertz contact stiffness often provides an appropriate basis" for determining the contact stiffness to be used for modeling bulky objects. Tr. 9625-26 (Singh); PFS Exh. 221 at 3-6. The Hertzian theory of contact is the standard state of the art used to simulate the interface between two bodies. Tr. 9228-29 (Singh/Soler).

165. Similarly, in Sandia's modeling of cask stability for the NRC, contact stiffness was not treated as a "physical behavior." It was determined in accordance with the intrinsic properties of the contacting materials and the applicable physical relationships for determining contact stiffness based on "well established theory" for this behavior. Tr. 6809-11 (Luk).

166. Because contact stiffness is an intrinsic property of the contacting bodies, it does not vary from one geographic location to another as the earthquake characteristics change, as it would under the approach espoused by Dr. Khan. Tr. 9617-19 (Soler). (Singh/Soler).

iii) Authoritative Guidance on Contact Stiffness

167. Drs. Singh and Soler refer to guidance provided in ANSYS manuals on choosing appropriate contact stiffness for computer modeling to support their position concerning the proper choice of contact stiffness here. Singh/Soler Dir. at A139-

[20] -The contract stiffness calculated in PFS Exh. 226 (one of the earlier cask stability analyses performed by Holtec for PFS) was utilized in the cask stability analysis for the 2,000 year design basis earthquake. See State Exh. 173 at 7, 12.

A144. ANSYS is a recognized, general purpose computer modeling program accepted by Dr. Khan as an authoritative source. Khan/Ostadan Dir. at A23. The ANSYS Training Manual refers to the Hertz contact stiffness theory applied by Holtec as "often provid[ing] an appropriate basis" for choosing a contact stiffness. PFS Exh. 221.[21] The Training Manual also contains more than 100 pages devoted almost entirely to friction and contact problems. In addition, the ANSYS Verification Manual contains sample problems covering friction and contact issues and related guidance. See, PFS Exhibit SS; Singh/Soler Dir. A140; Tr. 7208-10 (Khan).[22]

168. The guidance provided by the ANSYS Training Manual included in PFS Exhibit SS makes it clear that, in order to achieve realistic modeling, the choice of stiffness for the contact springs between two contacting surfaces should not produce analysis results predicting a measurable penetration or deflection of one of the bodies in contact, because such penetration or deflection is contrary to physical fact. In this respect, the guidance states that a contact stiffness that results in minimum penetration or deflection provides the "best accuracy," and "[t]herefore, the contact stiffness should be very great." PFS Exh SS at 3-3. The guidance goes on to note, however, that in order to avoid convergence difficulties that may arise from "too stiff a value," determining "a good stiffness value usually requires some experimentation" but that "if you can visually detect penetration in a true-scale displaced plot of the entire model, the penetration is probably excessive." In

[21] Dr. Khan was not "familiar with the Hertzian Contact Theory" and had never calculated the "contact stiffness between two objects." Tr. 9382-83 (Khan).

[22] The first five pages of PFS Exhibit SS are from the ANSYS Training Manual as the label in the upper right hand corner indicates. The last three pages are from the ANYSYS Validation Manual as reflected by the initials "VM" in the header on each page.

that case, one should "[i]ncrease the stiffness and restart." Id. at 3-14. The testimony of Drs. Singh and Soler concerning contact stiffness is wholly consistent with this guidance. Tr. 9641-45 (Soler).

169. Dr. Khan acknowledged that he had not seen these pages from the Training Manual before and that he had never taken an ANSYS training course that covered the use of contact stiffness. Tr. 7208-09, 9380 (Khan). He nevertheless claimed that this guidance was not relevant here for various reasons. He first suggested that the guidance was solely for penetration problems. Tr. 7210, 9373-74 (Khan). However, the Training Manual expressly describes its guidance as providing "Basic Concepts" for modeling the "contact interface" for "physical contacting bodies," that "do not interpenetrate," but that "[s]ome amount of penetration," or deflection of the contact spring at the contact interface, "is required mathematically" to model the interface. PFS Exh. SS at 3-2. Thus, the guidance on its face is for "physical contacting bodies" that "do not interpenetrate."

170. Dr. Khan also claimed that the guidance in the ANSYS Training Manual solely involved static cases not applicable to the dynamic analysis being undertaken here. Tr. 9371 (Khan). However, nothing in the Training Manual – which as noted above is generally described as involving "Basic Concepts" – suggests that the guidance therein is inapplicable to dynamic analysis, as claimed by Dr. Khan. To the contrary, as explained by Drs. Singh and Soler, using static Hertzian contact mechanics for developing contact stiffness input parameters for dynamic analysis of impacting bodies "is the standard state of the art." Tr. 9628-29, 9639-40 (Singh/Soler).

171. Thus, the selection of contact stiffness is a well-defined and understood problem when dealing with known properties of materials. Guidance exists for selecting

contact stiffness values in general purpose, validated, and well-established computer modeling programs, such as ANSYS, using tested mathematical solutions. Rather than being an unknown quantity to which dynamic analyses are extremely sensitive, the appropriate setting of a contact stiffness value is relatively straightforward for an experienced modeler.

iv) Dr. Khan's Choice of Contact Stiffness Produces Results Contrary to Physical Reality

172. A decisive factor in assessing Dr. Khan's suggested approach is that his choice of contact stiffness produces results that are contrary to physical reality. Using a contact stiffness of 1×10^6 lbs. per inch, as suggested by Dr. Khan, results in a deflection of 3/8 of an inch in the pad simply from having the cask rest on its surface. Singh/Soler Dir. at A143. This is totally unrealistic, since the pressure applied by a fully loaded cask on the reinforced concrete pad is 26 lbs. per square inch, equivalent to a man standing on one foot. Singh/Soler Dir. at A88. Dr. Khan does not dispute that 3/8 of an inch deflection is totally unrealistic. Tr. 7218-19 (Khan). Drs. Soler and Singh maintain that a model should be able to provide a physically correct answer for all conditions, including the initial static case, as well as under dynamic loading. Singh/Soler Dir. at A155. Dr. Luk agrees. Tr. 6816-17 (Luk). We also agree.

173. Further, Dr. Soler testified that there are simple mathematical relationships between the natural frequency of the cask under dynamic conditions, the static deflection of the pad caused by the cask resting on its surface, and its contact stiffness. PFS Exh 225; Tr. 9632-34 (Soler). Those relationships involve the same formula that Dr. Khan cites would use as the basis for choosing a contact stiffness. According to those relationships, the frequency of the cask vibrating or os-

cillating on the pad is a function of the static deflection of the pad caused by the cask resting on its surface, or in other words, the static contact stiffness. See Khan/Ostadan Dir. at A31; Tr. 9382-89 (Khan); PFS Exh 225 at 21; Tr 9632-33 (Soler); Singh/Soler Dir. A136 & A143.

174. The existence of this relationship is highly significant in several respects: (1) it refutes Dr. Khan's assertion that neither static deflection nor contact stiffness derived under static conditions have relevance to the dynamic analyses; (2) it demonstrates that, since the natural frequency of the cask can be directly related to the static deflection, it is controlled by the physical characteristics of the cask-pad interface and not the incoming earthquake excitation; thus it is incorrect to adjust the contact stiffness to tune the natural frequency of the cask to the external earthquake excitation; (3) it allows one to ascertain whether the amplified range of the response spectral curve – which Dr. Khan claims should guide the choice of contact stiffness – corresponds to a physically realistic static deflection, i.e., whether such a static deflection would be seen in the real world; and (4) conversely, using the simple mathematical relationship between cask natural frequency and contact stiffness, it allows one to quickly ascertain what the natural frequency of the cask is. Using the contact stiffness of 454 x 10^{-6} in the above formulas results in a natural frequency of the cask vibrating or oscillating on the pad of 111 hertz. Tr. 9634-35 (Soler). This frequency is far above the spectral range of the earthquake input spectra, and far above the frequencies of interest for seismic earthquake analysis, which are well below 50 hertz. Tr. 6045-46, 9636-37 (Singh); see also Tr. 6794-95 (Luk).

175. The ability to derive the natural frequency of the cask in terms of its static deflection refutes the underlying basis of Dr. Khan's testimony that it is virtually impos-

sible to choose a contact stiffness based on physical principles because the contact stiffness changes throughout a dynamic earthquake event. Therefore, Dr. Khan claims that, absent shake table testing, one must conservatively choose a contact stiffness that tunes the natural frequency of the cask to the amplified range of the earthquake response spectral so as to maximize vertical excitation of the cask. Khan/Ostadan Dir. at A31; Tr. 7215-16 (Khan). Indeed, Drs. Singh and Soler have shown that the natural frequency of a 360,000 pound HI-STORM 100 System cask sitting on a concrete pad at the PFSF site would be 111 Hz, far greater than the amplified spectral range of the input earthquake for the site. Tr. 9635 (Soler). Therefore, to follow Dr. Khan's approach would be to artificially create a resonance between the natural frequency of the cask on the pad and the natural frequency of the earthquake and obtain results that bear no semblance to how the cask will perform under earthquake conditions.

176. The above discussion has focused solely on vertical contact stiffness. Additionally, for computer modeling one needs to use a horizontal stiffness for the friction spring at the cask pad interface that must be overcome for sliding to occur. Horizontal stiffness is a mathematical artifice, necessary only because of the use of computer numerical analysis for arriving at the solution as opposed to analytical classical solutions. Tr. 7214-15 (Khan); Tr. 9652 (Soler).

177. For his analysis, Dr. Khan choose a contact stiffness of 100,000 lbs/inch. Khan/Ostadan Dir. at A32. This value appears to be unreasonably low and therefore results in unrealistic predictions for cask sliding. The ANSYS Validation Manual states that the horizontal stiffness for the friction spring "should be selected high enough to minimize elastic contact" or movement. PFS Exh. SS at VM 73.1; Singh Soler Dir. A141; Tr. 6040-41 (Soler).

178. As demonstrated by Drs. Singh and Soler, Dr. Khan's assumption of a contact stiffness of 100,000 lbs/inch for the 360,000 lb cask on the pad results in his model predicting a 0.72 inch horizontal displacement of the cask just prior to the actual initiation of cask sliding. Singh/Soler Dir. at A130 & A147; PFS Exh 92. The prediction of such displacement prior to the initiation of sliding is again not consistent with reality.

179. Dr. Khan's assertion that contact stiffness should vary according to geographic location and as an earthquake's characteristics change is without any support and belies the guidance provided by modeling programs, such as ANSYS, that make no mention of this supposedly essential fact. Dr. Khan's concerns about the use of an appropriate contact stiffness are not credible when looked at in terms of the results of Dr. Khan's choice of a vertical contact stiffness of 1×10^6 and a horizontal contact stiffness of 100,000 lbs./inch. Not only would the use of these values result in a storage cask literally sinking nearly half an inch into a reinforced concrete storage pad, they would result in a model that predicts displacement of three-quarters of an inch, before actual cask sliding occurred. Neither of these predictions about how the cask and storage pad would behave comports with physical reality.

v) Validation and Licensing Acceptance of Model

180. As discussed above, the Holtec DYNAMO model has undergone extensive NRC scrutiny and successful validations that show that the model can reproduce known solutions and thus be confidently used to make predictions from known or assumed input parameters. Moreover, it has also been used to support numerous license applications and its results have been approved by the NRC in numerous

dockets. Singh/Soler Dir. at A28. By contrast, Dr. Khan's model has not been accepted for use, cannot reproduce classical solutions, has not been benchmarked, or otherwise demonstrated to produce valid results. See Tr. 7219-20 (Khan). The only step Dr. Khan took to attempt to validate his model was to compare the solution of his initial simple mass model using SAP2000 with runs using ANSYS. This exercise, however, only demonstrated that the model algorithm had been properly programmed using both computer codes, such that when both programs were given the same model input they provided the same output. As Dr. Khan readily acknowledged, this did not establish that the model was appropriate; the same wrong input parameters to two valid computer codes will lead to equally erroneous result for both. Tr.7517-18 (Khan); PFS Exh PP at 77.

vi) Additional Holtec Computer Simulations Using Different Vertical Contact Stiffnesses

181. In response to Dr. Khan's claims, Holtec performed additional VisualNastran computer simulations using lower contact stiffnesses than in its analyses using DYNAMO. Holtec ran VisualNastran using a vertical contact stiffness in the middle of the range of values that Dr. Khan claimed should have been used. Even though this brought the model within the spectral range of the earthquake input spectra, and thereby contaminated the results, the program still showed displacements on the order of inches and not feet as claimed by Dr. Khan. PFS Exh. 225 at 29-30; Tr. 9671-76 (Soler). Holtec also ran Dr. Khan's model using the unreasonably low values for vertical contact stiffness (1,000,000 lbs/inch) and impact damping (1.0%) that Dr. Khan had used in his analysis. Even at these unreasonably low values the casks did not tip over or impact each other. PFS Exh. 225 at 24-26; Tr. 9606-07, 9611-14 (Soler).

vii) Conclusion on Validity of Holtec Methodology for Choice of Contact Stiffnesses

182. While the State has asserted a variety of inadequacies in the Holtec methodology for choice of contact stiffness, it has completely failed to produce any evidence that would call the Holtec methodology into question. To the contrary, the State's concerns have been demonstrated to not be credible, giving rise to absurd results. By contrast, PFS has demonstrated that the Holtec methodology is well-accepted, based on the well-understood and state-of-the-art Hertzian theory of contact between two bodies, and follows guidance set forth in computer modeling programs, such as ANSYS, designed to deal with problems involving contact stiffness. Moreover, solely between Dr. Singh and Dr. Soler, Holtec has a cumulative 40 years of experience in dealing with dynamic analyses. By contrast, Dr. Khan has essentially no experience in the modeling of freestanding structures before this proceeding. As Dr. Cornell testified, non-linear analyses "depend to a greater extent on the expertise of the user than does a linear analysis." Tr. 8010-11 (Cornell). Dr. Singh and Dr. Soler have conducted, supervised or reviewed numerous dynamic analyses involving freestanding structures that have been subjected to regulatory as well as peer review. In all this experience including many submissions to the NRC, their modeling of contact stiffness has followed the well-accepted methodology that they used for the dynamic analyses for the PFSF. The Holtec methodology has thus been shown to not only be acceptable, but to produce verifiable and realistic results.

iii) Holtec's Choice of Impact Damping

183. As stated above, in Holtec's cask stability analysis for the 2,000 year earthquake using DYNAMO, Holtec used a 5% value for impact damping at the cask-pad interface to represent the dissipation of energy that occurs when the cask and the

concrete pad impact each other during an earthquake event. Singh/Soler Dir. at A160. In the subsequent analyses using VisualNastran, Holtec used higher impact damping values based on analysis and test data that showed that the dissipation of energy through impact damping between a steel and concrete surface is much greater and would justify impact damping values of 40% or more. Tr. 6094-99 (Soler).

184. Impact damping differs from structural or material damping, which is the damping or loss of energy associated with the deformation of structures and materials. Impact damping is the loss of energy associated with the impact of two bodies, such as that of a ball bouncing on a rigid surface. Tr. 6095-6099, 9658-59 (Singh/Soler). Holtec did not take credit for structural or material damping of the cask, canister and their internals in its cask stability analysis. Holtec similarly did not take credit for the damping associated with internal impacts within the cask and canister. Tr. 9658-59 (Singh). Holtec's analysis was therefore conservative in that it neglected damping everywhere, except that due to impacts between the cask and the pad.[23]

185. To account for the damping associated with vertical cask-pad impacts, Holtec included dampers in its model in parallel with the vertical compression springs between the casks and the pads representing the vertical contact stiffness at the cask-pad interface. Holtec did not include any dampers associated with the horizontal frictions springs between the cask and the pad. Tr. 5904-05, 10639-640 (Soler); see also PFS Exh 86C at 15 (sketch of vertical springs and dampers as cask-pad interface). Therefore, upon sliding of the casks, friction will be the sole source of

[23] Holtec did include radiation damping in its modeling of the soil, which is a separate topic discussed later.

energy dissipation, and upon rocking and tipping of the casks resulting in impacts between the cask and the pad, the dampers in parallel with the vertical contact springs will be the sole source of energy dissipation. Id; see also Singh/Soler Dir. at A160-164.

186. Dr. Kahn takes issue with Holtec's use of 5% damping in its DYNAMO model for the 2,000 year design basis earthquake. Dr. Kahn claims that the "results" of his cask stability analyses "show that 5% damping values significantly reduce the estimated cask response, whereas in reality the BETA damping would be small or insignificant for a rigid cask, and only friction should be the primary energy dissipation mechanism." Dr. Khan states that "Holtec's use of high BETA damping coefficients also underestimates potential sliding and rocking displacements." Khan/Ostadan Dir. at A30; see also Tr. 9392-93 (Khan).

187. The BETA damping to which Dr. Khan refers is a method of finite element modeling that accounts for structural damping by applying a multiplier to the model's stiffness elements. Tr. 5902-03 (Soler). Dr. Khan's report specifically refers to the damping that he utilized in his modeling of cask stability as "structural damping." State Exh. 122, "Analytical Study of HI-STORM 100 Cask System for Sliding and Tip-Over Potential during a High-Level Seismic Event," Technical Report No. 01141-TR001, Rev. O ("Altran Report") at 12. Dr. Khan further confirmed at the hearing that he utilized structural damping associated with the "whole structure" in his analyses of cask stability. Tr. 9396-97 (Khan).

188. Structural damping is significant for structures and components that are anchored, as is typically the case for the nuclear power plant structures and components with which Dr. Khan is familiar. The principal mode of damping for a free-standing structure, however, is impact damping, not structural damping, for which different

physical principles and analyses are applicable. Tr. 9658-65; 9915-16 (Singh/Soler); Singh/Soler Dir. at A160-A161; PFS Exh TT; PFS Exh 225 at 22-24.

189. Therein appears to lie the difference of opinion between Dr. Khan and Holtec. Dr. Khan used structural damping in his analysis; as noted above, Holtec did not. Tr. 5902-05, 6095-6099, 9658-59 (Singh/Soler). Dr. Khan's assessment of Holtec's modeling is therefore based on the mistaken assumption that Holtec used structural damping in its modeling. Id; see also Singh/Soler Dir. at A160. This mistaken view leads to errors in Dr. Khan's evaluation of the appropriateness of the damping values used by Holtec.

190. First, because Dr. Khan mistakenly regarded structural damping as the applicable form of damping, he viewed damping as being associated with all of the stiffness elements in his model. As a result, he included dampers with the horizontal friction springs as well as the vertical compression springs. Tr. 9397-98 (Khan). Dr. Khan acknowledged that if there were no dampers associated with the horizontal springs, friction would be the energy dissipation mechanism in a purely sliding situation, with no dissipation of energy due to damping. Tr. 9399-9400 (Khan).

191. As stated above, the Holtec model does not include dampers in parallel with the horizontal friction springs and accordingly there is no dissipation of energy due to damping in Holtec's model with respect to cask sliding. Therefore, Dr. Khan's criticism that Holtec's high values of damping result in undue energy dissipation in the sliding of the casks (see, e.g., Tr. 9392-93) is unwarranted.

192. Second, Dr. Khan's perception that the applicable damping is structural damping led him to conclude that the damping values and recommendations in Reg. Guide 1.61 for structural and material damping were applicable for Holtec's analysis.

Tr. 9408, 9804-05 (Khan). However, Reg. Guide 1.61 concerns structural damping associated with deformation of structures and materials under stress. Tr. 9657-58, 9722 (Singh); Tr. 9805 (Khan). It does not apply to impact damping and is therefore not applicable to Holtec's analyses.

193. To illustrate the reasonableness of the impact damping values used in Holtec's cask stability analysis, Drs. Singh and Soler provided computer simulations showing the effect of impact damping on a ball or cask dropped from a height of 18 inches using impact damping values of 1%, 5% and 40%. At 1 percent damping, which is the value that Dr. Khan would have Holtec use in accordance with structural damping guidelines, the ball or cask would require more than 73 bounces before it came to rest; at 5% the ball or cask would come to rest after approximately 14 bounces and at 40% the ball or cask would come to rest after 2 or 3 bounces. Tr. 9664-68 (Soler); PFS Exh 225 at 22-24; PFS Exh 225A, file "three dropped spheres 4-23-02.avi". Based on this demonstration, it is clear that Dr. Khan's damping values of 1% or less used in his cask stability computer runs are completely unrealistic.

194. Dr. Khan's professional experience has mostly involved bodies that were anchored and none of his prior experience involved evaluating the loss of energy between a body and a concrete pad. Therefore, he does not know whether the appropriate percent of damping to represent the loss of energy from the cask hitting the pad "would be 1 percent, 3 percent, 5 percent or some other percent" and he is "not purporting to give an opinion as to what percent damping would or would not be appropriate in this case." Tr. 9412-13 (Khan).

195. Drs. Singh and Soler, on the other hand, have extensive experience evaluating the dynamic analysis of free-standing objects, including both free-standing spent fuel

racks and spent fuel storage casks. Singh/Soler Dir. at A26-A30; see also Tr. 5903-05. (Soler). They have thoroughly studied and evaluated the impact damping associated with a spent fuel storage cask hitting a concrete pad using applicable physical principles, available test data, and computer simulation. Tr. 6096-99, 9659-64 (Singh/Soler); Singh/Soler Dir. at A160-A161; PFS Exh TT; PFS Exh 225 at 22-24.

196. Based on the record, the Board concludes that Holtec used reasonable damping values to model the impact damping between the cask and the pad. The 5% impact damping value used in the DYNAMO cask stability analyses for the 2,000 year return period design basis earthquake is a conservative choice to represent impact damping at the cask-pad. Holtec's modeling of damping is also conservative in that it ignores the other modes of damping that would occur during an earthquake event.

197. For the same reasons, the higher damping values used in the beyond design basis analyses are also realistic and proper values to be used for beyond design basis analyses, in which the purpose is to realistically assess the margins provided for by a structure's design. Tr. 12954 (Cornell).

d. Evaluation of Cask Stability Analyses Performed by Sandia Laboratories for the Nuclear Regulatory Commission Staff

198. The NRC Staff performed its own cask stability analysis using a different methodology than that employed by Holtec. For the 2,000-year design basis earthquake the NRC predicted maximum cask displacements on the order of 3 to 4 inches, and maximum rotation of 0.40 degrees. Luk/Guttman Dir. at A13, A16. For the 10,000-year return period beyond design basis earthquake, the NRC

methodology predicted maximum cask displacements on the order of 15.94 inches and cask rotation of 1.16 degrees. Luk/Guttman Dir. at A13, A16. The results of the NRC Staff's analyses, utilizing a different methodology than Holtec's, provide independent confirmation of the Holtec results, that the cask displacement for the 2,000 design basis earthquake is on the order of a few inches and that even for the 10,000 year earthquake the casks will not tip over.

199. The NRC commissioned Sandia Laboratories to perform a confirmatory analysis of the behavior of the Holtec HI-STORM 100 cask system to be used at the PFSF under the design-basis 2,000-year return period seismic event and under a beyond-design basis 10,000-year return period seismic event. The results were set forth in a report entitled "Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage Facility", Rev. 1 (Mar. 31, 2002) ("Sandia Report"). Staff Exh. P; Luk/Guttman Dir. at A3(b). The Sandia Report analyses predicted only limited motion of a cask during the ground motions for either a 2,000 or 10,000 year return period earthquake and showed that the casks would not tip over during either seismic event. Luk/Guttman Dir. at A16, A17. Further analysis of the behavior of the storage cask and pad system under actual earthquake time histories from the large 1971 San Fernando Earthquake (Pacoima Dam record) confirmed that cask movement would be minimal and that the casks would not tip over if subject to the ground motions produced by a "real" earthquake with ground motions similar in magnitude to the 2,000-year design basis earthquake. Luk/Guttman Dir. at A10.

i) Methodology

200. To perform their analyses, Dr. Luk, and his Sandia colleagues, designed a three-dimensional finite element model using the ABAQUS/Explicit code, a state-of-

the-art, industry-accepted and verified finite element modeling code. The model contained finite element submodels of:

- the HI-STORM 100 storage cask;
- the storage pad;
- the layer of aggregate surrounding the storage pads;
- the layer of soil cement surrounding the storage pads;
- the layer of cement-treated soil underlying the pads and the soil cement layer; and
- the underlying soil foundation.

Staff Exh. P § 3.2.3.

201. Each of the submodels, with the exception of the cement-treated soil layer, accurately reflected the dimensions and properties of the corresponding component as reflected in design parameters or test data (e.g., underlying soil characteristics obtained through test data were used in modeling the soil foundations). See Staff Exh. P, Ch. 3, §§ 3.2, 3.3, and 3.4. The model included interfaces between: (1) the cask and pad; (2) the pad and underlying cement treated soil; and (3) the cement treated soil and underlying soil foundation. Luk/Guttman Dir. at A7.

202. The cement-treated soil layer was modeled as having both the maximum depth of the cement-treated soil layer at the PFSF and as having a Young's modulus of 270,000 p.s.i., higher than the maximum 75,000 p.s.i. that the cement-treated soil layer at the site would have. Staff Exh. P Ch. 3. Both of these assumptions conservatively maximize the seismic loads transferred from the underlying soil foundation to the storage pad and cask and therefore maximize the potential for horizontal cask displacement due to sliding, cask rotation and potential tipover. Tr. 11542-46, 11624-25 (Luk). The choice of a higher value of Young's modulus

was appropriate, because sensitivity studies conducted by Sandia indicated that the dynamic behavior of the soils, including the cement-treated soil, was relatively insensitive to variation in the value of Young's modulus, demonstrating that changes in the Young's modulus value would not have significant non-linear effects in running a dynamic analysis of cask stability. Tr. 11631-32 (Luk).

203. The dynamic responses of the storage cask were obtained by applying to the model the seismic time histories generated for the 2,000 and 10,000 year earthquakes and the time histories generated by the 1971 San Fernando Earthquake. The analyses used three different sets of soil properties and varied interface conditions for each seismic event to envelope a variety of conditions that may hypothetically be present at the PFSF. See Staff Exh. P §§ 3.5; Luk/Guttman Dir. at A10.

204. Various interface conditions were used to bound maximum horizontal sliding displacement and angular rotation of the cask. The Sandia team used sensitivity studies, testing various combinations of upper bound and lower bound coefficients of friction at each of the model interfaces, in order to determine which parameters would maximize cask response to either sliding or tipping. Luk/Guttman Dir. A11. In the final analyses, the coefficients of friction at the soil cement/soil and the pad/soil cement interfaces were set at 1.0 (upper bound) and 0.31 (lower bound). The coefficients of friction for the cask/pad interface were 0.8 (maximizing cask rotation, i.e., potential for tipping) and 0.2 (maximizing horizontal sliding displacement). Luk/Guttman Dir. at A11.

205. The modeling of interface conditions through the use of coefficients of friction is a well-established method of finite element analysis. Tr. 11587-88 (Luk). The use of an upper bound and a lower bound coefficient of friction is intended to

bound a range of possible conditions that may exist, hypothetically, at the interface. The upper bound of 1.0 minimizes the possibility of sliding at the interface, whereas the lower bound of 0.31 allows for the possibility of the materials at the interface to slide and move relative to one another causing displacement. These values are not intended to represent material properties but only the interface itself; the material properties at the interfaces are incorporated into the finite elements that comprise the abutting soil layers at the interface of the conditions at the PFSF, but to envelope all possible conditions. Tr. 11573-76, 15888 (Luk).

206. The soil foundation was further submodeled to a depth of 140 ft. using six horizontal layers. Luk/Guttman Dir. at A12; Staff Exh. P § 3.4. For the 2,000 year DBE and Pacoima Dan analyses, three sets of soil profile data – upper bound, best estimate and lower bound – were used to envelop all possible soil conditions at the site and bound their effects on the dynamic behavior of the storage casks, including any synergistic effects that may be attributable to non-vertically propagating waves and pad flexibility. Luk/Guttman Dir. at A12; Staff Exh. P, § 3.4, Tables 2, 3 and 4; Tr. 6820-21 (Luk). For the 10,000 year earthquake, the soil profile data were adjusted to account for the shear strain effects of the higher earthquake loadings on soil properties. Luk/Guttman Dir. at A.12; Staff Exh. P § 3.4, Tables 5, 6, and 7.

207. The Sandia Report analysis also evaluated the effect of the presence of soil cement, cement-treated soil, and adjacent storage pads loaded with casks on the behavior of the storage pad and cask system. The Sandia Report analyzed two alternate models in order to assess the potential effects on cask stability of seismic forces being imparted from neighboring pads. Luk/Guttman Dir at A13. In the first alternate model, the Sandia Report analyzed cask behavior if the soil cement

layers were removed from the model. Luk/Guttman Dir. at A13. In the second alternate model, the dead loads of seven additional casks on the storage pad, and the dead loads of fully loaded neighboring pads were included in the coupled model to assess the potential effects of pad-to-pad interaction and transfer of seismic forces from neighboring pads. Luk/Guttman Dir. at A13; Staff Exh. P, Ch. 4 at 28; Tr. 6782 (Luk).

ii) Results

(a) Horizontal Sliding Displacement

208. The maximum horizontal cask sliding displacement produced by any 2,000 year DBE simulation was 3.98 inches. A similar maximum horizontal cask displacement of 3.0 inches was obtained from the analyses using the Pacoima Dam record. For the 10,000 year return period earthquake analyses, the maximum horizontal cask sliding displacement was 15.94 inches. Luk/Guttman Dir. at A13. Thus, if two casks were to move hypothetically toward one another at the maximum horizontal sliding displacement, even under the postulated 10,000-year earthquake they would move approximately only 32 inches. See Luk/Guttman Dir. at A15. Because the distance between neighboring storage casks is 47.50 inches, the casks would not collide. See Luk/Guttman Dir. at A15.

209. Further, Sandia's analysis of the effects of soil cement and pad-to-pad interaction demonstrated the insignificance of such concerns. In the first alternate model (where all soil cement was removed and the pad sat on a free field), the analysis showed that the maximum horizontal sliding displacements of the cask were less than the original coupled model. This illustrates that allowing the pad to slide actually reduces cask movement, due to the dissipation of force due to pad sliding. Luk/Guttman Dir. at A13; Tr. 6783-85 (Luk). In the second alternate model (in-

cluding the effects of fully loaded neighboring pads), the analysis showed that a smaller maximum horizontal sliding displacement resulted from this model than the maximum displacement in the original coupled model.

(b) Cask Rotation

210. Maximum cask rotation was measured in either horizontal direction with respect to the vertical axis. Under the 2,000 year DBE, a maximum cask rotation of 0.40 degrees or less was achieved out of all models examined. Luk/Guttman Dir. at A16. The maximum cask rotation for the Pacoima Dam record ground motions was equal to or less than 0.07 degrees, and 1.16 degrees for the 10,000 year ground motions for all models examined. Luk/Guttman Dir. at A16. Thus, the analyses did not result in a cask tipover – the cask rotations never came anywhere near the approximately 29 degree rotation required to cause cask tipover - under any conditions examined, which included incorporating the effects of pad-to-pad interaction, and incorporating the effects of non-vertically propagating waves.

(c) Uplift

211. The cask did not lift off the surface of the storage pad in any of the 2,000 year DBE or Pacoima Dam record analyses. Luk/Guttman Dir. at A19. Under a permutation of the 10,000 year return period ground motions, the analysis predicted a cask uplift (i.e., the cask base was entirely lifted off the surface of the storage pad) of 0.26 inches for less than 0.30 seconds. Luk/Guttman Dir. at A19.

iii) *Challenges Raised by State of Utah*

212. In oral rebuttal, the State's witness Dr. Bartlett testified that he could not comment on the appropriateness of the modeling technique used by Dr. Luk and his Sandia colleagues, but restricted his comments to the properties of the materials

as listed in the Sandia Report and whether those properties were representative of conditions at the PFSF site. Tr. 10347 (Bartlett).[24] Dr. Bartlett cited three concerns regarding the Sandia Report analyses:

- Dr. Bartlett believed that the use of coefficients of friction at material interfaces appeared to model the materials at the PFSF site incorrectly. Specifically, he believed that the use of coefficients of friction at the interfaces between materials improperly treated the underlying soil foundation and the cement-treated soil layer as granular or frictional materials. Tr. 10348-59, 10530, 10534-35 (Bartlett). Moreover, he believed that this also failed to account the cohesion of the materials. Tr. 10533-34 (Bartlett).

- Dr. Bartlett further asserted that using coefficient of frictions to model the interface would allow sliding at the interface contrary to the PFS design to prevent sliding at the cement-treated soil/soil foundation interface (Tr. 10350, 10377? (Bartlett)); and

- Dr. Bartlett noted that the cement-treated soil underneath the pad was assigned a Young's modulus of 270,000 p.s.i., rather than the 75,000 p.s.i. limit imposed by the PFS design. Tr. 10352, 10378 (Bartlett).

213. Dr. Bartlett concluded that, according to his understanding of how the model was developed, the model would greatly underestimate the actual forces necessary to cause sliding of the pad, which would underpredict the inertial forces transferred to the pads and casks. Tr. 10377 (Bartlett). As the Sandia Report and Dr. Luk made clear, these concerns are unfounded. But, to the extent that sliding of the cement-treated soil relative to the soil foundation or of the storage pad relative to the soil cement may occur, Dr. Bartlett agrees that such sliding would dissipate energy and would reduce the seismic forces acting on the casks. Id.

[24] Despite this profession of limited expertise, Dr. Bartlett's rebuttal testimony extended beyond his professed limits. He passingly referred to the Pacoima Dam record as smaller in comparison to the PFSF design basis earthquake because it was 0.461g in the horizontal and 0.433g in the vertical direction. Tr. 10535 (Bartlett). Dr. Bartlett offered no opinion regarding what the effect, if any, of this difference would have on the Sandia analyses that used the Pacoima Dam record. Dr. Luk nevertheless addressed why the use of the Pacoima Dam record was appropriate for sensitivity analyses. Tr. 11553-55 (Luk).

(a) Use of Coefficients of Friction at Material Interfaces

214. Dr. Bartlett was concerned that the use of a coefficient of friction at the interfaces did not appropriately model the properties of the interface. For example, he misunderstood a coefficient of friction at the interface between the cement-treated soil and the underlying Bonneville Clay soil foundation to treat the Bonneville Clay as if it were "sand," which would only occur after yield failure of the soil. Tr. 10533-35 (Bartlett). But, as Dr. Luk testified, the coefficient of friction at the interface does not represent a property of a material. Tr. 11573, 11580-81 (Luk). Dr. Bartlett also misunderstood that the coefficient of friction might represent some other granular material at the interface (see e.g., Tr. 10375-78 (Bartlett)), but as Dr. Luk testified, the model did not include a granular material at the interface. Tr. 11587-89 (Luk). Likewise, the model did take into account the internal cohesion of the materials modeled (Tr. 11573-75, 11580-81 (Luk)), contrary to Dr. Bartlett's assertion. Tr. 10349-51 (Bartlett).

(b) Allowing Sliding at the Interface

215. Dr. Bartlett testified that he was concerned that allowing sliding at the interface between the cement-treated soil layer and the underlying soil foundation was contrary to the intent of the PFS design. Tr. 10350-51 (Bartlett). This was a misunderstanding on the part of Dr. Bartlett of what the Sandia analyses represented. The Sandia analyses enveloped all possible conditions at the PFSF site in order to assess what would happen to a cask on a storage pad during an earthquake under a variety of conditions. Staff Exh. P at 5-6; see e.g., Tr. 11533-37 (Luk). Dr. Bartlett correctly pointed out that allowing sliding between the cement-treated soil layer and the underlying soil foundations would reduce inertial forces acting on casks sitting on a storage pad (i.e., sliding is beneficial to cask stability). Tr.

10375-77 (Bartlett). Allowing the possibility of sliding by using a lower bound coefficient of friction at the cement-treated soil/soil foundation and storage pad/cement-treated soil interfaces (0.31), was one set of bounding cases the Sandia Report examined. State Exh. P at 5-6; Tr. 11533 (Luk). However, Dr. Bartlett apparently failed to note the other set of bounding analyses where a coefficient of friction 1.0 was used at both interfaces, meaning that potential sliding would be minimized at those interfaces. Tr. 11588 (Luk). Minimizing sliding at those interfaces would maximize inertial forces acting upon a cask sitting on a storage pad. The actual interfaces, the displacement that occurred between the cement-treated soil and the underlying soil foundation or the cement-treated soil and storage pad was very small with no significant relative displacements even for the 10,000 year return period ground motions. Tr. 11516-29, 11575-78, 11586-88, 11610-11 (Luk); Staff Exh. YY. The displacements observed were "well within the elastic" properties of the soil and cement treated soil. Tr. 11529, 11578 (Luk).

(c) Young's Modulus

216. Dr. Bartlett also challenged the fact that the Sandia analyses used a higher Young's modulus for the cement-treated soil layer (270,000 p.s.i.) than that expected to be used at the PFSF (75,000 psi). Tr. 10377-79 (Bartlett). Dr. Bartlett did not state what, if any, difference he believed that this may have on the accuracy of the Sandia results. Dr. Luk testified that the higher value of Young's modulus would conservatively maximize the seismic loads transferred from the underlying soil foundation to the storage pad and cask and therefore maximize the potential for horizontal cask displacement due to sliding, cask rotation and potential tipover. Tr. 11542-46, 11624-25 (Luk). Likewise, as discussed above, using

a higher value of Young's modulus does not affect the dynamic behavior of the cement-treated soil, because soils have been demonstrated by sensitivity studies to be relatively insensitive to variation in the value of Young's modulus. Thus, changes in the Young's modulus value would not likely have significant non-linear effects in a dynamic analysis of cask stability. Tr. 11631-32 (Luk).

217. Thus, the State's concerns regarding the modeling of the interface, the allowance for sliding in some cases, and the value of the Young's modulus for the cement-treated soil layer in the Sandia analyses are unfounded. The Sandia analyses appropriately model the properties of each of the materials present in the PFSF design and on-site at the PFSF site. The use of a higher Young's modulus for the cement-treated soil is a conservative design element that addresses, inter alia, the State's concern about underestimating forces transferred to a cask. Likewise, the model appropriate takes into account bounding conditions at the interfaces between these materials. It demonstrates that sliding of the pads is beneficial to cask stability, and that even in the absence of any sliding of the pads, cask displacement and rotation remain minimal under any possible conditions at the PFSF site.

e. Evaluation of Cask Stability Analyses Performed by the State of Utah

i) Methodology

218. The State of Utah challenges the validity of both the Holtec and NRC analyses, claiming that deficiencies exist in both methodologies. Khan/Ostadan Dir. at A4. The State conducted its own analysis in an attempt to demonstrate these asserted deficiencies in the Holtec methodology and analyses which was performed on its behalf by Dr. Moshin Khan of Altran Corporation. See the Altran Report, State

Exh. 122.[25] As discussed below, Dr. Khan's analyses are flawed in several important respects. He used unreasonable values for key input parameters and used a computer code and model that had not been validated or benchmarked to show that it could make accurate predictions.

219. As part of the analysis, Dr. Khan ran three models. His initial model is a simple mass weighing 360,000 lb that can slide. Dr. Khan used this simple mass model to benchmark the computer code used in his analysis, SAP2000, by running the model on both ANSYS and SAP2000. The second model simulates a HI-STORM System cask by a small, single, rigid beam element that can slide and uplift. The third model simulates a HI-STORM System cask using 72 beam elements. The Altran Report claims that under this third model the "cask can slide, lift and rock, or tip-over under the specified seismic impact motions." Altran Report at 12. However, in both his pre-filed testimony and at the hearing, Dr. Khan stated that this third model could not be used to model cask tipover because SAP2000 is not a large deflection computer code. Kahn/Ostadan Dir. at A25; Tr. 7173-74 (Khan).

220. In the second and third models run on SAP2000, Dr. Khan performed several analyses in which he attempted to show the effect of changing various parameters (contact stiffness, coefficient of friction, and damping) that may bear upon the movement of a HI-STORM System cask on a concrete storage pad during a seismic event. He varied the coefficient of friction as Holtec had done, using coefficient of friction values of 0.20 and 0.80 for the cask-pad interface. For the values of the vertical contact stiffness and impact damping between the cask and the pad, he chose a wide range of values seeking to show what effect changing these pa-

[25] The State conducted no analyses to controvert the results obtained by Sandia for the NRC Staff.

rameters would have on the movement of the cask on the concrete storage pad. Altran Report at 10-13.

221. The cask displacements predicted by Dr. Khan's analysis ranged widely from a few inches to many feet. One run showed the cask lifting up from the surface of the pad by more than 2 ft. and moving laterally on the order of 40 ft. Altran Report at 13, Table 3, Study Run 1. Dr. Khan did not claim that the results of this or other runs were a "correct" or "realistic" prediction of what would occur at the PFSF under earthquake conditions, but that the purpose of his runs was to show the wide variability of the results that could occur from choosing different parameters at the cask-pad interface to model movement of the casks on the pad. Tr. 7178-79 (Khan). Similarly, the State's expert Dr. Ostadan, who testified with Dr. Khan, readily acknowledged that he did not believe the results of Dr. Khan's analysis, particularly that the casks would lift 2 ft. up in the air and move 40 feet under earthquake conditions. Tr. 7391-92 (Ostadan).

ii) Issues Concerning Dr. Khan's Analysis

(a) Dr. Khan's Experience and Expertise

222. Dr. Kahn has <u>no</u> experience conducting evaluations or analyses of the stability of free-standing casks (such as those to be used at the PFS facility) in the event of an earthquake. Tr. 7136 (Khan); PFS Exh. 88 at 67. Moreover, he has virtually no experience whatsoever in analyzing the sliding and tipping of large free standing objects. Previously he had only analyzed the potential sliding of some small hypothetical blocks (weighing on the order of 100 or 200 lbs.) and had reviewed an analysis done by Holtec for the free-standing spent fuel racks for Diablo Canyon in the late 1980s, for which he did some confirmatory analyses. Tr. 7142-53, 9470-71 (Khan); PFS Exh 88. at 23-24, 67-69. With respect to the latter, his re-

view did not focus on the potential sliding or tipping of the spent fuel racks, but on their structural strength; he acknowledged that the Holtec sliding analysis for the spent fuel racks was far more sophisticated than his "simple" analysis. Tr. 7146-47 (Khan); PFS Exh 88. at 37-38. He had in the past reviewed the stability analyses that Holtec performed for dry cask storage at Diablo Canyon and Humboldt Bay, and did not take issue at the time with how Holtec performed the analysis and did not advise Holtec to change in any way its modeling of the casks' stability. Tr. 7154-55 (Khan).

223. Further, Dr. Khan has published no papers and made no conference presentations concerning the seismic analysis of free-standing objects. Tr. 7154 (Khan). Finally, unlike Holtec, Dr. Khan has no work documented at the NRC that supports the licensing or design basis for a free-standing object. Tr. 7155 (Khan).

224. The major fault that Dr. Khan finds with respect to the Holtec cask stability analysis is Holtec's choice of vertical contact stiffness for the cask-pad interface. Dr. Khan, however, has previously never selected a "contact stiffness" value for purposes of analyzing the sliding or tipping of a free-standing object, such as a storage cask. He was not aware of the Hertzian method for calculating contact stiffness referenced in the ANSYS training manual. Tr. 9382 (Khan). This is a standard method for developing a contact stiffness. Tr. 9618-19 (Soler)

225. Virtually all of Dr. Khan's work has involved seismic qualification of pieces of electrical equipment that are anchored, not free-standing. Because they are anchored, sliding and tipping are not of concern. Further, such equipment is much smaller than the casks involved here, and the tolerances of the equipment are much narrower than for the casks, and must be satisfied to ensure their proper op-

eration. Tr. 9680-81 (Singh) Thus, the nature of the equipment with which Dr. Khan did the bulk of his work is far different than free-standing spent fuel casks.

226. In summary, Dr. Khan has essentially no relevant experience in analyzing large free standing objects such as the HI-STORM 100 storage casks.

(b) Dr. Kahn's Use of SAP2000

227. Dr. Khan chose to perform his parametric analysis for different contact stiffnesses and impact damping using the SAP2000 computer code. He testified that the only reason he chose SAP2000, as opposed to a more general purpose program such as ANSYS, is because SAP2000 has a very efficient solution algorithm that takes less time to run than ANSYS. Tr. 7171 (Khan); see also Tr. 9346 (Khan).

228. SAP2000 is highly focused on the analysis of structures and is "designed to be used for structural systems which are primarily linear elastic." Tr. 7159, 9343-46 (Khan). It, however, does have the capability for "a limited number of pre-defined nonlinear elements" and may be used "to model **local** structural non-linearities such as gaps, isolators and the like." Tr. 9346-48 (Khan) (emphasis added). Thus, SAP2000, like DYNAMO, is a small deflection program. Tr. 7173-74 (Khan).

229. Dr. Khan insisted on cross examination that the fact that SAP2000 is a small deflection program did not bring into question the validity of the results of his analysis, in particular runs 1 and 3 of his third model, which show casks lifting off the ground by one or two feet and moving laterally 30 to 40 feet. Tr. 9348-60 (Khan); see also Altran Report at 13, Table 3, Studying, Runs 1 and 3. Dr. Kahn explained these results by stating that in these runs, the casks (1) moved essentially straight up by more than one or two feet, but did not rotate significantly, and

(2) moved laterally 30 to 40 feet in relation to the pad and the ground by bouncing up and down on the pad. Dr. Khan claimed that, because the casks assertedly did not rotate significantly, his analysis did not run afoul of the small deflection limitations of SAP2000. See Tr. 9354-60, 9512-15 (Khan); PFS Exh. 89. Only if the casks showed large rotations would Dr. Khan consider there to be geometric nonlinearities that would affect the validity of his SAP2000 results. Id.

230. These results are even more suspect because Dr. Khan's model has not been accepted for use anywhere. Dr. Khan has acknowledged that he did not take steps to assure that his cask stability modeling could reproduce known classical solutions, benchmark his model, or otherwise produce valid results. See Tr. 7219-20 (Khan). For example, he did not attempt to compare solutions derived from simulations using his models with known classical solutions, as the NRC had required Holtec to demonstrate with respect for DYNAMO, and as mandated by ASME NQA-2a-1990. Singh/Soler Dir. A129 & A134.

231. The only step Dr. Khan took to attempt to validate his model was to compare the solution of his initial simple mass model using SAP2000 with runs using the program ANSYS. This exercise, however, only demonstrated that the model algorithm had been properly programmed using both computer codes, such that when both programs were given the same model input they provided the same output. As Dr. Khan readily acknowledged, this did not establish that the model was appropriate; the same wrong input parameters to two valid computer codes would lead to equally erroneous result for both. Tr. 7157-58 (Khan); PFS Exh. PP at 77.

232. That is the case here. The simple mass model that Dr. Khan ran on both SAP2000 and ANSYS is the same model discussed above, which Dr. Khan used for a simple friction problem for which his model predicted 0.71 inches of cask displace-

ment even before the onset of sliding. Singh/Soler Dir. A130 & A147; PFS Exh. 92. Thus, while his two solutions using SAP2000 and ANSYS show good agreement with each other, the input to the codes is erroneous, and leads to results that defy physical reality. Id.

233. Dr. Khan's model was also unable to duplicate known classical solutions. Singh/Soler Dir. A148-151. Dr. Khan's response to the inability of his model to reproduce the solution to classical problems was to say that he could arrive at those solutions analytically, without using his model. Tr. 7219-20 (Khan). However, the point is not whether he could solve the problem analytically but whether he could duplicate the problem's solution by use of his computer numerical simulation, which he did not do. Tr. 9647-52 (Soler).

234. The incredible results obtained from Dr. Khan's model and his choice of contact stiffness are further cast into doubt by Dr. Khan's failure to verify his results in any manner. Although Dr. Khan's model was the first model of a large freestanding structure that he ever produced, it has not been validated, verified, or benchmarked against either any classical problems or against any real world data. Moreover, his code has not been subject to review by any third party and certainly has not been subjected to the kind of scrutiny that an NRC submission would require to validate his results. In short, there are absolutely no independent indicia of reliability that support his model. To the contrary, all comparisons with other models and simulations indicate that Dr. Khan's model's behavior is unrealistic and inherently unreliable as further demonstrated by Holtec's subsequent analysis of Khan's results.

235. Holtec performed an analysis of Dr. Khan's model and input parameters for run 3 of his third model using VisualNastran (PFS Exh. 225 at 15-16, 24-26), a com-

puter code which is acknowledged by the State to be capable of handling large deflections (Khan/Ostadan Dir. at A26).[26] Specifically, Holtec used the same contact stiffness (1,000,000 lbs./in.) and damping at the cask pad interface (1%) as Dr. Khan had used in his run 3, as shown on Table 3 of the Altran Report. As discussed above, these are the two parameters that Dr. Khan claimed that Holtec did not properly apply in its model and which gave rise to the difference between his results and those obtained by Holtec in its cask stability analyses.

236. Holtec could not duplicate Dr. Khan's results using VisualNastran and, even with the unrealistic input parameters used by Dr. Kahn, the VisualNastran simulation showed no bouncing up and down of the cask by one to two feet over a lateral distance of 25 or more feet as predicted by Dr. Khan. Rather, there was only a slight bouncing of the casks up and down and although the casks rocked and tipped, they never came close to tipping over. And instead of the lateral displacement of 25 feet or more, Holtec obtained displacements of less than a foot or two. PFS Exh 225 & 225A; Tr. 9602-04, 9610-15 (Soler). Based on his 40 years of experience and his failed attempt to duplicate Dr. Khan's model, Dr. Soler concluded that the large displacements predicted by Dr. Khan evidently occurred because SAP2000 had been used beyond its small deflection capability, PFS Exh. 225 at 31; Tr. 9603-04, 9615-16; 9925-28, 9651-54 (Soler).

237. In his rebuttal testimony, Dr. Khan claimed that Holtec did not properly replicate his model because Holtec had only changed a few parameters but used the same DYNAMO code which he claimed does not predict rocking behavior properly. Tr. 9795-96 (Khan). On cross-examination, however, he acknowledged that his

[26] Indeed, as discussed below, the State argues that Holtec should have used VisualNastran instead of DYNAMO for its design basis cask stability analyses for the PFSF.

criticism was limited to taking issue with respect to the stiffness and damping values that Holtec had used in its model. Tr. 9799-9801 (Khan). His direct testimony also identified no other aspect of the Holtec modeling with which he took issue. Since Holtec used in its VisualNastran simulation the same stiffness and damping values as Dr. Khan did, Dr. Khan's claim that Holtec had failed to duplicate his analysis is baseless.

238. It is very clear based on the record that the SAP2000 gave erroneous results and that it was used beyond the limits of its applicability.

(c) Choice of Contact Stiffness and Damping Values

239. In addition to the use of SAP2000 beyond its capabilities, the values that Dr. Khan used for vertical contact stiffness of 1 x 10^6 lbs./inch and damping values of .01% and 1% that produced large cask movements were contrary to well understood physical principles. The use of such parameters would as a result lead to totally unrealistic predictions. See Findings 172-179, supra.

f. Cask Stability Conclusions

240. The cask stability analyses conducted by the Applicant and Sandia demonstrate the safety of the PFSF seismic design, the importance of experience in modeling freestanding structures, and the necessity of properly applying an accurate understanding of the underlying physics in order to develop models that produce meaningful results. Both Holtec and Sandia had the resources of extremely experienced individuals who had modeled numerous storage cask stability simulations. Drs. Singh and Soler for Holtec have decades worth of experience in understanding the mechanics and dynamics of storage cask behavior. Dr. Luk at Sandia, with the assistance of a variety of other experts, had been involved in a cask mod-

eling project for several years prior to analyzing the PFSF cask stability issues, has conducted a large-scale generic cask stability analysis, and several site specific cask stability analyses. Holtec and Sandia each constructed models of the cask and storage pad system using well-established modeling methodologies that appropriately and conservatively depicted the properties of the materials used in constructing the storage cask and pad system at the PFSF, including realistic site conditions for soils. The two different methodological approaches taken by the Holtec and Sandia model consistently produced results that showed no tip-over of the casks under a wide range of conditions.

241. In this respect, both Holtec and Sandia ran multiple analyses to envelope all possible conditions that could be present at the PFSF site, and in the case of many Holtec analyses, additional analyses, using unrealistic assumptions designed to maximize cask responses were conducted to further envelope the potential range of results. Under no circumstances analyzed by either Holtec or Sandia for the 2,000-year design basis earthquake was there the potential for storage cask sliding and collision, tipover or uplift. The results of the analyses conducted by Holtec and Sandia for realistic conditions at the PFSF site were generally consistent with one another, both showing small cask displacements and small cask rotations under a DBE.

242. By contrast, the analysis conducted by Dr. Khan was internally inconsistent, showing unrealistically large variations in results based on changes in parameters, and externally inconsistent, incapable of replicating solutions to classical problems, not being subject to any kind of verification or review, generating obvious absurd results, and using input parameters that were obviously unrealistic. Dr. Khan has virtually no prior experience in modeling freestanding structures and

their dynamic behavior. His experience in finite element modeling has been in the modeling of components that are not freestanding, but bolted or otherwise affixed to another surface. This lack of experience is reflected in the anomalous results of his analysis. Indeed, Dr. Khan was not familiar with the basic mathematical methods for determining contact stiffness, the property he believed to be the defect in the Holtec cask stability analyses.

243. During the course of the hearings, Holtec performed numerous analyses that addressed each of the conditions and parameters that the State had claimed were not adequately considered. The results of these confirmatory analyses were uniform and consistent: the storage casks do not tip over, even during a 10,000-year return period ground motion, no matter how the values of contact stiffness, damping or other parameters were changed. This demonstrates the ability of the PFSF cask/pad configuration to withstand design basis (and beyond) seismic loadings without cask tipover.

3. State Claims re Newness of Proposed Seismic Design of Storage Pads and Casks

244. In their prefiled direct testimony on Section D of Contention L/QQ, State witnesses Drs. Bartlett and Ostadan identify as a primary purpose of their testimony to demonstrate that the PFSF seismic design is "unique, unprecedented and unproved." State of Utah Testimony of Dr. Steven F. Bartlett and Dr. Farhang Ostadan on Unified Contention Utah L/QQ (Dynamic Analyses) (inserted into the record after Tr. 7268) [hereinafter "Bartlett/Ostadan Section D Dir."] at A5. The State witnesses go on to identify the features of the design that they regard as unique: "PFS's design contains many unique features. One unique feature of the

PFS design is that there will be thousands of unanchored casks sitting in groups of 2 x 4 casks on concrete pads that are 30 feet wide, 67 feet long and three feet thick. SAR Fig. 1.2-1 (Rev. 21). There will be up to 500 pads in the pad emplacement area and the pads will be surrounded by an approximate 2-foot layer of soil cement and underlain by a 1 to 2-foot thick layer of cement treated soil." Id. at A9.

245. At the hearing, however, the evidence showed that the features the State witnesses identified as unique are not such. The PFSF is not the first or only away-from-the-reactor spent fuel storage facility. Tr. 7304 (Ostadan). The PFSF is not the first or only facility to deploy unanchored HI-STORM storage casks. Tr. 7305-06 (Ostadan), nor is it the first or only nuclear facility that deploys unanchored safety-related equipment. Tr. 7308 (Ostadan). PFSF is not the first or only facility to use shallow concrete pads. Tr. 7362 (Ostadan). To the contrary, the use of concrete pads to support storage casks is a conventional design. Tr. 6633 (Pomerening). And, from the analytical standpoint, having 500 pads at a site as opposed to a few makes little or no difference. Tr. 7029-30 (Luk).

246. The State also raises the use of soil cement and cement-treated soil to enhance the stability of the foundations of the storage pads and the CTB as a unique feature of the PFSF design. Bartlett/Ostadan Section D Dir. at A9. However, as discussed in Section C above, the use of soil cement to provide foundation stability is neither new nor unique to the PFSF.

247. The State witnesses also identified as a "unique feature" of the PFSF design that it utilizes a "controlled sliding" design concept for the HI-STORM Holtec storage casks. Bartlett/Ostadan Section D Dir. at A9. According to the State, "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." Id.

248. These assertions were shown at the hearing to be incorrect. Dr. Ostadan acknowledged that the storage cask design does not "control" sliding, but merely allows it to occur. Tr. 7335-39 (Ostadan). And the fact that sliding may occur in a "uniform and controlled manner" is not a design requirement but, rather, the result predicted by the cask stability analyses conducted by Holtec. Tr. 7341-42 (Ostadan).

249. In addition, the State asserts that PFS uses pad sliding as a mechanism to reduce the seismic loading to the pad foundations. Bartlett/Ostadan Section D Dir. at A9; Tr. 7333 (Bartlett); Tr. 7347 (Ostadan). It is true that, if pad sliding occurs, such sliding has the beneficial effect of reducing the seismic loading to which the cask is subjected. See, e.g., Tr. 7348-49, 7354 (Ostadan); Tr. 6633-35 (Pomerening); Tr. 6155-56 (Trudeau). However, to the extent that such effect occurs, it is again a consequence of the design and not a design feature or mechanism. Tr. 5537 (Tseng). In any event, any facility that features unanchored casks (such as those at the Hatch and San Onofre plants) resting on a concrete foundation will be subject to potential sliding of the foundation, and will thereby experience a beneficial reduction in the seismic loadings on the casks. Hence, this feature of the PFSF is also not unique. Tr. 7306-07 (Ostadan).

250. Ultimately, the State witnesses acknowledged that none of the elements of the PFSF design is unique, but insisted that their combined use is. Tr. 7363 (Ostadan). However, if taken to that extreme, every facility is unique. As the record shows, none of the ISFSIs currently deployed or in licensing is identical to any

other, without this being interpreted by anyone as a deficiency. See, e.g., Tr. 6915, 6999 (Luk); Tr. 6639 (Pomerening).

251. In short, we are not persuaded that there are any "unique" features of the PFSF facility that represent deficiencies, render the design unconservative, or require special scrutiny.

4. State Claims re Alleged Lack of Margin in the Design

252. In their prefiled direct testimony, State witnesses Drs. Bartlett and Ostadan also assert that "the lack of safety elements in PFS's design means that deviations or missteps in estimating the material properties and dynamic response of the casks, foundation structures, soil-cement, cement treated soil and native soils at the proposed facility may be sufficient to create unintended consequences or to result in design failure. This point should be kept in mind when we raise specific challenges to the way in which PFS has conducted its seismic analysis." Bartlett/Ostadan Section D Dir. at A10. At the hearing, the State witnesses reaffirmed that their concerns should be viewed against the backdrop of their opinion that the PFS seismic design lacks sufficient margin to accommodate the potential impact of the deficiencies they allege. They testified as follows:

> DR. OSTADAN: Exactly. I think it is fair to say a lot of my comments would not be here if you had a large margin. I would recognize the short- coming, but I would also recognize they may not adversely impact the design.
>
> Q. All right. And you think the same way, Dr. Bartlett?
>
> DR. BARTLETT: Yes.

Tr. 7390 (Ostadan, Bartlett).