R+S 4225

RELATED CORRESPONDENCE

April 1, 2002

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

DOCKETED
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OFFICE OF SECRETARY
RULEMAKINGS AND
ADJUDICATIONS STAFF

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

APPLICANT'S OUTLINE OF PROPOSED KEY DETERMINATIONS FOR UNIFIED CONTENTION UTAH L/QQ

I. KEY DETERMINATIONS FOR SECTION C OF UTAH L/QQ

1. Applicable standard: 10 CFR § 72.102(d): "Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading."

2. PFS has met standard by conducting comprehensive program of geotechnical investigations and laboratory tests. The soils investigations performed at the PFSF are sufficient to properly characterize the site from geotechnical standpoint. Those investigations have demonstrated that the soil conditions at the PFSF site are adequate for the proposed foundation loading.

3. The soil layering at the PFSF site and the properties of the various soil layers of geotechnical interest are well understood and defined.

4. The soils at the PFSF site are reasonably uniform in the horizontal direction.

5. There is no directly applicable regulatory guidance on the density of borings that should be made at an independent spent fuel storage installation ("ISFSI") site. The considerations that dictate the recommended spacing of borings set forth in NRC Reg. Guide 1.132 do not apply to ISFSIs like the PFSF.

6. The borings drilled by PFS in the pad emplacement area are spaced approximately 600 feet apart. Because the soils at the PFSF site are reasonably uniform, such a density of borings is sufficient.

Template = SECY-055

SECY-02

7. Even if there were variations in soil properties across the pad emplacement area, such variations would be accommodated by the fact that PFS generally used the least favorable measured value of each property (e.g., lowest shear strength) as representative of entire subsoil.

8. PFS concentrated its sampling and its laboratory testing program on the soil layer (known as "Layer 2" or "Lake Bonneville deposit") where the soil is weakest and most compressible, so that the measured and laboratory-determined soil properties are conservative.

9. The soils below the upper 30 ft. of the profile are very dense and have great strength.

10. PFS has conducted continuous sampling through Layer 2 at several boreholes, and has conducted borings deep into the soil profile. The sampling and testing program conducted by PFS is sufficient to establish the properties of the site's soils.

11. There is no applicable regulatory guidance on number of tested samples, continuous sampling or depth to which samples must be taken.

12. PFS conducted a variety of laboratory tests, as called for in NRC Reg. Guide 1.138. Some of the tests that have been called for by the State are either unnecessary (e.g., triaxial extension tests) or adequately substituted for by other tests (e.g., instead of cyclic triaxial tests, PFS conducted resonant column tests).

13. PFS has selected to improve subsurface conditions at the site by installing cement-treated soil with a minimum unconfined compression strength of 40 psi under the cask storage pads, and soil cement with a minimum unconfined compression strength of at least 250 psi around the CTB and around and between the storage casks. These respective soil cement and cement-treated soil characteristics are readily achievable.

14. While the specific application of soil cement and cement-treated soil to an ISFSI is new, the use of soil cement to stabilize foundations is not. The properties of soil cement are well understood and soil cement is a well established technology.

15. PFS has committed in the Safety Analysis Report ("SAR") for the PFSF to demonstrate through testing conducted in accordance with industry standards that the soil cement and cement-treated soil mixes it intends to use meet design requirements. PFS has also committed to follow the recommendations of the American Concrete Institute Report ACI 230.1R-90 (1998), the "State of the Art Report on Soil Cement" with respect to the mix proportioning, testing, construction and quality control of soil cement and cement-treated soil.

16. PFS is in the process of conducting a site-specific test program in accordance with the guidance of ACI 230.1R-90 and intends to complete that program to identify and qualify suitable soil cement and cement-treated soil mixes.

17. Since the design requirements, acceptance criteria, and governing standards for the testing and installation of soil cement and cement-treated soil at the PFSF are well defined, there is no need to conduct any pre-licensing, "proof of design" testing.

18. PFS intends to demonstrate prior to the start of construction that the techniques it uses to install the soil cement and cement-treated soil will not adversely affect the surface of the underlying native soils. Construction techniques and procedures are readily available to avoid construction-related damage to the underlying soils.

19. The formation of tiny, shallow shrinkage cracks is a normal phenomenon in the installation of soil cement and cement-treated soil. Steps can be taken during curing to minimize the potential for crack formation and to seal those cracks that form. Shrinkage cracking, however, will not affect the performance of soil cement or cement-treated soil. The presence of shrinkage cracks could arguably cause slight horizontal motions of the CTB to maintain the passive resistance function of the soil cement, but such motions would have no safety consequences.

20. Soil cement cracking due to differential settlement between the soil cement layer and an adjacent structure foundation, or due to earthquake tension loads, is inconsequential and will have no adverse impact on the safety performance of the storage casks or the CTB.

II. KEY DETERMINATIONS FOR SECTION D.

1. PFS stability calculations for the storage pads and the CTB, performed using the combination of static loads and dynamic loads from the design basis earthquake using very conservative assumptions and methods, demonstrate that the designs have large factors of safety against sliding, overturning and bearing capacity failures.

2. PFS has performed a number of "beyond base case" stability calculations assuming extreme, hypothetical conditions that result in reductions of the factors of safety against failure. Those "what if" cases, although important to identify the bounding characteristics of the PFSF site soils, are not to be taken as representative of anticipated conditions.

3. The stability calculations assume that the pads (and for the CTB, the building's foundation mat) are rigid. The assumption of rigidity is correct, since computations show that both foundations exhibit very small deflections under design basis loads.

4. Use of the peak ground acceleration to compute earthquake loadings on the storage pads is appropriate and yields a factor of safety against sliding that is consistent with that obtained from the time history of forces acting on the base of the pad.

5. The State's concern about unsymmetrical loading on a sliding pad due to collisions with soil cement around the pads is unrealistic because the pads will not slide and if they do the soil cement will move in concert with the pads. Any impact between the pad and the soil cement will be a low energy impact that will have no effect on the stability of the pad or any of the casks. Likewise, there is no potential for pad interaction since there will be no differential sliding of the pads.

6. A similar concern about out –of- phase motion between the CTB and the surrounding soil cement cap will also be inconsequential.

7. Due to the small departure from vertical of their angle of arrival, non-vertically propagating seismic waves will not introduce significant bending or rocking motions in the storage pads, and the effect of such waves is being incorporated into the design of the CTB by the introduction of a mass eccentricity factor.

8. That the soil input parameters used by Holtec in calculating soil spring and damper values are a good approximation for the soil foundation impedances for the fundamental frequency of the soil foundation system.

9. Using a single set of time histories to model the earthquake motions is consistent with NRC regulatory positions and is conservative.

10. Fault fling is conservatively included in the input time histories to the analysis of the storage pads.

11. Any impact on the soil impedance parameters for the CTB due to the presence of the soil cement cap would be minimal and can be disregarded in accordance with standard industry practice

12. Any kinetic interaction between the soil cement layer and the CTB mat foundation can be disregarded.

13. The various claims raised by the State in Section D are either incorrect or if addressed as sought by the State, the resulting variations in the design analyses would be inconsequential and would not affect the adequacy of the final design.

14. Holtec's cask stability analysis shows large safety margins at the 2000 year design basis earthquake, such that any small increase in earthquake loads would be inconsequential.

15. The Holtec computer code used to model the HI-STORM has been validated and approved by the NRC and has been used as the licensing basis for spent fuel technology at more than 40 nuclear plants throughout the world.

16. The Holtec cask stability analysis was confirmed by independent analysis done on behalf of the NRC Staff.

17. The State's witness who modeled the HI-STORM 100 Cask System had never before modeled large free standing objects such as the HI-STORM. He ignored authoritative guidance in his modeling of the HI-STORM 100 cask and made fundamentally flawed assumptions that cause his model to predict results that defy Physics.

18. Holtec's computer code has been benchmarked to provide results that correspond to physical reality.

19. Additional computer simulations by Holtec show that the casks will not tip over under the postulated 10,000 year ground motions, even under unduly harsh worst case assumptions.

III.KEY DETERMINATIONS FOR SECTION E

1. Use of Probability Seismic Hazard Analysis both to characterize the seismic hazard at the site and to set the seismic design basis of the PFSF is fully consistent with both current NRC policy and practices as well as broader engineering policy and practice.

2. Most modern seismic design criteria are based on a graded approach to seismic safety, permitting facilities or structures with less severe failure consequences to have larger mean annual probabilities of failure.

3. Dry cask ISFSIs, such as the PFSF, are recognized by the NRC as being inherently less hazardous than operating nuclear power plants(NPPs) and less vulnerable to earthquake-initiated accidents than an operating NNP.

4. Because of the lower radiological hazards posed by dry cask ISFSIs, it is appropriate to allow a higher probability of failure ISFSIs than NPPs due to an earthquake than for operating nuclear power plants.

5. The average mean Safe Shutdown Earthquake ("SSE") for a typical NPP of approximately $1x10^{-4}$ is the appropriate NPP benchmark on which to determine the higher probability of seismic failure allowed for ISFSIs.

6. Two factors are relevant to determining the likelihood of seismic failure of an important to safety structures systems and components ("SSC") due to an earthquake event. These are (1) the seismic design basis earthquake ("DBE") for the facility or structure and (2) the conservatisms embodied in the codes and standards applicable to its seismic design.

7. SSCs at the PFSF are designed in accordance with the NRC SRPs and other nuclear industry standards that provide comparable conservatisms.

8. Typical SSCs designed to NRC SRPs or comparable nuclear codes have been shown to have large factors of conservatism against seismic failure, on order of 5- to 20.

9. Such typical SSCs at the PFSF would include the CTB and the crane and the seismic struts inside the CTB.

10. One would expect that other SSCs designed to the same conservative nuclear codes and acceptance criteria would have similarly large factors of safety against seismic failure.

11. The foundations for the CTB at the PFSF have quantifiable large margins against failure due to overturning or loss of soil bearing to conclude that no overturning or hazardous-to-release bearing failure would be expected under ground motions with MRPs of more than 5 times the 2000-year DBE

12. There are large margins of safety against overturning and soil bearing failure of the storage pad that allows one to reasonably conclude that no overturning or hazardous-to-

release bearing failure would be expected under ground motions with MRPs of more than 5 times the 2000 DBE.

13. Loaded HI-STORM storage casks will not tipover under a beyond design basis, 10,000 year return period seismic event postulated to occur at the PFSF site.

14. Review of the stability of the casks undergoing a postulated 10,000 year earthquake event reveals large margins remaining against cask tipover and radioactive release.

15. Even assuming hypothetical cask tipover under 10,000 earthquake return period conditions, there would be no breach of the multipurpose canister confinement to case a radioactivity release. Velocities of a seismic initiated tipover would very likely be in the same region as those of the hypothetical tipover event..

16. Even if higher velocities were to occur, the MPC canisters would have large margins to protect against radioactive release.

17. The radiological consequences of cask tipover of a single cask, multiple casks, or any number of casks would be far below the 5 rem limit for accident conditions.

18. Based on the above, the 2000 DBE for the PFSF provides adequate protection to the public health and safety in accordance with well established Commission policy.

19. The above determinations can be easily made without preparing fragility curves or other similarly complex analyses claimed to be necessary by the State.

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CERTIFICATE OF SERVICE

I hereby certify that copies of the Prefiled Testimony of Robert R. Youngs and Wen-Shou Tseng; Krishna P. Singh and Alan I. Soler; Paul J. Trudeau; Bruce E. Ebbeson; Donald W. Lewis; C. Allin Cornell; Paul J. Trudeau and Anwar E. Z. Wissa; and Krishna P. Singh, Alan I. Soler and Everett L. Redmond, and associated Exhibits, Resumes and Prefaces for Unified Contention Utah L/QQ and Applicant's Key Determinations for Unified Contention Utah L/QQ were served on the persons listed below (unless otherwise noted) by e-mail with conforming copies by U.S. mail, first class, postage prepaid, this 2nd day of April, 2002. The ASLB Board members, the NRC Staff and the State of Utah will be served by next-day service. Due to the size of the electronic file, Exhibit OO will be served only by mail, and will be under separate cover.

Michael C. Farrar, Esq. Chairman
Administrative Judge
Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001
e-mail: MCF@nrc.gov

Dr. Jerry R. Kline
Administrative Judge
Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001
e-mail: JRK2@nrc.gov; kjerry@erols.com

Dr. Peter S. Lam
Administrative Judge
Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001
e-mail: PSL@nrc.gov

*Office of Commission Appellate Adjudication
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001

Office of the Secretary
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001
Attention: Rulemakings and Adjudications Staff
e-mail: hearingdocket@nrc.gov
(Original and two copies)

* Adjudicatory File
Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001

Catherine L. Marco, Esq.
Sherwin E. Turk, Esq.
Office of the General Counsel
Mail Stop O-15 B18
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555
e-mail: pfscase@nrc.gov

Denise Chancellor, Esq.
Assistant Attorney General
Utah Attorney General's Office
160 East 300 South, 5th Floor
P.O. Box 140873
Salt Lake City, Utah 84114-0873
e-mail: dchancel@att.state.UT.US

John Paul Kennedy, Sr., Esq.
David W. Tufts, Esq.
Confederated Tribes of the Goshute Reservation and David Pete
Durham Jones & Pinegar
111 East Broadway, Suite 900
Salt Lake City, Utah 84105
e-mail: dtufts@djplaw.com

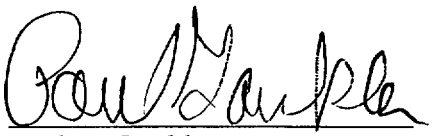
Joro Walker, Esq.
Land and Water Fund of the Rockies
1473 South 1100 East
Suite F
Salt Lake City, UT 84105
e-mail: lawfund@inconnect.com

Diane Curran, Esq.
Harmon, Curran, Spielberg & Eisenberg, L.L.P.
1726 M Street, N.W., Suite 600
Washington, D.C. 20036
e-mail: dcurran@harmoncurran.com

Tim Vollmann, Esq.
Skull Valley Band of Goshute Indians
3301-R Coors Road, N.W.
Suite 302
Albuquerque, NM 87120
e-mail: tvollmann@hotmail.com

Paul EchoHawk, Esq.
Larry EchoHawk, Esq.
Mark EchoHawk, Esq.
EchoHawk PLLC
P.O. Box 6119
Pocatello, ID 83205-6119
e-mail: paul@echohawk.com

* By U.S. mail only



Paul A. Gaukler

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APPLICANT'S PREFACE OF THE TESTIMONY OF DR. ROBERT YOUNGS AND DR. WEN TSENG ON UNIFIED CONTENTION UTAH L/QQ

I. WITNESSES

A. Robert Y. Youngs

Dr. Youngs is a Principal Engineer employed by Geomatrix Consultants Inc., in Oakland, California. He has over 25 years of professional consulting experience, primarily focused in the analysis of seismic hazards. Dr. Youngs' 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. He has conducted these types of analyses for many nuclear power plants throughout the country and the world and has performed similar studies for existing and proposed Department of Energy ("DOE") nuclear facilities at Hanford, Washington; INEEL, Idaho; Rocky Flats, Colorado; Savannah River, South Carolina; and Yucca Mountain, Nevada.

B. Wen Shou Tseng

Dr. Tseng is President of International Civil Engineering Consultants, Inc. ("ICEC"), which provides specialty consulting services in the general areas of civil and structural engineering with special emphasis on earthquake engineering. Dr. Tseng has been doing research and development, and performing consulting services for more than 30 years in the area of earthquake engineering and soil-structure interaction effects on structures. He has published many technical papers and technical and project reports on soil-structure interaction. During his 12 years of experience at ICEC, Dr. Tseng has work for numerous nuclear facilities. For the 12 years prior to his joining ICEC, Dr. Tseng was head of Bechtel's Special Structures Group performing research and development and providing technical consulting services to many nuclear power generating facilities, including the Susquehanna, Limerick, Pilgrim II, Hope Creek, Skagit, Trojan, Tsuruga II, Sequoyah, Browns Ferry, Watts Bar, Bellefonte, and Diablo Canyon. The work on all these plants involved elements of seismic analysis and design of the plant structures, systems and components, including soil-structure interaction.

II. TESTIMONY

A. Scope

Drs. Youngs and Tseng will be addressing in whole or in part issues related to claims raised by the State concerning (1) the effect of non-vertically propagating waves on the storage pad raised in D.1.a and D.1.d of the Unified Contention, (2) flexibility of the storage pads raised in D.1.b; (3) the effect of soil cement around the storage pads raised in D.1.c, (4) the frequency dependency of soil springs and damping values for the storage pad raised in D.1.e, and (5) time histories for fault fling raised in D.1.h(ii) Drs. Young and Tseng will testify based on their evaluation that, even if these claims were addressed as sought by the State, the resulting variations in the design analyses would be inconsequential and would not affect the adequacy of the final design.

B. Non-Vertically Propagating Waves

Drs. Youngs and Tseng will testify that seismic waves will impinge the storage pads with small angles of incidence off the vertical and that within the dominant frequency range of interest for the cask response, the effect of earthquake motions on the pads and the casks resting on the pads at the PFSF may be represented by the use of vertically propagating earthquake waves. The effect of non-vertically propagating waves is insignificant.

C. Rigidity of the Storage Pad

Dr. Tseng will testify that the State's reliance on ICEC's accounting for pad flexibility in performing its detailed structural design to claim that the pad should be considered flexible in evaluating the global dynamic response of the casks and the pads is misplaced because the purposes of the two analysis are different. Dr. Tseng will testify that in accordance with recognized authorities in the field of soil structure interaction the storage pads may be treated as rigid bodies in evaluating their global response. The effect of pad flexibility will be small.

D. Effect of Soil Cement

Dr. Tseng will testify that the loads imparted by the soil cement surrounding the pads will have only second order effects on the stability of the casks.

E. Frequency Dependency of Soil Spring and Damper Values

Drs. Youngs and Tseng will testify that the soil input parameters used by Holtec in calculating soil spring and damper values are a good approximation for the soil foundation impedances for the fundamental frequency of the soil foundation system.

F. Time Histories for Fault Fling

Dr. Youngs will testify that fault fling was conservatively incorporated into the set of time histories that was used for the design of the PFSF.

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JOINT TESTIMONY OF ROBERT YOUNGS AND WEN TSENG ON UNIFIED CONTENTION UTAH L/QQ

I. WITNESSES

A. Robert R. Youngs ("RY")

Q1. Please state your full name.

A1. Robert R. Youngs.

Q2. By whom are you employed and what is your position?

A2. (RY) I am a Principal Engineer employed by Geomatrix Consultants Inc., in Oakland, California.

Q3. Please summarize your educational and professional qualifications.

A3. (RY) My professional and educational experience is summarized in the curriculum vitae attached to this joint testimony. I have over 25 years of professional consulting experience, primarily focused in the analysis of seismic hazards. My 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. I have conducted these types of analyses for seven NRC-regulated nuclear power plants located in the Western United States. I have also performed similar studies for nuclear power plants in Canada, Spain, Slovakia, and Bulgaria, and am currently involved in similar studies for nuclear power plants in Switzerland and Slovenia. In addition, I have performed similar studies for existing and proposed Department of Energy ("DOE") nuclear facilities at Hanford, Washington; INEEL, Idaho; Rocky Flats, Colorado; Savannah River, South Carolina; and Yucca Mountain, Nevada.

Q4. Are you familiar with the Private Fuel Storage Facility ("PFSF") and the activities that will take place there?

A4. (RY) Yes.

Q5. What is the basis of your familiarity with the PFSF?

A5. (RY) I was part of a Geomatrix team that performed the seismic hazard analysis for the PFSF. I was one of the authors of the Geomatrix Report, "Fault Evaluation Study and Seismic Hazard Assessment, Private Fuel Storage Facility." I was specifically responsible for conducting the probabilistic seismic hazard analysis and developing the design basis ground motions for the PFSF from the results. I was also responsible for developing a set of "time histories" to represent the design basis ground motions, and for developing dynamic soil properties for use in the dynamic analyses of the storage cask pads and the Canister Transfer Building ("CTB") at the PFSF. I have also reviewed Unified Contention Utah L/QQ, in which the State of Utah raises various challenges to the seismic analysis for the PFSF site, and related materials.

B. Wen Shou Tseng ("WT")

Q6. Please state your full name.

A6. Wen Shou Tseng.

Q7. By whom are you employed and what is your position?

A7. (WT) I am President of International Civil Engineering Consultants, Inc. ("ICEC"). 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, I am responsible for all aspects of the company operation including technical, administrative, financial, contractual and business development matters.

Q8. Please summarize your educational and professional qualifications.

A8. (WT) My professional and educational experience is described in the *curriculum vitae* attached to this joint testimony. I have been doing research and development, and performing consulting services in the general areas of civil and structural engineering, for more than 30 years. My area of specialization is earthquake engineering with special emphasis on the evaluation of soil-structure interaction effects on structures. I have published many technical papers and technical and project reports on soil-structure interaction subjects.

Q9. What is your experience with nuclear facilities and the NRC's requirements for the design and licensing of dry cask storage systems?

A9. (WT) ICEC has performed work for numerous nuclear facilities, in which I have been personally involved. While at ICEC in the last 12 years, we have performed consulting work on seismic analyses, including analyses for soil-structure interaction, for TVA's Browns Ferry and Bellefonte Nuclear Power Plants, PG&E's Diablo Canyon Nuclear Power Plant, and Taiwan Power Company's Fourth Nuclear Power Plant in Taiwan. Further, during my last 12 years at Bechtel prior to joining ICEC, I was head of Bechtel's Special Structures Group performing research and development and providing technical consulting services to many nuclear power generating facilities, including the Susquehanna, Limerick, Pilgrim II, Hope Creek, Skagit, Trojan, Tsuruga II, Sequoyah, Browns Ferry, Watts Bar, Bellefonte, and Diablo Canyon nuclear power plants. The work on all these plants involved elements of seismic analysis and design of the plant structures, systems and components, including soil-structure interaction.

Q10. Are you familiar with the PFSF and the activities that will take place there?

A10. (WT) Yes.

Q11. What is the basis of your familiarity with the PFSF?

A11. (WT) ICEC is the designer of the reinforced-concrete storage pads to be constructed at the PFSF site on which the HI-STORM 100 storage casks will be placed. In that capacity, ICEC performed the necessary analyses to support the design of the PFSF reinforced-concrete storage pads. ICEC has already designed the pads based on the design calculations. The storage pad, as designed, is a 30-ft. wide, 67-ft. long and 3-ft. thick reinforced concrete pad supported directly on cement-treated soil to be installed at the site. I was the independent reviewer for the ICEC design calculation for the storage pads. As independent reviewer, I was responsible for assuring the technical adequacy of the design calculations and the design. This independent review was made to satisfy quality assurance ("QA") requirements of ICEC for nuclear projects, as specified in ICEC's Quality Assurance Manual for Nuclear Projects.

Based on this experience and my general oversight function of ICEC's activities for the PFSF project over the past several years, I am familiar with the site-specific soil characteristics, design seismic ground motions, and other project design requirements, as specified in the PSFS project's design criteria document. I have also reviewed Unified Contention Utah L/QQ, in which the State of Utah raises various challenges to the seismic analysis for the PFSF, and related materials.

II. RELEVANT PFSF DESIGN AND DESIGN PARAMETERS

A. Design Basis Parameters Developed by Geomatrix for PFSF Design

Q12. Dr. Youngs, please describe the design basis ground motions developed by Geomatrix for the design of the PFSF.

A12. (RY) The design basis ground motions for the PFSF are those for the probabilistic 2000-year return period earthquake for the PFSF site. These motions are represented by a horizontal peak ground acceleration of 0.711g, a vertical

peak ground acceleration of 0.695g, and associated response spectra corresponding to motions at the ground surface in the free field. These ground motions for the design of the PFSF were developed based on the characterization of potential sources of future earthquakes and the characterization of the expected response of the underlying soils, including a surface soil cement layer, to earthquake motions.

Q13. What other related design information did Geomatrix develop as part of its work for the PFSF?

A13. (RY) Geomatrix developed (1) the lower range, best estimate, and upper range soil properties to be used in dynamic analyses for the CTB and the storage pads; (2) the soil mass, soil spring, and soil damping values to be use for dynamic analyses of the storage pads; and (3) the time histories to be used for these analyses. Items (1) and (2) incorporated the presence of the surface soil cement layer.

Q14. What nuclear codes and standards did Geomatrix use in its development of the above design parameters?

A14. (RY) The probabilistic seismic hazard analysis ("PSHA") conducted for the site followed the general provisions for such an analysis presented in Regulatory Guide 1.165. The procedures outlined in Appendix C of Regulatory Guide 1.165 were used to develop the design earthquake response spectra from the results of the probabilistic seismic hazard analysis. The three-component set of time histories was developed to meet the requirements specified in Section 3.7.1.2 of the NRC's Standard Review Plan (NUREG 0800). Dynamic soil properties were developed for the site incorporating the uncertainty ranges recommended in Section 3.7.2 of NUREG 0800 and in the American Society of Civil Engineers Standard ASCE 4-86 for the seismic analysis of safety-related nuclear structures.

Q15. Are these the same codes and standards that one would follow for developing the design of nuclear power plants?

A15. (RY) Yes.

Q16. Did you apply the relevant provisions of these codes or standards in developing the above design information for the PFSF the same way you would have for a nuclear power plant?

A16. (RY) Yes, with the exception that the reference probability used for establishing the design ground motions for the PFSF is not the same as that specified for a nuclear power plant.

Q17. Please identify the soil properties for which Geomatrix developed best, lower and upper range estimates for use in the design of the PFSF.

A17. (RY) The dynamic soil properties developed for PFSF represent the stiffness, mass, and energy dissipation characteristics (damping) of the foundation soils during the design earthquake shaking condition. Two types of soil properties were developed. The seismic response analyses of the CTB were performed using an approach, in which the underlying soil medium is represented by a continuum. For this analysis Geomatrix developed three (layered) models of the site in which the soil stiffness is represented by the compression wave velocity and strain-compatible shear wave velocity of each soil layer, the soil mass is represented by the density of each layer, and the soil damping is represented by the strain-compatible damping ratio for each layer. These three models consist of a lower range estimate, a best estimate, and an upper range estimate. The dynamic analysis of the response of the storage cask pads and storage casks used a lumped-parameter approach in which the dynamic impedance characteristics of the underlying soil medium are represented by lumped soil mass, soil spring, and soil damping (dash-pot) values. For this approach, three sets of soil-springs, soil-masses, and soil dash-pots were developed. These three sets consist again of a lower range estimate, a best estimate, and an upper range estimate.

Q18. Why were two different methods used to develop soil properties for CTB and for the cask storage pads and casks?

A18. (RY, WT) The choice of two different methods of analysis is a matter of convenience and/or necessity, considering the specific design purpose and requirements. Either method will give valid results when properly utilized. For the CTB, which is essentially a linear system, only linear seismic responses are to

be computed, thus representing the foundation soil medium as a continuum and producing a set of frequency-dependent foundation impedance functions is convenient, since the analysis lends itself to a frequency-domain seismic response method. To calculate the seismic response of the free-standing storage casks, which involves nonlinear sliding and rocking responses, a nonlinear time-history response analysis is required. For the cask and pad case, representation of the dynamic characteristics of the foundation soil medium must be provided, represented by frequency-independent lumped parameters that are invariant with respect to time. Therefore, a lumped-parameter approach was adopted for the cask and pad seismic response analysis.

Q19. Why does one need to develop lower and upper range estimates of these soil properties in addition to a best estimate?

A19. (RY, WT) The development of lower and upper range estimates of these soil properties in addition to a best estimate is intended to account both for variations in the soil material properties at the site and for other seismic modelling uncertainties that are difficult to quantify, as discussed in ASCE Standard 4-86 to which the PFSF project has committed.

Q20. How did Geomatrix develop the "best estimate" and the lower and upper range estimates of the soil properties for use in design of the PFSF?

A20. (RY) The best estimate soil properties were developed by first calculating the average seismic wave velocities in the subsurface soils using the data collected from wave velocity measurements at the site. The shear wave velocity and damping in soils is dependent upon the level of shaking, with the shear wave velocity decreasing and the damping increasing as the level of shaking increases. Site response analyses were conducted using the design time histories to obtain the shear wave velocities and damping ratios representative of the design earthquake shaking levels. These are termed "strain-compatible" soil properties. Upper and lower range soil properties were obtained by varying the best estimate soil properties following the guidelines given in the NUREG 0800 and ASCE 4-86. Site response analyses where then conducted using the design time histories

to obtain the strain-compatible shear wave velocities and damping ratios for the upper and lower range soil-property profiles representative of the design earthquake shaking levels.

Q21. What was the range of these soil property parameters as developed by Geomatrix?

A21. (RY) The low strain shear moduli were varied by a factor of 1.5 down to a depth of 30 feet and, varied by a factor of 2 for depths below 30 ft.

Q22. Please describe what time histories represent and how they are used in the seismic design of structures and components.

A22. (RY) Time histories represent the variation of ground acceleration with time during an earthquake. They are used to represent the motions to which the site structures would be subject during the design earthquake.

Q23. Please describe the time histories that Geomatrix developed for the PFSF.

A23. (RY) Geomatrix provided a set of time histories for the 2000 year design basis earthquake for the PFSF site showing the earthquake accelerations in the two horizontal directions (generally referred to as the x and y coordinates) and the vertical direction (generally referred to as the z coordinate). For the PFSF site, the x direction represents east-west motion, which is normal to the faults that are the primary source of earthquake hazards to the site. The y-direction represents north-south motion, which is parallel to these faults. It has been shown that for low frequency motions (generally 1 Hz or less) the fault-normal component of motion is larger that the fault-parallel component, especially when the site is near the causative fault.

Q24. What methodology did you generally follow in developing this set of time histories for use in the PFSF design?

A24. (RY) NUREG 0800 describes two approaches for developing design time histories. One approach is to use multiple sets of time histories that in the aggregate envelop the design response spectra, although any individual time history may fall well below the design spectrum at some frequencies. The second approach is to develop a single set of time histories that envelops the design

response spectra. Time histories developed using the second approach are often called spectrum-compatible time histories. The spectrum-compatible approach was appropriate for use to develop the set of time histories for the 2,000-year design earthquake for the PFSF for the reasons explained in the testimony of Krishn P. Singh and Alon Soler of Holtec, International being filed simultaneously.

Q25. Please describe generally how you developed the set of time histories for use in the PFSF design using the spectrum compatible approach.

A25. (RY) The first step was to select an earthquake recording that is representative of the type of earthquakes contributing to the seismic hazard at the PFSF site. The Sturno recording of the November 23, 1980 **M** 6.9 Irpinia, Italy normal-faulting earthquake was selected. The Sturno site was located approximately 11 km from the northwest end of the fault rupture in the hanging wall block (above the fault), which is generally consistent with the relationship of the PFSF site to the Stansbury fault, the main source of seismic hazard to the PFSF site. The Sturno recording shows evidence of a velocity pulse representative of near-fault effects observed in a number of strong motion recordings. The two horizontal components of motion were rotated into fault-normal and fault-parallel directions. The three components of motion (fault-normal, fault-parallel, and vertical) were then modified until their resulting response spectra enveloped the design response spectra following the criteria specified in NUREG 0800.

B. ICEC Design and Analysis of the PFSF Storage Pads

Q26. Dr. Tseng, please describe the PFSF storage cask pads for which ICEC provided the design.

A26. (WT) The storage cask pads will be independent structural units constructed of reinforced concrete supported directly on cement-treated soil at the site. Each pad will be 30 ft wide, 67 ft long and 3 ft thick and will be capable of supporting eight loaded HI-STORM 100 storage casks. Each pad is designed to accommodate a 2 x 4 array of casks with a 15 ft pitch in the width direction and 16 ft in the length direction.

Q27. Would you please describe the number and relative location of the storage pads to be located at the PFSF?

A27. (WT) The layout of the storage pads is shown in Figure 4.2-7 of the PFSF Safety Analysis Report ("SAR"). At maximum capacity of the PFSF, there would be 500 cask storage pads designed as I described above. The storage pads will be constructed in a regular array with five ft. of spacing between adjacent pads in the longitudinal direction and 35 ft. spacing between adjacent pads in the lateral direction.

Q28. Please describe generally the process by which ICEC went about the design and analysis of the PFSF storage cask pads.

A28. (WT) The initial layout dimensions of the storage pads was provided to ICEC. ICEC then prepared a static and dynamic model of the pad/soil system and performed analyses of the pad/soil system under static and dynamic loading conditions to determine the internal stresses in the storage pad. Holtec provided the cask dynamic response forcing functions at the cask/pad interface boundaries, which were used in ICEC's pad dynamic analyses. The internal stresses calculated in the ICEC analysis were then used to determine the amount of reinforcing steel bars required for the reinforced concrete pad to resist the combined stresses in accordance with the project design criteria.

Q29. What was the purpose of the calculation that ICEC prepared for the design of the storage cask pads?

A29. (WT) The purpose of ICEC's design calculation was to determine the internal stresses induced in the storage pad when subjected to the design loading conditions and to check the ability of the pad as designed to resist the stresses caused by the specified loading conditions. The internal stresses determined from the design calculation were then used for establishing the amount of steel reinforcement required in order for the pad to resist the applied loading conditions. Since the design calculation is used to determine internal stresses under design loadings, the pad itself was modelled as a flexible pad supported on

flexible soil foundations using a finite-element model for the pad and soil spring representation for the soil foundation.

Q30. What nuclear codes and standards did ICEC follow in its design and analysis of the storage pads?

A30. (WT) The codes and standards used in design and analysis of the storage pad are (1) American Concrete Institute ACI 349-85 (1990), "Code Requirements for Nuclear Safety Related Concrete Structures" and (2) American Society of Civil Engineers, ASCE Standard 4-86, "Seismic Analysis of Safety Related Nuclear Structures and Commentary." The seismic soil-structure interaction analyses of the pad/soil system also followed the guidelines recommended in the NRC Standard Review Plan for nuclear power plants, NUREG-0800.

Q31. Are these the same codes and standards that one would follow for the design and analysis of similar structures for nuclear power plants?

A31. (WT) Yes.

Q32. Did ICEC apply the applicable requirements of these codes or standards in its design and analysis of the pads the same as it would have for a nuclear power plant?

A32. (WT) Yes.

Q33. Are there conservatisms embodied in the codes and standards as ICEC applied them in its design and analysis of the storage pads for the PFSF?

A33. (WT) Yes.

Q34. Please describe these conservatisms?

A34. (WT) As with all codes and standards, conservatism exists in specification of load factors, load combinations, and allowable material strengths to be used for the design. Additional conservatism exists in using large variations (a factor of 1.5 to 2 variations) of soil properties in the analyses and using the results enveloped from the lower range, best estimate, and upper range soil cases for design.

III. RESPONSE TO THE STATE OF UTAH'S CLAIMS IN SECTION D

A. Overview of Testimony

Q35. The State of Utah has raised several claims in Section D of Unified Contention Utah L/QQ ("Unified Contention"). Which of those claims will you be addressing in your testimony?

A35. (RY, WT) We will be addressing in whole or in part issues related to (1) the claims raised in Section D.1.a of the Unified Contention concerning non-vertically propagating waves, (2) the claims raised in Section D.1.b of the Unified Contention concerning pad rigidity, (3) the claims raised in Section D.1.c of the Unified concerning the evaluation of pad and cask sliding, (4) the claims raised in Section D.1.d of the Unified Contention concerning lateral variations in ground motion phase, (5) the claims raised in Section D.1.e of the Unified Contention concerning the frequency dependency of soil springs and damping values, and (6) the claims raised in Section D.1.h of the Unified Contention concerning the use of multiple time histories.

Q36. In general, what is your response to these claims raised by the State?

A36. (RY, WT) After review of the claims and examination of certain additional calculations made to evaluate some of the claims, we have concluded that, even if the claims raised by the State were incorporated, the resulting variations in the results of the analyses used for the design, would be inconsequential and would not affect the adequacy of the final design.

B. Specific Responses to The State of Utah's Claims Raised in Section D of the Unified Contention Utah L/QQ

1. Claims Raised in Section D.1.a of Unified Contention – Non-Vertically Propagating Seismic Waves

Q37. Please describe the claim raised by the State in Section D.1.a of the Unified Contention.

A37. (RY, WT) In Section D.1.a of the Unified Contention, the State claims that "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." The State claims that because of the location of the PFSF site near active faults, non-vertically propagating waves with large angles of incidence capable of causing additional rocking and torsional motion may impinge the pad, casks and foundations.

Q38. Do you agree that PFSF's location near active faults is more likely to produce nonvertically propagating seismic wave with large angles of incidence.

A38. No. PFSF's proximity to two active faults does not make it more likely that the incoming waves will have high angles of incidence.

Q39. What is your response to the claims raised by the State in Section D.1.a?

A39. (RY, WT) Based on our evaluation, we have concluded that the angles at which seismic waves would impinge the PFSF site are small (generally less than 10 degrees from vertical), and the waves can, for all practical purposes, be considered to be vertical. 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.

Q40. Dr. Youngs, please describe the analyses upon which you base your conclusion.

A40. (RY) Employing standard methodologies, I calculated the angle of incidence of the earthquake waves impinging the PFSF site originating from the primary sources of earthquake hazards to the PFSF, the Stansbury and East faults. I determined that the angle of incidence would be very close to vertical, typically less than 10 degrees for the frequencies of interest. 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 sites and the assumption of vertically propagating waves is reasonable for the site. This evaluation is set forth in the March 11, 2002 Geomatrix Evaluation of Spatial and Temporal Variation of Ground Motion for the Private Fuel Storage Facility, Skull Valley, Utah ("Geomatrix Evaluation") pages 1-4, identified as PFS Exhibit LL.

Q41. The State's witness Dr. Ostadan testified in his deposition that there are no standard or accepted methodologies for calculating the angle of incidence of earthquake waves. Do you agree with that statement?

A41. (RY) No. The method of ray tracing that I used is described in standard seismology textbooks, such as K Aki and P.G. Richards (1980) Quantative Seismology W.H. Freeman & Co., San Francisco. I confirmed, through discussions with a knowledgeable seismologist, Dr. Walter Silva of Pacific Engineering and Analysis, that the travel path of seismic waves can be readily calculated by what is termed "ray tracing."

Q42. Please describe the methodologies that you used to calculate the angle of incidence and state on what basis you conclude that you employed standard methodologies.

A42. (RY) The direct ray path of a body wave (such as the shear waves of primary interest to the shaking hazard from nearby fault ruptures) from a point source at depth to a point on the surface has two properties. The first is that it represents the minimum travel time path between the two points. The second is that the ray path obeys Snell's law at all layer boundaries such that the ratio of the sine of the angle of incidence (measured from the normal to the layered boundary) to the layer velocity is constant along the ray path ($\sin(i_i)/V_i$=constant). Using these properties, I performed two separate calculations. In the first, I solved iteratively for the minimum travel time path between two points without imposing Snell's law at the layer boundaries in the Skull Valley velocity model. In the second, I imposed Snell's law along the travel path and solved iteratively for the ray angle at the source that resulted in a ray path that reached the surface at the designated site. These two algorithms produced the same travel path. As a further check, I asked Dr. Walter Silva to perform several test calculations using his ray tracing computer program. His results agreed with those that I obtained.

Q43. Dr. Youngs, you referred to the frequencies of interest in your answer to an earlier question. What is meant by frequencies of interest?

A43. (RY) The frequencies of interest for the case of casks supported on pads are the dominant frequencies of the cask response motions, when the casks are undergoing their largest amplitude of dynamic response.

Q44. How did Geomatrix go about determining the frequencies of interest in its March 11, 2002 evaluation?

A44. (RY) As explained in Section C of the March 11, 2002 Geomatrix Evaluation (PFS Exhibit LL), Geomatrix requested and received from Holtec dynamic response time histories obtained at the top of the HI-STORM System casks for the "worst case" evaluations done by Holtec as part of its cask stability analysis for the PFSF 2000 year design basis earthquake. These response time histories (attached as Appendix A to the March 11, 2002 Geomatrix Evaluation) represent movement of the casks in response to the earthquake time histories that Geomatrix provided to Holtec for its analysis of the casks. These response time histories indicate that the largest cask movements occur principally in the time interval 4 to 7 seconds after initiation of the event, as shown in the design time histories. We computed the Fourier spectrum for that portion of the top-of-cask time history and the Fourier spectrum for the same time window of the input time histories that produces the cask response. The peaks in the ratio of these two spectra indicate the predominant frequencies of the cask's response to the input motion. The peak response of the cask occurred in the frequency range of 1 to 5 Hz.

Q45. Dr. Tseng, did you review Geomatrix's determination of the frequencies of peak cask response?

A45. (WT) Yes, I did. Geomatrix used a standard methodology for determining the dominant response frequency of a structure. I have reviewed Geomatrix's calculation results obtained by application of this methodology to the response time histories received form Holtec and agree that the peak cask response frequency range is between 1 and 5 Hz.

Q46. Dr. Youngs, to recapitulate, you calculated the angle of incidence of the earthquake waves for the frequencies for which peak cask response would be observed?

A46. (RY) Yes, the angle of incidence is generally less than 10 degrees off vertical for all frequencies in the 1-5 Hz range, the frequency range of peak cask response.

Q47. What else was done to evaluate the claims raised by the State in Section D.1.a of the Unified Contention?

A47. (RY, WT) Geomatrix evaluated the potential effects of the small departure from vertical of the angle of incidence of the earthquake waves impinging the PFSF site.

Q48. What effect would one expect and why?

A48. (WT) Because of the small departure of the angle of incidence from vertical and the small size of the pads (30 by 67 ft in plan dimensions), one would expect that this slight departure from vertical would cause only very minor effects on the pad response. The results of the Geomatrix evaluation confirm that the small departure in the angle of incidence from vertical causes negligible effects on the response motion of the storage pads.

Q49. Dr. Tseng, have you reviewed this evaluation done by Geomatrix?

A49. (WT) Yes.

Q50. And do you agree that that the effects of the small departure in the angle of incidence from vertical, as shown by Geomatrix, are negligible for the storage pads?

A50. (WT) Yes.

Q51. Please describe the evaluation done by Geomatrix of the potential effects of the small variance of the angle of incidence from vertical of the earthquake waves impinging the PFSF site.

A51. (RY) First, one can evaluate the potential effect of inclined waves on the storage pads by calculating the difference in arrival times at two adjacent points on the pads. The storage pads have a width of 30 ft. in the east-west direction, which is also the fault normal direction. The primary faults are oriented in an approximately north-south direction. Therefore, for nearby ruptures of the Stansbury fault, the strongest shaking will be due to earthquake waves arriving

from the east. Calculating the difference in the arrival times of earthquake waves at the east and west edges of the pads for the small angle of incidences determined by Geomatrix, one obtains differences in arrival times on the order of 0.001 to 0.002 seconds. These time differences would only affect motions in very high frequency, higher than about 50 to 100 Hz, which are far above the dominant frequency range of peak cask response of 1 to 5 Hz.

Q52. Please explain why a time difference in arrival of earthquake waves on the east and west edges of the pads on the order of 0.001 to 0.002 seconds would not be of significance.

A52. (RY, WT) A seismic wave generally requires a minimum of 10 equal time steps to define it. A time lag of the order of 0.001 to 0.002 seconds will start to affect a seismic wave having a period of 0.01 to 0.02 seconds. The inverse of the period of a wave is the frequency of the wave. Thus, the seismic waves that will be affected by a time lag of the order of 0.001 to 0.002 seconds will be those having their frequencies higher than 50 Hz (= 1/0.02 seconds) to 100 Hz (= 1/0.01 seconds). Such high-frequency waves are beyond the frequency range that are generally of interest for seismic design, which is normally below 50 Hz, and are far below the dominant frequency range of peak cask response of 1 to 5 Hz.

Q53. What else did Geomatrix do to evaluate the potential effects of the small departure of the angle of incidence from vertical of the earthquake waves impinging the PFSF site?

A53. (RY, WT) Geomatrix also evaluated the effects of low incident angle waves on the pad response using published work of Luco (1976) and Wong and Luco (1978).

Q54. Please describe the nature of this evaluation.

A54. (RY, WT) In the near field there are two major types of seismic waves that are responsible for strong ground shaking, compression waves (P-waves) and shear waves (S-waves). Compression waves represent push-pull motion in the direction of propagation and are analogous to sound waves in air. Shear waves represent side-to-side motion at right angles to the direction of wave propagation (shearing). This side-to-side motion occurs in two planes. Side-to-side motion in the

horizontal plane is denoted by SH-waves and side-to-side motion in the vertical plane is denoted by SV-waves.

When seismic waves strike a structure at an angle of incidence (from vertical) greater than 0 they can induce additional components of motion beyond horizontal and vertical translation (side-to-side and up-and-down motions). Inclined SH-waves tend to induce torsional motions (rotation about a vertical axis) and inclined P and SV waves tend to introduce rocking motions (rotation about a horizontal axis). The amount of this additional motion depends on the angle of incidence and the dimensions of the structure. Studies by Luco (1976) and Wong and Luco (1978) provide evaluations of the amount of this additional motion as a function of two dimensionless parameters. The first is the normalized frequency of the foundation and represents the ratio of the foundation dimension to the wave velocity in the underlying material. The second is the ratio of the wave velocity in the underlying material to the apparent wave-passage velocity and is equivalent to the sine of the angle of incidence.

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

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

Q55. What conclusions can be drawn from these various analyses of the potential effects of the small departure from vertical of the angle of incidence of the earthquake waves impinging the PFSF site?

A55. (RY, WT) These analyses show that the additional rocking and torsional motion of the pad caused by inclined incident waves at the PFSF would be small compared to the motion caused by the vertically propagating waves. The calculations presented by Holtec show that there are very large margins in the range of cask movements calculated for the design earthquakes. Any small additional motion induces by inclined waves would be insignificant compared to these margins.

Q56. How do the effects of non-vertically propagating waves at the PFSF site discussed above relate to the conservatisms embodied in the ASCE Standard 4-86?

A56. (WT) As discussed in the ASCE Standard 4-86, Section 3.3.1.7, there are various uncertainties in modeling and analysis of soil-structure interaction effects. The variation of soil properties from the best-estimate values to their lower-range and upper-range values is a means intended to account for many such uncertainties. A conservative variation of soil moduli by a factor of 1.5 to two for the lower and upper ranges was used for the PFSF which provides a way to account for uncertainties.

Q57. What conclusions do you draw based on your evaluation of the State's claims in Section D.1.a of the Unified Contention?

A57. (RY, WT) With the small angles of incidence (off vertical) of the seismic waves that may potentially occur at the site, and within the dominant frequency range of interest for the cask response, the effect of earthquake motions on structures and components at the PFSF may be represented by the use of vertically propagating earthquake waves, and the effect of non-vertically propagating waves alleged by the State is insignificant.

2. Pad Rigidity Claims Raised in Section D.1.b of Unified Contention

Q58. Please describe the claim raised by the State in Section D.1.b of the Unified Contention.

A58. (WT) In Section D.1.b of the Unified Contention, the State claims that calculations done by the Applicant incorrectly assume that the pads will behave rigidly during the design basis earthquake and that this assumption of rigidity leads (i) to "[s]ignificant underestimation of the dynamic loading atop the pads, especially in the vertical direction," and (ii) to "[o]verestimation of foundation damping."

Q59. What calculations is the State referring to in its claims raised in this Section of the Unified Contention?

A59. (WT) The State is referring to two calculations, the first performed by Stone & Webster of the stability of the storage pads and the second performed by Holtec of the stability of the casks on the storage pads. The Stone & Webster Calculation 05996.02-G(B)-04, Rev. 9, Stability Analyses of Cask Storage Pads (July 26, 2001) analyzes three potential failure modes for the pads, sliding, overturning, and bearing capacity failure. The Holtec calculation assesses the earthquake loads of the casks imposed on the pads as well as the stability of the Holtec casks under design basis earthquake loads. As described in Dr. Ostadan's deposition, the State's claims of pad flexibility affect the two calculations differently. See Ostadan Dep. at 82-84, 109-120.

Q60. Please describe the claims raised by the State with respect to the Holtec calculation?

A60. (WT) The claims concern Holtec's assumption that the concrete storage cask pads are rigid and the effect that this allegedly erroneous assumption has on the calculation of the soil spring and dash pots as related to foundation damping. See Ostadan Dep. at 109-115. The State claims that as a result of this erroneous assumption Holtec underestimates the loads on the pads and overestimates foundation damping. See Ostadan Dep. at 105-106, 112-113.

Q61. Please describe the claims raised with respect to the Stone & Webster calculation?

A61. (WT) The claims raised with respect to the Stone & Webster calculation concern Stone & Webster's assumption that the pad and the surrounding soil cement are rigid and the effect that this assumption has on the earthquake accelerations used

by Stone & Webster in its stability calculation. See Ostadan Dep. at 109-111, 116-120. According to the State, the assumption of pad rigidity results in Stone & Webster's use of the peak ground acceleration in its calculation of pad stability instead of the ground acceleration associated with the natural frequency of the casks-pads-soil system. This allegedly erroneous assumption leads to an underestimation of the earthquake loads used by Stone & Webster in its stability analyses. Id. at 119.

Q62. What is the essence of the State's claims with respect to the Holtec calculation?

A62. (WT) The essence of the State's claims is that Holtec should have modeled the concrete storage cask pad as being flexible in its stability calculations instead of analyzing the cask stability assuming the pads to be rigid.

Q63. What considerations generally determine whether a concrete foundation pad should be analyzed as being rigid or flexible?

A63. (WT) All structures are flexible to some degree. However, depending upon the specific purpose of an analysis, the degree of flexibility may or may not have a significant effect on the analysis' results.

Q64. The State claims that Holtec's assumption of pad rigidity in its cask stability calculations is contradicted by ICEC's calculation for the analysis and design of the storage pads in which ICES's analyses showed the pad to be flexible. Do you agree?

A64. (WT) No. The ICEC calculation was performed for the design of the reinforced concrete pad. Thus, in order to determine the internal stresses in the pad when subjected to applied cask loads, the pad flexibility was important and thus was included. The Holtec calculation was done for to a different purpose. The calculation was to evaluate the global response of the casks supported on the pad for which the effect of pad flexibility may depend on the frequency ranges of interest.

Q65. Have you evaluated the rigidity of the pad for frequency range of interest for the peak cask response for purposes of calculating foundation damping and related parameters.

A65. (WT) Yes. I have received a Stone & Webster evaluation of the effect of pad flexibility on foundation stiffness and damping based on published results of Iguchi and Luco (1981). Using the relevant parameter values for the pad and the foundation soil, this evaluation demonstrated that the effect of flexibility on the foundation stiffness and damping properties of the pad is insignificant in the frequency range of importance to the cask response. A copy of the calculation is included as PFS Exhibit MM. I have independently reviewed this calculation and agree with the conclusions it reached.

Q66. Please describe this evaluation and its basis.

A66. (WT) Using bending rigidity of the pad as designed and shear moduli of the soils supporting the pad, Stone & Webster evaluated the dimensionless rigidity ratio of the pad relative to soil as defined in the 1981 paper of Iguchi and Luco. Based on this dimensionless rigidity ratio and the dimensionless frequencies corresponding to the frequency range of cask response between 1 and 5 Hz, the effect of pad flexibility on the pad's vertical and rocking foundation impedance functions was determined from the published results in Iguchi and Luco's paper. These impedances for the flexible pad foundation were then compared with the corresponding impedances for the rigid pad foundation case to assess the amount of differences between them. The result of this comparison shows that the effect of pad flexibility causes very small deviations in the foundation impedances from the rigid pad foundation impedances within the frequency range of interest.

Q67. Is this paper by Iguchi and Luco a recognized work in this area?

A67. (WT) Yes it is. The paper was published in a peer-reviewed journal and the results published in this paper have also been used for validating numerical analysis results using a computer program such as SASSI.

Q68. How does this evaluation relate, if at all, to ICEC's treatment of the pads as flexible in its calculation for the analysis and design of the pads?

A68. (WT) ICEC's calculation was for the purpose of determining internal stresses in the pad induced by imposed dynamic loadings of the casks. For this purpose, the

pad flexibility was included. For the purpose of determining the dynamic response motions of the casks, the insignificant effect of pad flexibility on the foundation stiffness and damping properties implies that a rigid pad assumption is reasonable for the purpose of determining the global dynamic response motions of the casks.

Q69. Referring to ICEC's calculation, Table 5.2.5-1 at page 214 of the calculation, the State's expert, Dr. Ostadan, has claimed that your calculation "showed that the displacements [of the pad] varied by more than a factor of two and a half from one corner of the pad to the other" which clearly shows that the pad is not rigid.[1] Do you agree with this interpretation by Dr. Ostadan of your calculation?

A69. (WT) No, I do not agree with Dr. Ostadan's interpretation of the seismic loading condition. The ICEC calculation for which results were shown in Table 5.2.5-1 was performed by ICEC only for calibration purposes, to compare the results obtained using the CECSAP code to those that obtained using the SASSI code under a concentrated vertical load. The calculation was not intended to be representative of actual earthquake loadings on a pad. Thus, the displacements shown in Table 5.2.5-1 of the ICEC calculation are due to a vertical load applied to a single node of the finite element model of the pad. This node is near the corner of the pad. Under such a concentrated vertical load, vertical displacements will vary from node to node. That is to be expected. Under a more uniform loading, such as would take place under earthquake conditions, the variation off the vertical displacements of the pad would be less significant. The ICEC calculation includes one case of more uniform, 8 cask symmetric loading. The results for that case are presented in Table S-2 (page 229). For that case, the vertical displacements at all nodes are quite uniform.

Q70. Dr. Ostadan also refers to Table D-1(d) at page 234 of your calculation to support his contention that Holtec should have treated the pad as flexible. What does this table show?

[1] Declaration of Farhang Ostadan, January 30, 2001, paragraph 25.

A70. (WT) This Table shows the maximum displacements of the pad in the vertical direction as computed by ICEC at various nodes on the pad assuming two, four, and eight casks respectively are placed on the pad for the lower range, best estimate and upper range soil properties. It must be emphasized that these are maximum displacements observed at any point in time during the analysis and do not occur at a simultaneous response displacement in time. Further, the displacements in the Table are very small, being expressed in 1×10^{-3} ft. Thus, the largest displacements are on the order of 3/8 of an inch. These displacements, however, include displacements of the pad acting as a rigid body as well as any local deformations of the pad.

Q71. What do you mean when you say that the displacements set forth in your Table D-1(d) at page 234 include the displacement of the pad acting as a rigid body?

A71. (WT) When a rigid pad supported on soil is subjected to a vertical load, the pad will undergo vertical displacements without local deformations. This vertical displacement is included in the displacements on Table D-1(d) at page 234 cited by Dr. Ostadan.

Q72. Has ICEC determined the maximum local deformation or displacement of the pad for the cases set forth in Table D-1(d) at page 234?

A72. (WT) Yes. The maximum deviation of local displacement from the rigid body for the nine cases shown on Table D-1(d) is of the order of 0.01 ft, or approximately 1/8 of an inch.

Q73. Of what significance is this maximum local displacement?

A73. (WT) As stated, it depends on the purpose of the calculation. Insofar as determining internal stresses of the pad for the design of the pad, the local displacement should be included in order to capture the local maximum stresses in the pad. Insofar as determining the gross soil spring and soil damping properties for purpose of analyzing global response of the cask/pad/soil coupled system, this small local displacement would produce only secondary effects on the global dynamic response of the system.

Q74. On page 114 to 115 of his deposition, Dr. Ostadan claims that the force that ICEC calculated of the casks and the pad transferred to the soil shows an effective acceleration of less than 0.60 g, which he claims is too low given a peak ground acceleration of 0.71g. From this Dr. Ostadan concludes that the loads provided to ICEC by Holtec were not "adequate." Do you agree with Dr. Ostadan's claims?

A74. (WT) No. Since the casks on the pad are allowed to slide and/or tip with partial base up-lifting under earthquake loading, sliding and rocking of casks produce lower effective horizontal inertial load as compared to the case of casks being rigidly attached to the pad.

Q75. What are your conclusions with respect to the claims raised by the State in Section D.1.b of the Unified Contention with respect to pad rigidity?

A75. (WT) Based on the previously discussed evaluation performed by Bruce Ebbeson, the effect of pad flexibility on the pad's foundation soil stiffness and damping is small.

3. Claims Raised in Section D.1.c of Unified Contention – Evaluation of Potential Storage Pad Motion in Relation to Sliding of the Casks on the Pads

Q76. Please describe the claim raised by the State in Section D.1.c of the Unified Contention.

A76. (WT) The State claims in D.1.c of the Unified Contention 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 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.

Q77. On which portions of this claim are you testifying?

A77. (WT) I will be testifying with respect to (i) the effect of soil-cement around the pads once the pads undergo sliding motion and (ii) the flexibility of the pads under DBE loading.

Q78. What do you understand to be the nature of the State's claims regarding the effect of the soil cement around the pads once the pads undergo sliding motion?

A78. (WT) I understand that the State takes issue with a calculation performed by Holtec to show the effect on cask stability of having the storage pads undergo sliding. The calculation is described in an August 6, 2001 Holtec letter which PFS forwarded to the NRC on August 7, 2001.

The State claims that Holtec's calculation is overly simplistic and incorrect because it has "ignored the effect of soil-cement around the pad and the unsymmetric loading that the soil-cement will impart on the pad once the pad undergoes sliding movement." According to the State, "[t]he cement-treated soil will create an active and a passive side" and the "cracking and potential crushing of the soil-cement on the passive side and separation of the soil-cement on the active side due to lack of tensile capacity of soil-cement will impart unbalanced forces on the pad and severely impact the stability of the casks on the pads." State of Utah's Response to Applicant's Eighth Set of Discovery Requests, Response to Interrogatory No. 6.

Q79. What is your view of the State's assertion?

A79. (WT) Under PFS's 2000 year design basis earthquake, the pads have a minimum safety factor of 1.27 against sliding and thus would not be expected to slide on top of the soil underneath the pads. The sliding parametric study undertaken by Holtec was not a design basis calculation, but was intended to show the general effect that sliding of the pads would have on cask movement in the event such sliding were to occur.

The calculation demonstrates that a reduction in movement of the casks can be expected to occur should the pads undergo sliding. Sliding of the pads would reduce the loads on the casks and would be beneficial, not detrimental, to the stability of the casks. The soil cement around the pads will contribute to resisting sliding of the pad on the soil and will limit the amount of sliding if sliding were to occur.

Q80. On what do you base your opinion that the loads imparted by the soil cement would have only a secondary order effect on the stability of the casks and would not affect the validity of Holtec's calculation?

A80. (WT) The pad is surrounded by and embedded into the side soil only up to thickness of the pad which is 3 ft. Such a shallow side soil embedment contributes very little to the pad's foundation soil impedances. Thus, during a seismic event, the majority of the soil resistance to pad's motion is from the resistance of soil underneath the pad and only a relatively very small amount of resistance will be contributed by the side soil. Furthermore, since the pad stability analyses under the design basis earthquake have demonstrated that the friction or shear resistance of the soil beneath the pad alone is sufficient to resist the seismic shear load imposed on the pad, the movement of the pad relative to soil will be limited to elastic deformation of soil which is small.

Q81. What do you understand to be the nature of the State's claim in Section D.1.c(ii) that the Applicant ignores the "the flexibility of the pads under DBE loading" in evaluating the motion of the casks once the pads undergo sliding?

A81. From the deposition testimony of the State's expert witness, Dr. Ostadan,[2] I understand that this is the same claim as raised by the State in Section D.1.b of the Unified Contention which I have already addressed above.

4. Claims Raised in Section D.1.d of the Unified Contention – Lateral Variations in the Phase of the Ground Motions

Q82. Please describe the claim raised by the State in D.1.d of the Unified Contention.

A82. (RY, WT) In Section D.1.d of the Unified Contention, the State claims that the "Applicant has failed to consider lateral variations in the phase of ground motions and their effect on the stability of the pads and casks."

Q83. What is your understanding of this claim?

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[2] Ostadan Dep. at 163-64, 172.

A83. (RY, WT) We understand from Dr. Ostadan's deposition that this claim is subsumed within the State's claims raised in Section D.1.a of the Unified Contention,[3] which we have discussed at length above.

5. Claims Raised in Section D.1.e of the Unified Contention – Frequency Dependency of Soil Spring and Damping Values

Q84. Please describe the claims raised by the State in D.1.e of the Unified Contention.

A84. (WT) In Section D.1.e of the Unified Contention, the State claims 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."

Q85. What is the nature of the issue raised by the State in this claim?

A85. (WT) According to the State, Holtec inappropriately used constant numbers for the spring and damping values of the foundation soils that did not take into account the frequency dependency of these parameters. The State claims that Holtec similarly should have picked a value for soil spring and damper that corresponds to the natural frequency of the soil foundation system.

Q86. Do you agree with the State's claims?

A86. (WT) No. Based on my understanding of how the soil spring, mass, and damping coefficient values were developed and incorporated into Holtec's calculation, as described below, I do not agree that the frequency dependency effect was improperly ignored..

Q87. Why not?

A87. (RY, WT) The foundation soil springs, masses, and dampers used by Holtec were developed by Geomatrix in such a manner that they took into account the frequency-dependency of the soil foundation system.

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[3] Ostadan Dep. at 178-79.

Q88. How did Geomatrix develop the springs, mass, and damping values for the foundation soils so as to take into account the frequency-dependency of the foundation soil system?

A88. (RY, WT) The impedance functions developed by Geomatrix in Calculation No. 05996.02-G(PO18)-2) (2001), "Soil and Foundation Parameters for Dynamic Soil-Structure Interaction Analyses, 2000-year Return Period Design Ground Motions," and used by Holtec in nonlinear analyses of the cask/pad/soil interaction include soil springs, dashpots, and virtual (effective) soil masses. Different sets of these parameters for each mode of vibration (i.e., horizontal, vertical, and rocking) were developed based on formulations by Newmark and Rosenblueth in *Fundamentals of Earthquake Engineering*, Prentice Hall, Inc. (1971). Newmark and Roseblueth's treatise shows that use of spring and dashpot constants together with virtual (effective) soil masses for each mode of vibration results in excellent prediction of response of circular plates on soil throughout most of the range of excitation frequencies when compared with available "exact" solutions. Therefore, the foundation parameters (spring and dashpot constants plus virtual soil masses) used by Holtec account for the frequency dependence of the foundation impedance functions. Use of virtual soil mass as one of the foundation parameters in addition to the spring and dashpot constants is equivalent to use of frequency-dependent impedance functions in the frequency domain solution, as described below.

The frequency-dependent impedance functions of a foundation are generally defined as follows:

$$K_{ij}(\omega) = k_{ij}(\omega) + i\omega c_{ij}(\omega)\,(i, j = 1,6) \qquad (1)$$

where k_{ij} is the real part of the impedance, ωc_{ij} is the imaginary part, and ω is circular frequency. When the virtual soil mass is used in the impedance functions together with the static soil spring stiffness, the real parts of the impedance functions, k_{ij}, become frequency-dependent as:

$$k_{ij}(\omega)=(k_0)_{ij}-\omega^2 m_{ij} \qquad (2)$$

where $(k_0)_{ij}$ is the static stiffness and m_{ij} is the virtual soil mass (as defined above).

Thus, the real parts of the impedance functions expressed equation (1) are frequency-dependent when the virtual soil mass is used along with soil spring stiffnesses.

Q89. What is your conclusion regarding the State's claims in Section D.1(e) of the Unified Contention?

A89. (WT) Since soil masses were used along with soil springs and dash-pots, the resulting foundation impedance functions used by Holtec as represented by the constant soil springs, masses, and dash-pots are a good approximation of the soil foundation impedances for the fundamental frequency of the soil foundation system for each of the six rigid-pad motion degrees of freedom.

6. Claims Raised in Section D.1.h of the Unified Contention – Use of One Set of Time Histories

Q90. Please describe the claims raised by the State in D.1.h of the Unified Contention that you will be addressing.

A90. (RY) I will be addressing the claim in Section D.1.h (ii) of the Unified Contention in which the State claims that the use of one set of time histories in Holtec's (nonlinear) cask stability analysis is inadequate because (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 will address how we incorporated the effects of fault fling in the development of the set of time histories used for the PFSF.

Q91. Are you familiar with the term "fault fling?"

A91. (RY) Yes.

Q92. Please describe what this term means.

A92. (RY) Fault fling is a term generically used to describe enhanced ground motions that have been observed in a number of earthquake recordings obtained very near to the causative fault rupture. A specific model that has been developed to quantify these near-fault effects is a model for what is called forward directivity. As an earthquake ruptures towards a site, the rupture moves at a speed that is near to that of the seismic waves radiating from the fault plane. Consequently, the seismic waves build up into a coherent, strong velocity pulse that arrives in the early portion of the strong shaking. In addition, there are recognizable trends in the amplitudes of ground motions that depend on the orientation of the recording location relative to the fault. Specifically, low frequency motions in the direction perpendicular to the fault (fault-normal) are, on average, greater than those in the direction parallel to the fault rupture (fault-parallel).

Q93. Did you account for these near-fault effects in the set of time histories that you developed for the PFSF?

A93. (RY) Yes.

Q94. How did you go about including these effects in the time histories for the PFSF?

A94. (RY) The first step was to account for forward directivity in the design response spectra. The model developed by Somerville and others (1997) was used to enhance all three components of the design response spectra for forward directivity effects. The east-west horizontal spectrum was then increased for fault-normal effects and the north-south component was reduced for fault parallel effects. The second step was to select a starting time history that exhibited a velocity pulse in the early portion of strong shaking. The Sturno recording of the Irpinia earthquake has large amplitude – low frequency (~0.5 Hz) motions that begin approximately 4 seconds after the start of the record. The recordings were then scaled upward until their response spectra enveloped the design response spectra.

Q95. Were conservatisms with respect to near-fault effects incorporated in the set of time histories that you developed for the PFSF design?

A95. (RY) Yes.

Q96. What are these conservatisms?

A96. (RY) The design response spectra are based on a probabilistic analysis which allows for a range of possible earthquake locations and rupture geometries. However, the near-fault effects (forward directivity and fault-normal effects) were applied using a deterministic worst-case rupture geometry that maximized their effects. The time histories were then scaled so that they envelop the design response spectra over a very broad frequency range. As a result, the response spectra for the time histories are on average five percent larger than the design response spectra.

Q97. How did you go about using a deterministic approach in determining near fault effects and why was it conservative?

A97. (RY) The near-fault effects are a function of the location of rupture initiation. I assumed the worst possible location for rupture initiation instead of randomizing the location over a distribution of possible initiation points.

Q98. How do the conservatisms embodied in the time histories developed for the PFSF compare to the conservatisms in time histories that you have either developed or are aware of for use in the design of nuclear power plant structures?

A98. (RY) In terms of enveloping the design response spectra by spectrum-compatible time histories, I would expect that the conservatism in the PFSF time histories is at least comparable to that in time histories developed for other nuclear power plants. I am unaware of any time histories for nuclear power plant design that include near-fault effects as ours do. (I understand that near-fault effects are being incorporated into the design ground motions for interim storage facilities at Diablo Canyon. However, I do not know if the near fault effects are being estimated probabilistically, or in a worst-case deterministic manner, as we have done.)

Q99. Does this conclude your testimony?

A99. (RY, WT) Yes, it does.

ROBERT R. YOUNGS
PRINCIPAL ENGINEER

EDUCATION

University of California: Ph.D., Geotechnical Engineering, 1982

University of California: M.S., Geotechnical Engineering, 1973

California State Polytechnical University, Pomona: B.S., Civil Engineering, 1969

REGISTRATION

Geotechnical Engineer, California No. 924, 1987
Civil Engineer, California No. 22519, 1973

AFFILIATIONS

American Society of Civil Engineers
American Geophysical Union
Earthquake Engineering Research Institute
Seismological Society of America
Society for Risk Assessment

SKILLS AND EXPERIENCE

Dr. Youngs has 24 years of consulting experience, with primary emphasis in hazard analysis. He has pioneered approaches for incorporating earth sciences data, and their associated uncertainties, into probabilistic hazard analyses; The work has focused on developing quantitative evaluations of hazard by combining statistical data and expert judgment. Within Geomatrix's Decision Analysis (DA) operating unit, Dr. Youngs has helped develop capabilities that integrate the fields of earth sciences, hazard analysis, and risk assessment. Representative project experience includes:

Regional Seismic Hazard Mapping/Microzonation Studies: Ech Cheliff Region, Algeria; San Juan Province, Argentina, PG&E; Mendoza Province, Argentina; Seismic Design Mapping Project, State of Oregon, Oregon Department of Transportation

Seismic Source/Ground Motion Characterization for Hazard Analysis: Diablo Canyon Power Plant, PG&E; WNP-2 Hanford Power Plant, WPPSS; Hanford Reservation, Westinghouse Hanford Co.; Palo Verde Nuclear Generating Station, Arizona Power; Yucca Mountain Nuclear Waste Repository Site, U.S. Department of Energy

Development of Hazard Methodologies/Uncertainty Treatment: Seismic Hazard in the Eastern United States, Electric Power Research Institute (EPRI); Maximum Earthquakes in Eastern United States, EPRI; Expert Elicitation Methodology Demonstration for Yucca Mountain Performance Assessment, EPRI; Characterization of seismic hazard in Southern Ontario, Atomic Energy Control Board, Canada

Hazard Analyses for Performance Assessment of Built Structures: seismic hazard at San Francisco-Bay Area bridges, California Department of Transportation (CDOT); seismic hazard at Humboldt Bay bridges, CDOT ; seismic hazard and site response studies for K-reactor, Westinghouse Savannah River Co.; seismic hazard analysis for operating nuclear power plants in Spain, Westinghouse Energy Systems Europe; seismic hazard analysis and development of earthquake ground motions for Blue River Dam, Oregon, USACOE.

Hazard Analyses for Development of Design Criteria: Seismic hazard assessment for the New Production Reactor at Savannah River Site and Idaho National Engineering Laboratory (DOE); WNP-1, 2,4 Hanford and WNP-3, 5 Satsop, WPPSS; Potential High-Level Radioactive Waste Repository Site, Yucca Mountain, DOE; Waste Tank Sites at Hanford, Washington, Westinghouse Hanford Co.

Performance Assessment of Natural Systems: Demonstration of risk-based total system performance assessment, EPRI, DOE; Earthquakes/tectonics expert elicitation project, EPRI; Probabilistic volcanic hazard analysis, Yucca Mountain, TRW and DOE; Fault displacement hazard analysis for Yucca Mountain, USGS, DOE

PUBLICATIONS

"Strong ground motion attenuation relationships for subduction zone earthquakes." Youngs, R.R., Chiou, S.J., Silva, W., and Humphrey, J.: Seismological Research Letters, v. 68, n. 1. January/February 1997.

"Seismic hazard mapping for highway design in the state of Oregon." Youngs, R.R.: Proceedings, Design of Highway Bridges for Extreme Events, Federal Highway Administration, Atlanta, Georgia. December 1996.

"Regional probabilistic seismic hazard mapping with uncertainty: An example from the state of Oregon, USA." Youngs, R.R., Coppersmith, K.J., Hanson, K., DiSilvestro, L., and Wells, D.: Fifth International Conference on Seismic Zonation, Nice, France. October 17-18, 1995.

"Earthquake ground shaking hazard in Utah." Proceedings, Earthquake Engineering Research Institute Wasatch Front Seismic Risk Regional Seminar, v. 1, Salt Lake City, Utah. November 29-30, 1994.

"Magnitude dependent variance of peak ground acceleration." Youngs, R.R., Abrahamson, N., Makdisi, F., and Sadigh, K.: Bulletin, Seismological Society of America, accepted for publication. 1994.

"Computer applications in geotechnical earthquake engineering." Chang, C.-Y., and others: Geotechnical News, v. 12, n. 2, p. 36-38. June 1994.

"Specification of ground motions and response spectra for seismic evaluation of nuclear power plants." Youngs, R.R.: Proceedings, Fourth Symposium on Current Issues Related to Nuclear Power Plant Structures, Equipment, and Piping, Orlando, Florida. December 1993.

"Assessing fault rupture hazard for the proposed repository at Yucca Mountain, Nevada: Demonstration of a methodology using expert judgments." Perman, R.C., Coppersmith, K.J., Youngs, R.R., and Shaw, R.: Proceedings, Fourth Annual International Conference on High Level Radioactive Waste Management, v. 1, p. 2086-2091. 1993.

"Preliminary assessment of fault rupture hazard at the Yucca Mountain site based on expert judgment." Coppersmith, K.J., Youngs, R.R., Perman, R., and Shaw, R.: Proceedings, Fourth Annual International Conference on High Level Radioactive Waste Management, v. 1, p. 6-13. 1993.

"A comprehensive seismic hazard model for the San Francisco bay region." Youngs, R.R., Coppersmith, K.J., Taylor, C., Power, M.S., Di Silvestro, L., Angell, M., Hall, T., Wesling, J., Mualchin, L.: Proceedings, Second Conference on Earthquake Hazards in the Eastern San Francisco Bay Area, California Division of Mines and Geology Special Publication 113, p. 431-441. 1992.

"A stable algorithm for regression analyses using the random effects model." Abrahamson, N.A., and Youngs, R.R.,: Bulletin, Seismological Society of America, v. 82, n.1, p. 505-510. 1992.

PUBLICATIONS (continued)

"Modeling fault rupture hazard for the proposed repository at Yucca Mountain, Nevada." Coppersmith, K.J., Youngs, R.R.: Proceedings, 1992 International High Level Radioactive Waste Management Conference, v. 1, p. 1142-1150. 1992.

"Site specific ground motion assessment for K-Reactor, Savannah River Site." Coppersmith, K.J., and others: Proceedings, Third Department of Energy Natural Phenomena Hazards Mitigation Conference, p. 184-194. 1991.

"Assessment of liquefaction potential in the San Jose, California urban area." Power, M.S., Perman, R., Wesling, J., Youngs, R.R., and Shimamoto, M.: Proceedings, Fourth International Conference on Seismic Micro Zonation, Stanford, California, v. II, p. 677-625. 1991.

"Seismic microzonation of the Ech Cheliff region, Algeria." Power, M.S., and others: Proceedings, Fourth International Conference on Seismic Micro Zonation, invited case study paper, Stanford, California, v. I, p. 539-588. 1991.

"Improved methods for seismic hazard analysis in the western United States." Coppersmith, K.J.: Proceedings, Fourth U.S. National Conference on Earthquake Engineering, v. 1, p. 723-731. 1990.

"Probabilistic seismic hazard analysis using expert opinion: An example from the Pacific Northwest." Coppersmith, K.J., and Youngs, R.R.: Geological Society of America Memoir on Neotectonics in Earthquake Evaluation: The Geological Society of America, v. 8, p. 27-46, Boulder, Colorado. 1990.

"The impact of fault segmentation on estimates of earthquake recurrence and seismic hazard." Youngs, R.R., and Coppersmith, K.J., Proceedings, Fourth International Conference on Seismicity and Seismic Risk, Bechyne Castle, Czechoslovakia, September 4-9, v. II, p. 440-446. 1989.

"Estimating maximum earthquakes for seismic sources in the central and eastern United States: A progress report." Coppersmith, K.J., Youngs, R.R., Johnston, A.C., Kanter, L., Schneider, J., and Arabasz, W.: Proceedings, Fourth International Symposium on Seismicity and Seismic Risk, Bechyne Castle, Czechoslovakia, September 4-9, v. I, p. 115-122. 1989.

"Keeping pace with science: Seismic hazard analysis in the western United States." Youngs, R.R., and Coppersmith, K.J.: Proceedings, Second Department of Energy Natural Phenomena Hazards Mitigation Conference, p. 262-270. October 1989.

"Keeping pace with science: seismic hazard analysis in the central and eastern United States." Coppersmith, K.J., and Youngs, R.R.: Proceedings, Second Department of Energy Natural Phenomena Hazards Mitigation Conference, p. 252-261. October 1989.

PUBLICATIONS (continued)

"Issues regarding earthquake source characterization and seismic hazard analysis within passive margins and stable continental interiors." Coppersmith, K.J. , Youngs, R.R.: Earthquakes at North-Atlantic Passive Margins–Neotectonics and Postglacial Rebound (Gregersen, S. and Basham, P., eds.), NATO ASI Series C, v. 266, p. 601 – 631. 1989.

"Use of detailed geologic data in regional probabilistic seismic hazard analysis: An example from the Wasatch Front, Utah." Youngs, R.R., Swan, F.H., and Power, M.S.: Proceedings, Earthquake Engineering and Soil Dynamics II ASCE, Park City, Utah, p. 156-172. June 27-30.

"Nearfield ground motions for large subduction zone earthquakes." Youngs, R.R., Day, S.M., and Stevens, J.L.: Proceedings, American Society of Civil Engineers–Specialty Conference on Earthquake Engineering and Soil Dynamics II, Park City, Utah, p. 445-462. 1988.

"Probabilistic analysis of earthquake ground shaking hazard along the Wasatch Front, Utah." Youngs, R.R., Swan, F.H., Power, M.S., Schwartz, D., and Green, R.: United States Geological Survey–Professional Paper on Seismic Hazards in Utah (in press). Preprinted and Assessment of Regional Earthquake Hazards and Risk along the Wasatch Front, Utah United States Geological Survey Open File Report 87-585, v. 2, p. M1-110.

"Geotechnical data in seismic risk evaluations." Arango, I.: Proceedings, Eighth Pan American Congress for Soil Mechanics and Foundation Engineering p. 495-506. August 1987.

"Probabilistic assessment of seismic hazards in the Ech Cheliff Region, Algeria and seismic microzonation of urban areas in the Ech Cheliff Region, Algeria." Swan, F.H., and others: Proceedings, Eighth European Conference on Earthquake Engineering, Lisbon, Portugal. September 7-12, 1986.

"Seismic hazard methodology for the central and eastern United States, Volume 1: Methodology." with Risk Engineering, Woodward-Clyde Consultants, and Cygna Corporation. Electric Power Research Institute Publication NP-4726. 1986.

"Capturing uncertainty in probabilistic seismic hazard assessments within intraplate environments." Coppersmith, K., Youngs, R.R.: Proceedings, Third National Conference on Earthquake Engineering, Charleston, South Carolina, v. 1, p. 301-312. August 24-28, 1986.

"Seismic hazard assessment of the Hanford region, eastern Washington State." Coppersmith, K.J., and others: Proceedings, Department Of Energy Natural Phenomena Hazards Mitigation Conference, p. 169-176. October 1985.

"Implications of fault slip rates and earthquake recurrence models to probabilistic seismic hazard estimates." Youngs, R.R., Coppersmith, K.J.: Bulletin, Seismological Society of America v. 75, p. 939-964. 1985.

"Geotechnical features of Fur Seal Island design." Luscher, U., and others: Proceedings, American Society of Civil Engineers Conference on Civil Engineering in the Arctic Offshore, San Francisco. March 25-27, 1985.

PUBLICATIONS (continued)

"Assessment of confidence intervals for results of seismic hazard analysis." Kulkarni, R., Youngs, R.R., and Coppersmith, K.J.: Proceedings, Eighth World Conference on Earthquake Engineering v. 1, p. 263-270. 1984.

"Incorporation of geologic information and associated uncertainty in seismic hazard analysis." Invited paper presented at Specialty Seminar on Fundamentals of Probabilistic Risk Assessment, Stanford University, Stanford, California, July 19, 1984, and published in Earthquake Engineering Research Institute Publication No. 84-06, v. 11, p. 38-58.

"Incorporation of uncertainties in probabilistic seismic exposure analyses effects on completed seismic exposure." Sadigh, K: Invited paper presented at 78th Annual Meeting, Seismological Society of America, Earthquake Notes, v. 54, n. 1, p. 23. 1983.

"Peak horizontal and vertical accelerations, velocities and displacements on deep soil sites during moderately strong earthquakes." Sadigh, K., and Power, M.S.: Proceedings, Second International Conference on Microzonation, San Francisco, California, v. II, p. 801-811. 1978.

"Drainage effects on seismic stability of rockfill dams." Sadigh, K., and Idriss, I. M.: Proceedings, American Society of Civil Engineers Specialty Conference on Earthquake Engineering and Soil Dynamics, Pasadena, California. 1978.

EDUCATION

Ph.D. - Civil Engineering, University of California, Berkeley, 1971
M.S. - Civil Engineering, University of California, Berkeley, 1968
B.S. - Civil Engineering, National Taiwan University, 1964

EMPLOYMENT HISTORY

Professional Experience

1990-present	- President, International Civil Engineering Consultants, Inc., Berkeley
1987-90	- Principal Engineer and Assistant Chief Civil/Structural Engineer, Bechtel Corporation, San Francisco
1985-87	- Principal Engineer and Head of Special Structures Group, Bechtel Corporation, San Francisco
1977-85	- Engineering Group Supervisor, Special Structures Group, Bechtel Power Corporation, San Francisco
1976-77	- Engineering Specialist, Offshore Development Engineering, Inc., Berkeley
1973-76	- Senior Engineer, Bechtel Power Corporation, San Francisco

Academic Experience

1971-73	- Post-Doctoral Research Engineer, Earthquake Engineering Research Center, University of California, Berkeley

PROFESSION REGISTRATION

Civil Engineer, State of California

PROFESSIONAL SOCIETIES

American Society of Civil Engineers, Member
Earthquake Engineering Research Institute, Member
American Society of Mechanical Engineers, Technical Committee Member

AWARDS AND HONORS

1990	Bechtel Outstanding Technical Paper Award
1989	Bechtel Outstanding Technical Paper Award
1988	Bechtel Outstanding Technical Paper Award
1970	William H. and Helena I. S. Popert Research Fellowship

PUBLICATIONS

Over 60 technical papers, over 150 technical and project reports, 14 engineering computer programs.

SUMMARY OF EXPERIENCE

Dr. Tseng has more than 29 years of professional experience. He received his Ph.D. from the University of California, Berkeley (UCB) in 1971 having specialized in structural engineering and structural mechanics. He then joined the UCB Earthquake Engineering Research Center (EERC) as a post-doctoral research engineer. During his 2-1/2 years in EERC he made major contributions to advancing the state-of-the-art of seismic design and analysis of bridge structures, including the development of computer programs BSAP for linear analysis and YIELD and NEABS for nonlinear analyses. These programs with subsequent enhancements and modifications are now being used widely by bridge designers.

In addition to his research in the early 1970's, Tseng also actively participated in the seismic design and analysis of bridges, including the long-spanned Parrott Ferry Bridge in California and the cable-stayed Penang Bridge in Malaysia. He also performed seismic analyses for several offshore platforms off the coasts of California, Alaska, and Mexico.

In 1973, Tseng joined Bechtel of San Francisco where he served 16 years before leaving his position as Principal and Assistant Chief Civil/Structural Engineer in March 1990 to join with Dr. Joseph Penzien in forming ICEC. During the last 12 years at Bechtel, he headed the Special Structures group performing research and development and providing technical consulting services to many nuclear power projects, including the Susquehanna, Limerick, Pilgrim II, Hope Creek, Skagit, Trojan, Tsuruga II, Sequoyah, Browns Ferry, Watts Bar, Bellefonte, and Diablo Canyon nuclear power plant projects. During the past 10 years, he played a lead role in evaluating the following engineered facilities:

(1) Diablo Canyon Nuclear Power Plant, developing plans and methodologies to assess soil-structure interaction, to evaluate structural response due to spatial incoherence of seismic ground motions, and to evaluate nonlinear base uplift response for the Long-Term Seismic Program, and assessing the performance of concrete masonry walls,

(2) Sequoyah, Browns Ferry, and Watts Bar Nuclear Power Plants, conducting seismic response analyses and performance evaluations of seismic Category-I structures for Tennessee Valley Authority (TVA),

(3) Diablo Canyon and Watts Bar Nuclear Power Plants, developing methodologies and computer programs for evaluating seismic response of equipment and systems supported on floors and platforms, including equipment-structure interaction effects,

(4) Nuclear Power Plant Containment Building Model (1/4-scale), Lotung, Taiwan, conducting soil-structure interaction analyses and correlating results with field-test data under the joint TPC/EPRI program and developing soil-structure interaction analysis guidelines for industry applications under EPRI sponsorship,

(5) Advanced Boiling-Water Reactor, performing seismic response analyses and providing SASSI technology transfer to General Electric Nuclear Energy System,

(6) Field-Test Structural Model, Hualien, Taiwan, developing conceptual designs and evaluating their expected seismic performance under EPRI sponsorship,

(7) Bellefonte Nuclear Power Plant, conducting seismic analyses of all seismic Category-I structures using the current state-of-the-art seismic modelling and analysis techniques to regenerate seismic loads and floor response spectra for seismic performance evaluations for TVA,

(8) Underground gas transmission pipelines, performing engineering evaluations of the structural fitness-for-service conditions of pipelines 57A and 57B under severe ground settlements at levee crossings for Pacific Gas & Electric Company (PG&E),

(9) Benicia-Martinez Bridge, performing seismic soil-structure interaction analyses for the deep caisson foundation systems of the bridge to develop the foundation impedances and scattered seismic input motions for super-structure seismic vulnerability evaluation,

(10) Mokelumne Aqueduct Seismic Upgrade Project, performing seismic response analyses for determining the seismic demands on the aqueduct system for the East Bay Municipal Utility District (EBMUD),

(11) Lafayette Reservoir intake/outlet tower, performing seismic response analyses, evaluating the structural capacity, and providing recommendations for seismic retrofit for EBMUD,

(12) Department of Energy Savannah River Facilities, as a member of the Peer Review Panel for soil-structure interaction performing a technical review of seismic SSI analyses conducted for the high-level waste underground storage tanks,

(13) Richmond-San Rafael Bridge, generating multiple-support seismic motion inputs and performing seismic soil-structure interaction analyses to develop the foundation impedances and scattered foundation input motions for super-structural seismic vulnerability evaluation,

(14) San Mateo-Hayward Bridge, conducting free-field site response analyses to determine the strain-compatible soil properties and associated free-field site soil response motions, developing the pile-group foundation stiffness matrices at the pilecaps for as-built and retrofitted piers, evaluating the effect of soil-pile kinematic interaction (foundation scattering) on seismic response motions at the pilecaps for two-bell piers,

(15) Bronx-Whitestone Bridge, New York, developing four sets of three-component rock motion time histories compatible with target response spectra and target coherency functions, developing foundation impedances and seismic scattered foundation input motions at four supports of the main-suspended spans of the bridge for use in seismic response analyses of bridge structural system,

(16) San Francisco-Oakland Bay Bridge, East Span Replacement Seismic Safety Project, performing independent check of the main-span cable-stayed and suspension bridge design options including assessing soil-structure interaction effects of the main-span tower foundation systems,

(17) Taiwan Power Company, Nuclear Power Plant No. 4, Lungmen Nuclear Advanced BWR Units 1 & 2 in Northern Taiwan, performing seismic analyses and developing seismic design forces and displacements to the detailed designer for all seismic Category-I nuclear-island structures and major systems, including the Reactor Buildings, Control Buildings, and Auxiliary Fuel Buildings, and

(18) Taiwan High Speed Rail Project, performing a two-phase study, in cooperation with CTCI Corporation in Taiwan, on assessing the HSR-train-operation-induced ground vibration characteristics and amplitudes in Tainan Science-Based Industrial Park, where vibration-sensitive high-tech manufacturing facilities are located, and on developing ground-vibration mitigation measures for implementation to the Taiwan HSR civil/structural works.

Currently, as a principal in ICEC, Tseng is actively engaged in projects similar to those described above and is expanding his activities to other specialty areas as well. He currently serves as a consultant to (1) Bechtel Infrastructure Corporation, (2) Pacific Gas and Electric Company on the Diablo Canyon Power Plant seismic related work, (3) Tennessee Valley Authority (TVA) on Watts Bar and Bellefonte Nuclear Plants seismic related issues, (4) Electric Power Research Institute (EPRI) on Hualien, Taiwan soil-structure interaction experimental program and on seismic instrumentation for nuclear power plants, and (5) GE Nuclear Energy on Taiwan Power Company, Nuclear Power Plant No. 4, Lungmen Nuclear Units 1 & 2 seismic design and analysis related work.

Recent Technical Papers **W. S. Tseng**

1. "Soil-Structure Interaction Analysis Guidelines Based on Lotung Experiment in Response to the Revised Standard Review Plan," (with A. H. Hadjian, Y. K. Tang, and H. T. Tang), Paper No. IX/3, *Proc.*, Third Symposium on "Current Issue Related to Nuclear Power Plant Structures, Equipment, and Piping," Orlando, Florida, December 5-7, 1990.

2. "The Learning from the Large-Scale Lotung Soil-Structure Interaction Experiment," (with A. H. Hadjian, et al.), *Proc.*, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Vol. III, St. Louis, Missouri, March 11-15, 1991.

3. "Seismic Performance Investigation of the Hayward-BART Elevated Section Instrumented Under CSMIP," Paper No. 9, SMIP91 Seminar on Seismological and Engineering Implications of Recent Strong-Motion Data, Sacramento, California, May 30, 1991.

4. "Parametric Evaluation of Intermediate SSI Solutions on Final Response," (with F. Ostadan, et al.), Paper No. K04/1, *Proc.*, 11th Structural Mechanics in Reactor Technology (SMiRT-11), Tokyo, Japan, August 18-23, 1991.

5. "Assessment of Soil-Structure Interaction Practice Based on Synthesized Results from Lotung Experiment--Earthquake Response," (with A. H. Hadjian, et al.), Paper No. K08/7, *Proc.*, 11th Structural Mechanics in Reactor Technology (SMiRT-11), Tokyo, Japan, August 18-23, 1991.

6. "Post-Prediction Analysis and Parametric Studies for the Lotung Soil-Structure Interaction Experiment," (with Kiat Lilhanand, Y. K. Tang, and H. T. Tang), Paper No. K04/3, *Proc.*, 11th Structural Mechanics in Reactor Technology (SMiRT-11), Tokyo, Japan, August 18-23, 1991.

7. "Development of Power Spectral Density Functions Consistent with Design Response Spectra," (with Kiat Lilhanand), Paper No. K01/5, *Proc.*, 11th Structural Mechanics in Reactor Technology (SMiRT-11), Tokyo, Japan, August 18-23, 1991.

8. "Soil-Structure Interaction Analysis Incorporating Three-Dimensional Spatial Incoherency of Ground Motions," (with K. Lilhanand and D. Hamasaki), *Proc.*, 12th International Conference on Structural Mechanics in Reactor Technology (SMiRT-12), Stuttgart, Germany, August 1993.

9. "Seismic Response Analysis of Nuclear Power Plant Structures Considering Spatial Incoherency of Ground Motions," (with K. Lilhanand, D. Hamasaki, H. T. Tang, and Y. B. Tsai), *Proc.*, 4th International Topical Meeting on Nuclear Thermal Hydraulics, Operations and Safety, Taipei, Taiwan.

10. "Development of Multiple-Support Ground Motions for Seismic Vulnerability Evaluations of Major Bridges in Northern California," (with K. Lilhanand, N. A. Abrahamson, and C.-Y. Chang) Proc., 5th U.S. National Conference on Earthquake Engineering, Chicago, July 10-14, 1994.

11. "Seismic Evaluation of Benicia-Martinez Bridge," (with W. D. Liu, K. Lilhanand, C.-Y. Chang, R. A. Imbsen, and F. Li), Proc., 5th U.S. National Conference on Earthquake Engineering, Chicago, July 10-14, 1994.

12. "Cable-Stayed Bridge for High Speed Rail in Taiwan," (with Jeder Hsieh and Jiri Strasky), *Proc.*, International Association of Bridge Structural Engineering (IBASE) Conference, Deauville, France, October 12-15, 1994.

13. "Vibrations of Elevated Structures and Bridges Caused by High Speed Train Loadings," (with J. Penzien and Kee-Dong Kang), *Proc.*, Korean Society of Civil Engineers (KSCE) Annual Meeting, Pusan, Korea, October 21-22, 1994.

14. "Development of Structural Fitness-For-Service Criteria for Evaluation of an Underground Natural Gas Pipeline," by Wen S. Tseng and Chih-Hung Lee, July 1994.

15. "Soil-Structure Interaction Effects for Deep Foundation Systems of Long-Span Bridges," by Wen S. Tseng, C. Y. Chang, W. D. Liu, and Rouppen Donikian, April 1995.

16. "Seismic Performance Evaluation of Major Steel Bridges in California" (with R. R. Donikian and C. Y. Chang). *Proc.*, American Society of Civil Engineers (ASCE) Structures Congress XIII, Boston, Massachusetts, April 2-5, 1995.

17. "Seismic Response Analysis of Nuclear Power Plant Structures Considering Spatial Incoherency of Ground Motions," (with K. Lilhanand, D. Hamasaki, H. T. Tang, and Y. B. Tsai), Nuclear Science Journal, Vol. 32, No. 2, April 1995.

18. "Structural Performance Criteria for Fitness-for-Service Evaluations of Underground Natural Gas Pipelines," (with Chih-Hong Lee), *Proc.*, 2nd International Conference on Advances in Underground Pipeline Engineering, Seattle, Washington, June 25-27, 1995.

19. "Soil-Foundation-Structure Interaction Analysis by the Elasto-Dynamic Method," *Proc.*, the Fourth Caltrans Seismic Research workshop, Sacramento, July 9-11, 1997.

20. "Hybrid Method for Evaluating Soil-Foundation-Structure Interaction Effects," W.S. Tseng and J. Penzien, *Proc.*, 5th Caltrans Seismic Research Workshop, sacramento, CA, June 16-18, 1998.

21. "Soil-Foundation-Structure Interaction," W.S. Tseng and J. Penzien, Chapter 42 of Handbook of Bridge Engineering, Chen, W.F. and Duan, L., Editors, CRC Press, LLC, 2000.

April 1, 2002

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

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

APPLICANT'S PREFACE OF THE TESTIMONY OF KRISHNA P. SINGH AND ALAN I. SOLER ON UNIFIED CONTENTION UTAH L/QQ

I. WITNESSES

A. Krishna P. Singh

Krishna P. Singh is President and CEO of Holtec International ("Holtec") and bears the ultimate corporate responsibility for the accuracy and correctness of Holtec's spent fuel dry storage systems. Dr. Singh has a Ph.D. in Mechanical Engineering and has extensive experience in the design and licensing of nuclear spent fuel systems extending 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. He is also the inventor of the honeycomb basket design utilized in the HI-STAR 100/HI-STORM Systems and the METCON™ construction used in the HI-STORM System overpack. His professional work in the field of applied heat transfer and structural mechanics consists of over 500 industry reports, over fifty published papers in the refereed technical literature, and academic courses taught at the University of Pennsylvania..

B. Alan I. Soler

Dr. Alan I. Soler is the Executive Vice President and Vice-President of Engineering for Holtec International. He is responsible for all corporate engineering activities by the company, including overseeing the analyses performed to establish the stability of the HI-STORM 100 System under postulated seismic events. Dr. Soler is the lead structural discipline expert responsible for the design of the HI-STORM System, including supporting analyses, and he has acted in this capacity since the design was conceptualized in the early 1990s. Dr. Soler 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 Dr. Soler's employment with Holtec International, he was a tenured Professor of Mechanical Engineering and Applied Mechanics at the University of Pennsylvania for over 26 years. Through Dr. Soler's professional and educational background and work experience, he is qualified to address matters pertaining to the effects of seismic and structural loadings on the HI-STORM System.

II. TESTIMONY

A. Scope of Testimony

Drs. Singh and Soler will testify in response to claims made by the State with respect to the seismic analysis of the HI-STORM 100 Cask System to be deployed at the PFSF. In response to these allegations, Dr. Soler and Dr. Singh will: (1) summarize the design of the HI-STORM System; (2) describe the features and conservatisms in the design of the HI-STORM System that enhance the ability of the casks and the fuel canisters inside the casks to withstand the forces imparted on them during a severe seismic event; (3) report the results of the analyses performed of the casks' response to a 2,000 year return period earthquake at the PFSF and other, more severe seismic events; (4) respond to claims raised by the Sate of Utah in Section C.3(e) and portions of Section D of the Unified Contention; (5) respond to claims concerning the modeling of the stability of the HI-STORM System under earthquake forces raised by the State's witness Dr. Moshin Khan, and (6) address the capability of the HI-STORM System to withstand earthquake forces significantly beyond those imparted by the 2,000 year return period design basis earthquake for the PFSF, including the forces due to the 10,000 year return period earthquake for the site.

B. Design Capability of the HI-STORM 100 to Withstand Seismic Events

Drs. Singh and Soler will testify to the ruggedness of the HI-STORM System and the analyses that Holtec has performed to show that the HI-STORM System can withstand seismic events far more severe than the 2,000 year design basis earthquake, including the postulated 10,000 year earthquake. They will present simulations of the HI-STORM 100 which show that the casks will not tip-over under the postulated 10,000 year ground motions, with significant margins remaining, even under unduly harsh worst case assumptions. They will further describe how their computer code used to model the HI-STORM has been validated and approved by the NRC and has been used as the licensing basis for spent fuel technology at more than 40 nuclear plants throughout the world. Finally, they will testify to the large margin in the multi-purpose canister ("MPC") in which the spent fuel is sealed that would serve to confine the spent fuel even assuming hypothetically that a HI-STORM storage cask would tip over under a seismic event.

C. Response to State of Utah Claims in Unified Contention

Drs. Singh and Soler will respond to many of the numerous claims raised by the State in the Unified Contention. They will show how those claims are either incorrect or constitute insignificant second order effects that have no bearing given the huge safety margins inherent in the HI-STORM 100 design. They will also respond to claims raised by the State's witness, Dr. Moshin Khan. They will testify that Dr. Khan, who has never before modeled large free standing objects such as the HI-STORM 100 Cask System, has ignored authoritative guidance on the modeling of friction contact problems and has made fundamentally flawed assumptions in his model that cause the model to predict result that defy Physics. They will explain how Holtec's computer code has been benchmarked to provide results that correspond to physical reality for the modeling of contact friction problems and how the model has been reviewed and approved by the NRC as the licensing basis for spent fuel storage systems throughout the country.

April 1, 2002

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

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

TESTIMONY OF KRISHNA P. SINGH AND ALAN I. SOLER ON UNIFIED CONTENTION UTAH L/QQ

I. BACKGROUND – WITNESSES

A. Krishna P. Singh ("KPS")

Q1. Please state your full name.

A1. Krishna P. Singh.

Q2. By whom are you employed and what is your position?

A2. (KPS) I am President and CEO of Holtec International ("Holtec"). In that position, I bear the ultimate corporate responsibility for the accuracy and correctness of the company's spent fuel storage systems engineered for dry storage under certification by the U.S. Nuclear Regulatory Commission ("NRC").

Q3. Please summarize your educational and professional qualifications.

A3. (KPS) My professional and educational experience is described in the *curriculum vitae* attached as to this testimony. I have a Ph. D in Mechanical Engineering, which I received from the University of Pennsylvania in 1972. I have extensive experience in the design and licensing of nuclear spent fuel systems which extends back to 1979. Over the past twenty-three years, I have personally led the

design and licensing of spent fuel storage systems for over forty nuclear power plants, and for Holtec's HI-STAR 100 System and HI-STORM 100 Storage Cask System ("HI-STORM System"). I am also the inventor of the honeycomb basket design utilized in the HI-STAR 100/HI-STORM Systems (Patent Number 5,898,747) and the METCON™ construction used in the HI-STORM System overpack (Patent No. 6,064,710). The internal thermosiphon feature of the HI-STORM System multi-purpose canisters, widely recognized as a seminal contribution to dry storage technology, was conceptualized and implemented under my technical leadership. My professional work in the field of applied heat transfer and structural mechanics, to which this testimony in part pertains, consists of over 500 industry reports, over fifty published papers in the refereed technical literature, and academic courses taught at the University of Pennsylvania. I have served as AN expert witness in three prior Atomic Safety and Licensing Board hearings dealing with wet storage of spent nuclear fuel.

Q4. What knowledge do you have of American Society of Mechanical Engineers Boiler and Pressure Vessel Code standards?

A4. (KPS) I have designed hundreds of pressure vessels to the ASME Boiler and Pressure Vessel codes. over 40 nuclear plants have pressure vessels designed by me, or under my supervision, in use throughout the world.

Q5. What is your experience with nuclear facilities and the requirements of the NRC for the design and licensing of dry cask storage systems?

A5. (KPS) My company, Holtec International, has three dockets with the NRC on dry storage systems (72-1014, 72-1008, and 71-9261) for the HI-STORM System, the HI-STAR 100 Cask Storage System and the HI-STAR 100 Cask Transport System, respectively. Each docket has obtained a Certificate of Compliance ("CoC"), all of which have been secured under my technical direction and leadership.

Q6. Are you familiar with the Private Fuel Storage Facility ("PFSF") and the activities that will take place there?

A6. Yes.

Q7. What is the basis of your familiarity with the PFSF?

A7. (KPS) I have provided consultation and technical oversight to the analysts involved in evaluating the effects of seismic excitations on the HI-STORM System which is to be deployed at the PFSF Independent Spent Fuel Storage Installation ("ISFSI"). I have personally visited the proposed dry storage facility in Skull Valley. I have been directly involved with PFS's technical management from the inception of the Skull Valley project, because PFS had selected the HI-STORM technology even before the selection of the most eligible site was made. I have also reviewed Unified Contention Utah L/QQ ("the Unified Contention"), in which the State of Utah raises various challenges to the seismic analysis of the HI-STORM System for the PFSF site, and related materials.

Q8. What is the purpose of your testimony?

A8. (KPS) The purpose of my testimony is to respond to allegations raised by the State of Utah in the Unified Contention concerning the seismic analysis of the HI-STORM 100 System to be deployed at the PFSF. In response to these allegations, Dr. Soler and I will: (1) summarize the design of the HI-STORM System; (2) describe the features of and conservatisms incorporated in the design of the HI-STORM System that enhance the ability of the casks and the fuel canisters inside the casks to withstand the forces imparted on them during a severe seismic event; (3) report the results of the analyses performed on the casks' response to a 2,000-year return period earthquake at the PFSF and other, more severe seismic events; (4) respond to claims raised by the State of Utah in Section C.3(e) and portions of Section D of the Unified Contention; (5) respond to claims concerning the modeling of the stability of the HI-STORM System under earthquake forces raised by the State's witness, Dr. Moshin Khan; and (6) address the capability of the HI-STORM System to withstand earthquake forces significantly beyond those imparted by the 2,000-year return period design basis earthquake for the PFSF, including the forces due to the 10,000-year return period earthquake for the site.

B. Alan I. Soler ("AIS")

Q9. Please state your full name.

A9. Alan I. Soler.

Q10. By whom are you employed and what is your position?

A10. (AIS) I am Executive Vice President and Vice President of Engineering for Holtec International. In that capacity, I am responsible for all corporate engineering activities by the company, including overseeing the analyses performed to establish the stability of the HI-STORM System under postulated seismic events. I am the lead structural discipline expert responsible for the design of the HI-STORM System, including supporting analyses, and have acted in this capacity since the design was conceptualized in the early 1990s. In particular, I have either performed or reviewed all HI-STORM System seismic analyses conducted in support of deployment of the HI-STORM System at the PFSF.

Q11. Please summarize your educational and professional qualifications.

A11. (AIS) My professional and educational experience is described in the *Curriculum Vitae* attached to this testimony. Prior to my current employment with Holtec International, I was a tenured Professor of Mechanical Engineering and Applied Mechanics at the University of Pennsylvania for over 26 years. During my academic career at the University of Pennsylvania, I taught graduate and undergraduate courses in mechanical engineering, engaged in funded research, and was an active consultant to the nuclear industry on various mechanical engineering matters, including spent fuel storage equipment. Through my professional and educational background and work experience, I am qualified to address matters pertaining to the effects of seismic and structural loadings on the HI-STORM System.

Q12. What knowledge do you have of American Society of Mechanical Engineers Boiler and Pressure Vessel Code standards?

A12. (AIS) In the course of my activities in seismic and structural analysis at Holtec International, I use Section VIII, Divisions 1 and 2, Section II, and Section III, Subsections NB-NG, extensively. I have also served for over ten years as a member of an ASME Working Group to develop Section VIII, Division I of the ASME code. These provisions of the code pertain to the design methodologies and fabrication of Nuclear and Non-Nuclear pressure vessels and pressure bearing components. Included, among other items I am familiar with in the various sections of the Code, are tables of allowable stresses for various materials of construction, classification of loads, and formulas for determining the state of stress in some common constructions.

Q13. What is your experience with nuclear facilities and the NRC's requirements for the design and licensing of dry cask storage systems?

A13. (AIS) I led the structural and seismic effort for obtaining the CoC for the HI-STAR and HI-STORM Systems, and in so doing I became familiar with the applicable sections of the NRC guidance documents for the design and licensing of dry cask storage and transport systems. I have also been responsible for the seismic and structural analysis of spent fuel racks for numerous nuclear plants. Over 40 nuclear plants have spent fuel storage devices that were designed using the analysis methodology that I developed. In addition to Holtec's dry storage systems, I have also performed seismic stability evaluations for other casks such as the TN -12 and 1F-300. The analysis I performed for the latter served as the basis for defueling the Shoreham Nuclear Plant in the early nineties.

Q14. Are you familiar with the PFSF and the activities that will take place there?

A14. (AIS) Yes.

Q15. What is the basis of your familiarity with the PFSF?

A15. (AIS) I performed the seismic analyses for the HI-STORM System to be deployed at the PFSF ISFSI. I developed the original model of 1-8 spent fuel dry storage casks on the ISFSI pad resting on a soil foundation using the Holtec validated computer code for dynamic simulation. I performed the original

analysis for PFSF using a deterministic earthquake and directed and reviewed the follow-on efforts utilizing various probabilistic seismic events. Most recently, I developed and performed the large motion dynamic simulation of the HI-STORM System, subject to the beyond-design-basis 10,000-year return seismic event. I also directed and reviewed the drop and tip-over analyses of the storage cask that are required to demonstrate that the enclosed spent fuel will not experience excessive deceleration levels in the event of a handling accident or a non-mechanistic tip-over. Based on my experience with the PFSF project over the past several years, I am familiar with the site-specific characteristics of the site's subsoil and the design features of the concrete pad on which the casks will rest, and understand how the subsoil characteristics affect the seismic analyses performed on the HI-STORM System at the PFSF ISFSI. I have also reviewed the Unified Contention, in which the State of Utah raises various challenges to the seismic analysis of the HI-STORM System for the PFSF site, and related materials.

Q16. What is the purpose of your testimony?

A16. (AIS) The purpose of my testimony is to respond to allegations raised by the State of Utah in the Unified Contention concerning the seismic analysis of the HI-STORM System to be deployed at the PFSF. Dr. Singh and I will (1) summarize the design of the HI-STORM System; (2) describe the features and conservatisms in the design of the HI-STORM System that enhance the ability of the casks and the fuel canisters inside the casks to withstand the forces imparted on them during a severe seismic event; (3) report the results of the analyses performed of the casks' response to a 2,000-year return period earthquake at the PFSF and other, more severe, seismic events; (4) respond to claims raised by the State of Utah in Section C.3(e) and portions of Section D of the Unified Contention; (5) respond to claims concerning the modeling of the stability of the HI-STORM System under earthquake forces raised by the State's witness, Dr. Moshin Khan; and (6) address the capability of the HI-STORM System to withstand earthquake forces significantly beyond those imparted by the 2,000-year return period design basis

earthquake for the PFSF, including the forces due to the 10,000-year return period earthquake for the site.

II. DESIGN FEATURES OF THE HI-STORM SYSTEM CASKS AND CANISTERS THAT ENABLE THEM TO WITHSTAND SEISMIC FORCES

Q17. Please describe the general design of the HI-STORM System to be used at the PFSF ISFSI.

A17. (KPS) The HI-STORM System features a massive cylindrical steel and concrete storage cask surrounding a multi-purpose stainless steel canister in which the spent nuclear fuel is sealed, as shown in the figure below: [Alan will attach a solidworks rendering] The casks are almost 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 cylindrical 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 multi-purpose canister ("MPC") in which the spent fuel is sealed is stored vertically within the storage cask. Loaded HI-STORM System casks are placed on concrete storage pads using a specially designed transporter.

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 inside the cask. 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.

The cask is engineered to minimize local area radiation doses and to provide a robust structural enclosure for the MPC located within it. Specifically, the storage cask is designed to withstand extreme natural phenomena, including strong earthquakes. The loaded HI-STORM System storage cask exhibits excellent

resistance to overturning under seismic events. This high resistance to overturning is partly due to its low height-to-diameter ratio (239.5 inches to 132.5 inches, a height-to-diameter ratio of 1.8). Its seismic resistance is further enhanced by the energy absorbing internal channels mentioned above, by the state of internal dissonance produced by the vibrating of the MPC within the cask and by the individual fuel assemblies in their respective storage locations.

Q18. How will the storage casks be stored at the PFSF site?

A18. (AIS) As described in Section 4.2.1.5.2 of the PFSF Safety Analysis Report ("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. 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 direction and 16 ft in the length 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. For a graphical representation of the cask storage arrangement, see Figure 4.2-7 in the PSFS SAR.

Q19. Please describe the codes and standards to which the HI-STORM System is designed and manufactured.

A19. (KPS) The array of codes and standards used in the design of the HI-STORM System are listed in the HI-STORM FSAR. In particular, the HI-STORM System is designed and constructed in accordance, as applicable, with Section III of the American Society of Mechanical Engineers Boiler and Pressure Vessel Code ("the Code"). The Code governs the design of pressure vessels for safety-related applications at nuclear power plants. The manner of compliance with the Code is described in the HI-STORM System FSAR. The multi-purpose canister is engineered in accordance with Subsection NB of the Code, which governs the construction of Class 1 nuclear components. Class 1 nuclear components include such items as reactor pressure vessels and primary coolant system piping. Use of

Subsection NB for the construction of the MPC is highly conservative since the MPC design pressure is much lower than the design pressure for a typical reactor coolant system (i.e., 100 psig versus 2,500 psig or higher) and there is no significant cycling of the stress state in the service condition of the MPC, eliminating fatigue as a concern. The internal fuel basket is designed to Subsection NG of the Code, which governs the construction of nuclear component core support structures. The HI-STORM System storage cask is designed in accordance with Subsection NF of the Code, which governs the construction of nuclear component supports, such as spent fuel racks and reactor coolant piping supports. Thus, the MPC and the storage casks are designed and built to the same standards, as applicable, as safety-related components used in nuclear power plants. In addition, the HI-STORM System components are designed in accordance with the standards specified in the governing NRC Standard Review Plan, NUREG-1536, "Standard Review Plan for Dry Cask Storage Systems", January 1997.

Q20. How do the standards specified in NUREG-1536 for dry storage cask systems compare to the standards specified in the NRC's Standard Review Plan for nuclear power plants set forth in NUREG-0800?

A20. (KPS, AIS) NUREG 1536 provides guidance to NRC reviewers of Dry Cask Storage Systems ("DCSS"). From the standpoint of seismic/structural considerations, NUREG-1536 for dry storage incorporates the lessons learned from the evolutionary development of its counterpart NUREG-0800 for reactor systems. The differences in the two NUREGs principally lie in the difference in their technical missions. For example, whereas NUREG-0800 does not dwell on the structural consequences of tornado-borne missiles on a spent fuel storage rack in the plant's fuel pool (the pools being completely enclosed, reinforced concrete monoliths), the ability of the storage cask, situated outdoors, to withstand impactive and impulsive tornado loads is treated as an important consideration in NUREG-1536. Likewise, the amplification of the earthquake by the interplay between the flexibility of the fuel storage buildings and the free field seismic motion is a matter of considerable attention in NUREG-0800. Because vertical

ventilated casks (particularly HI-STORM) are essentially rigid structures to a seismic input, the focus of consideration in NUREG-1536 is directed towards evaluating the effects of free-standing massive rigid bodies under seismic events.

In summary, NUREG-1536 calls for application of the same codes, standards and design procedures as does NUREG-0800. The difference in the details of the guidance are almost entirely due to the differences in the type, nature, relative significance and relevance of the anticipated loadings between dry storage casks and reactor installations.

Q21. Please describe in greater detail the design of the HI-STORM System storage casks.

A21. (KPS) As required by NUREG-1536 and other applicable codes and standards, the design of the HI-STORM System storage cask has significant built-in conservatisms and design margins that assure its ability to perform in accordance with design basis requirements and to withstand events well beyond its design basis. The HI-STORM System storage casks are stubby steel weldments with homogeneous concrete (without rebars or other potential sources of crack propagation), designed to tolerate very large earthquake-induced forces without tipping over. To assure utmost structural ruggedness, the HI-STORM System storage cask has been designed as a buttressed ASME Section III, Class 3, Subsection NF cylindrical structure. The 1¼ -inch thick inner steel shell and ¾ inch thick outer steel shell are both welded to a 2 inch thick baseplate, and are joined by four full-length inter-shell radial support plates, each ¾ -inch thick and welded to the inner and outer shells. The cask provides an internal cylindrical cavity, 191½ inches in height and 73½ inches in diameter, for housing the MPCs. 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 to minimize g-loadings imparted to the MPC under a hypothetical cask tip-over scenario.

Q22. Please describe in greater detail the design of the multi-purpose canister.

A22. (KPS) The multi-purpose canister is the component in which the spent fuel is placed. After the spent fuel is loaded into the MPC, the MPC is filled with an inert gas (helium) and welded shut for long-term storage at a site or ready 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 Code. The fuel basket is a stainless steel, continuously welded, stiff honeycomb structure that is designed to meet Subsection NG of the Code, as applicable, and serves to position the fuel in the MPC enclosure vessel. The MPC has the same relative design margins as those imposed by Subsection NB of the Code for reactor operation service, even though the MPC is not subject to the stresses that result from an operating reactor environment. Further, the MPC is designed for transportation as well as storage, giving it a ruggedness that allows it to resist very large earthquake induced forces. Thus, similar to the storage casks, the MPC has significant built-in conservatisms and design margins that assure its ability to perform in accordance with its design basis requirements and to withstand events well beyond its design basis.

Q23. Has Holtec performed any analyses that demonstrate the beyond-design basis conservatisms and capabilities of the MPC?

A23. (KPS, AIS) Yes. Holtec performed an analysis to determine whether the confinement boundary of the MPC would be breached in the hypothetical, postulated case of a crane failure or other malfunction that causes a drop of an MPC that is in the process of being loaded into a cask. At the PFSF, a loaded MPC will be transferred from the transportation cask in which it is shipped to the site to the HI-STORM System storage cask in the Canister Transfer Building. To perform this transfer, the HI-TRAC transfer cask is placed on top of the transportation cask, the MPC is lifted up into the transfer cask, the loaded transfer cask is moved by a crane over to the storage cask, and the MPC is placed inside the storage cask. (This process is described in the testimony of Donald Wayne Lewis being filed simultaneously with this testimony.)

In the analysis performed by Holtec, the MPC is assumed to free-fall over a distance of 25 feet, representing the height of the storage cask cavity plus an allowance for the thickness of the transfer cask bottom lid. The target surface is assumed to be essentially unyielding and is modeled as a 22 ft thick concrete slab of compressive strength 6,000 psi. The computed strain in the confinement boundary material as a result of this hypothetical drop is only 41% of the failure strain limits for the material. Therefore, the MPC confinement boundary integrity is maintained and radioactive material is not released into the environment even under this severe, hypothetical drop accident. This hypothetical drop accident is far more severe than either the drop accident analysis or hypothetical tip-over performed as part of the design basis of the HI-STORM System. It demonstrates the huge margins provided by the Code and design criteria that enable the MPC to withstand forces much greater than the design basis forces and still perform its safety function.

III. ABILITY OF THE HI-STORM SYSTEM STORAGE CASKS AND CANISTERS TO WITHSTAND SEISMIC EVENTS POSTULATED FOR THE PFSF

A. General Background

Q24. Please describe the regulatory requirements for the seismic performance of dry cask storage systems, such as the HI-STORM System.

A24. (KPS, AIS) The regulatory requirements for the seismic performance of Dry Cask Storage Systems are stated in 10 C.F.R. § 72.122(b)(2) and translated into guidance to the NRC Staff in NUREG-1536. 10 C.F.R. § 72.122(b)(2) states that "structures, systems, and components important to safety must be designed to withstand the effects of natural phenomena such as earthquakes,.without impairing their capability to perform safety functions. The design bases for these structures, systems, and components 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." NUREG-1536 addresses these requirements in Section V.1.d.(i)(3), subparagraph (g) and requires that ... "Cask designs must satisfy the load combinations that encompass earthquake, including those for sliding and overturning in ANSI/ANS-57.9, Section 6.17.4.1. The applicant should demonstrate that no tip-over or drop will result from an earthquake. In addition, impacts between casks should either be precluded, or should be considered an accident event for which the cask must be shown to be structurally adequate."

Q25. In general, how does one demonstrate that these requirements are satisfied?

A25. (KPS, AIS) To demonstrate that the above requirements are satisfied, a comprehensive dynamic model of the casks, the supporting pad, and the soil foundation is constructed and a series of dynamic simulations performed with the input loading being the specified three-dimensional seismic acceleration time histories for the design basis earthquake. Because the storage casks are free-standing (not anchored) on the pad, and since each storage cask contains a large free standing body (the MPC) inside, the dynamic simulation requires a non-linear analysis. A non-linear analysis recognizes that the relationships between load and deformation are not linear and that changes in orientation may be large enough to require a re-formulation of the governing equations of equilibrium at each instant in time. Classical solution methods, such as modal analysis in the time or frequency domain, are inapplicable to such a problem and the only recourse to ensure an accurate representation of the response is to use a direct solution of the differential equations of motion in the time domain. The modeled system is subject to the earthquake induced forces, and the solution over the event duration is obtained. At each instant in time, the position and orientation of each cask in the model is determined in order to draw conclusions concerning cask stability and cask-to-cask impact. In order to encompass the wide variety of configurations and the potential for sliding and/or overturning of one or more casks, multiple simulations are performed with upper and lower bound cask-to-pad coefficients of friction, and for varying numbers of casks on the pad.

Q26. Has Holtec developed a computer code to perform this dynamic analysis of the cask system?

A26. Yes. Holtec has developed a specialized computer code, referred to as DYNAMO, for modeling spent fuel systems to demonstrate their compliance with NRC seismic requirements. This code has been validated and has been reviewed and accepted by the NRC for the licensing of spent fuel storage systems.

Q27. Please describe the various seismic analyses that Holtec has performed for the HI-STORM System.

A27. (KPS, AIS) Holtec has performed general 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 System under 10 C.F.R. Part 72. Under the CoC, nuclear power plant licensees may use the HI-STORM System at their sites under the general license provision of 10 C.F.R. § 72.210 as long as they meet the conditions of both 10 C.F.R. § 72.212 and the CoC. Holtec has also performed seismic analyses for ISFSIs that do not fall under the general license provisions of 10 C.F.R. Part 72. In addition to the seismic analyses for the PFSF, Holtec has performed site-specific seismic analyses using DYNAMO for the HI-STORM System for Pacific Gas & Electric (Diablo Canyon), Exelon (Dresden), Energy Northwest (Columbia Generating Station), Entergy Nuclear Northeast (J.A. Fitzpatrick) and Tennessee Valley Authority (Sequoyah). These analyses were performed using DYNAMO to demonstrate that the HI-STORM System would perform satisfactorily under seismic conditions at all these sites.

Q28. Does Holtec have other relevant experience performing seismic analyses for spent fuel storage systems?

A28. (KPS, AIS) Yes. In addition to the work in dry storage system seismic analysis, Holtec has extensive experience in the seismic qualification of spent fuel racks used inside nuclear plants. The spent fuel racks are large rectangular structures of honeycomb construction that are free standing 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 spent fuel 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. The same computer code is used to model the spent fuel rack behavior that is used to model the behavior of one or more spent fuel casks on an ISFSI pad. No changes to the code were required in order to simulate the behavior of the casks; the input data for a particular site reflects the differences between simulating submerged spent fuel racks and simulating dry casks.

Holtec has employed its wet storage seismic simulation methodology at many nuclear sites, both in the U.S. and abroad. The list below provides a partial list of U.S. and foreign sites where Holtec has performed seismic analyses for spent fuel rack systems that were licensed by the applicable regulatory authority. In all such activities, Holtec's QA validated computer code DYNAMO was employed.

PLANT	DOCKET NUMBER(s)	YEAR
Enrico Fermi Unit 2	USNRC 50-341	1980
Quad Cities 1 & 2	USNRC 50-254, 50-265	1981
Rancho Seco	USNRC 50-312	1982
Grand Gulf Unit 1	USNRC 50-416	1984
Oyster Creek	USNRC 50-219	1984
Pilgrim	USNRC 50-293	1985
V.C. Summer	USNRC 50-395	1984
Diablo Canyon Units 1 & 2	USNRC 50-275, 50-323	1986
Byron Units 1 & 2	USNRC 50-454, 50-455	1987
Braidwood Units 1 & 2	USNRC 50-456, 50-457	1987
Vogtle Unit 2	USNRC 50-425	1988
St. Lucie Unit 1	USNRC 50-335	1987

Millstone Point Unit 1	USNRC 50-245	1989
Chinshan	Taiwan Power Company	1988
D.C. Cook Units 1 & 2	USNRC 50-315, 50-316	1992
Indian Point Unit 2	USNRC 50-247	1990
Three Mile Island Unit 1	USNRC 50-289	1991
James A. FitzPatrick	USNRC 50-333	1990
Shearon Harris Unit 2	USNRC 50-401	1991
Hope Creek	USNRC 50-354	1990
Kuosheng Units 1 & 2	Taiwan Power Company	1990

PLANT	DOCKET NUMBER(s)	YEAR
Ulchin Unit 2	Korea Electric Power Co.	1990
Laguna Verde Units 1 & 2	Comision Federal de Electricidad	1991
Zion Station Units 1 & 2	USNRC 50-295, 50-304	1992
Sequoyah	USNRC 50-327, 50-328	1992
LaSalle Unit 1	USNRC 50-373	1992
Duane Arnold Energy Center	USNRC 50-331	1992
Fort Calhoun	USNRC 50-285	1992
Nine Mile Point Unit 1	USNRC 50-220	1993
Beaver Valley Unit 1	USNRC 50-334	1992
Salem Units 1 & 2	USNRC 50-272, 50-311	1993
Limerick	USNRC 50-352, 50-353	1994
Ulchin Unit 1	KINS	1995
Yonggwang Units 1 & 2	KINS	1996
Kori-4	KINS	1996

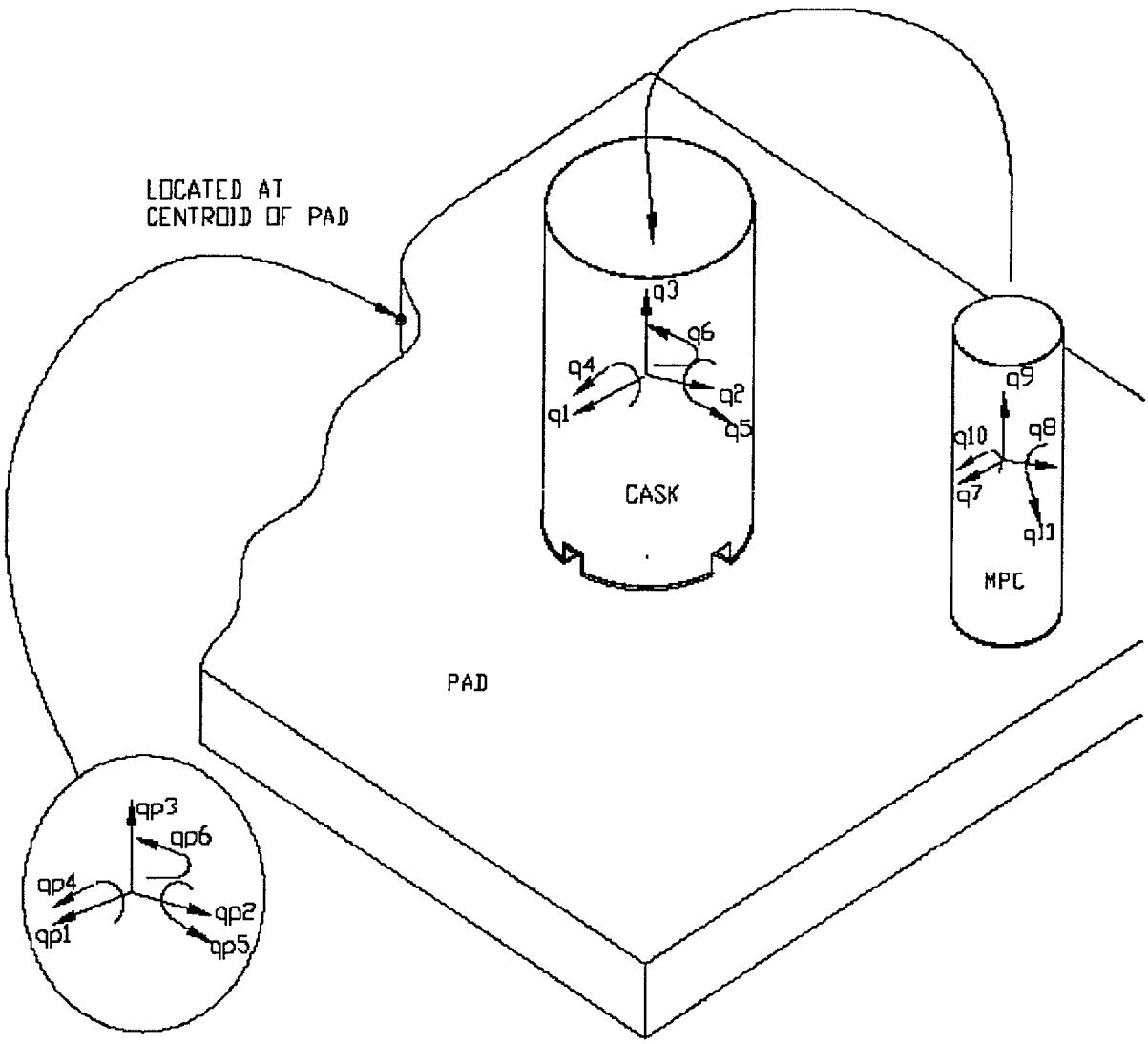
Connecticut Yankee	USNRC 50-213	1996
Angra Unit 1	Brazil	1996
Sizewell B	United Kingdom	1996
Waterford 3	USNRC 50-382	1997
J.A. Fitzpatrick	USNRC 50-333	1998
Callaway	USNRC 50-483	1998
Nine Mile Unit 1	USNRC 50-220	1998
Chin Shan	Taiwan Power Company	1998
Byron/Braidwood	USNRC 50-454, 50-455, 50-567, 50-457	1999
Wolf Creek	USNRC 50-482	1999
Plant Hatch Units 1 & 2	USNRC 50-321, 50-366	1999
Harris Pools C and D	USNRC 50-401	1999

Q29. Please generally describe the model used by Holtec for analyzing spent fuel storage systems.

A29. (AIS) The model used by Holtec for analyzing spent fuel storage systems (either casks for dry storage outside the plant structures on a separate pad, or racks for wet storage inside the plant facility) models the cask (or rack) as a multi-degree of freedom system. The contents of the cask or rack are modeled as a separate internal body that is free to contact the cask (or rack). The support on the floor is modeled by sets of compression-only contact elements with associated lateral resistance by friction elements. In the case of racks, the contact locations are beneath the support legs and the pool liner (generally located near the four corners of the structure), while in the case of casks, contact is defined to occur at a finite number of locations around the cask's circular perimeter. For the case of casks, a more detailed description of the model is provided below:

Each HI-STORM System cask is modeled as a two-body system. Each storage overpack is described by six degrees of freedom which capture the rigid body motion of the overpack in inertial space. Within each overpack, the internal MPC

is modeled by an additional five degrees of freedom sufficient to capture all but the rotational motion of the MPC about its own longitudinal axis. There is no loss of generality in this five degree of freedom system since there is no interest in the omitted rotational degree of freedom. Six degrees of freedom establish the rigid body motion of the ISFSI pad relative to inertial space. The complete system (multiple casks on a pad) is characterized by the aforementioned degrees of freedom (a set for each cask), by the mass and inertia properties of the component parts, and by the stiffness elements (linear and non-linear) that are used to characterize contact and friction between components and to characterize underlying pad and soil properties. The pad is subject to seismic movements at the base of soil springs, which represent the resistance of the soil foundation to pad translations and rotations. By changing the value for variables, the problem can be re-formulated as one in which the base of the soil springs is fixed, and three components of ground acceleration time histories of the earthquake, multiplied by the mass of the component are applied as specified inertia forces at the mass center of each moving body. The model simulates the application of earthquake forces with the pad, cask and canister are free to respond to the earthquake forces in any of the directional degrees of freedom described above. The figure below graphically illustrates the modeling concept.



HI-STORM 100 DYNAMIC
MODEL (DYMAMO)

Q30. You stated that your model has been validated and accepted by the NRC for the licensing of spent fuel storage systems. Please describe this validation process.

A30. (KPS, AIS) 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 where it could be 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. The problems were chosen to exercise all of the features that are built into DYNAMO (compression only behavior, friction resistance, etc.). In addition, problems that had no simple analytical solutions were also evaluated and shown to give good agreement with numerical solutions using finite element 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 certain wet storage 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 resolutions to the problem. As noted above, on numerous dockets, the NRC has accepted the results from DYNAMO as the basis for NRC licensing action. In summary, DYNAMO has been extensively benchmarked to confirm its veracity as a non-linear dynamics code.

B. Cask Stability Seismic Analyses of the HI-STORM System for Use at the PFSF

Q31. Please describe generally the seismic analyses that Holtec performed for the HI-STORM System to be used at the PFSF.

A31. (AIS) 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. Over the time period that Holtec has participated in the Project, a number of time history analyses were performed using different seismic events. The simulation model, however, was consistent through all of the analyses; namely, the casks, along with their loaded internals, were modeled as rigid bodies, the pad was modeled as a rigid rectangular slab, and the effect of the soil/soil cement foundation was modeled by appropriate springs and dampers characterizing the soil resistance in deflection and rotation. The casks were modeled as free-standing structures with compression-only

contact and with friction elements modeling the interfaces between casks and the pad. Seismic design input (acceleration time histories and soil properties to characterize the soil springs and dampers) were provided as design input by Geomatrix Consults, Inc. ("Geomatrix").

Q32. What were the PFSF site-specific ground motions and related information used by Holtec in its seismic analysis of the HI-STORM System for the PFSF?

A32. (AIS) The ground motions for the 2,000-year return period design basis seismic event were provided to Holtec by Geomatrix in the form of three acceleration time histories for 5% damping entitled "Fault Normal", Fault Parallel", and "Vertical". It is our understanding that these seismic ground motions were developed from 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 were developed in accordance with the requirements of Standard Review Plan 3.7.1 and had the following peak acceleration amplitudes:

Fault Normal – 0.73 g

Fault Parallel – 0.71 g

Vertical – 0.73 g

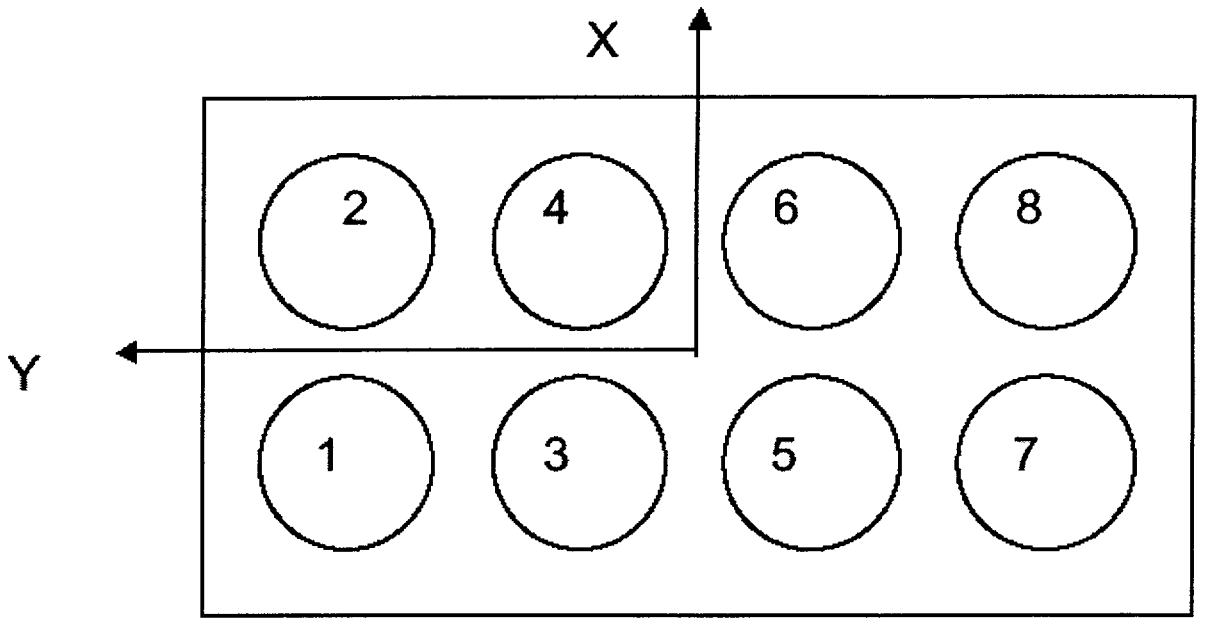
Along with the time histories, Geomatrix provided Holtec with the property values for the soil under the pad, including the effect of soil cement, as applicable. The "Best Estimate," "Lower Range," and "Upper Range" soil properties provided by Geomatrix are summarized in the table below:

RANGE OF SOIL PROPERTY VALUES 2000 Yr. Seismic Event			
	Young's Modulus, ksf	Shear Modulus, ksf	Poisson's Ratio
Lower Range	2,546	955	0.333
Best Estimate	5,194	2,027	0.281
Upper Range	12,234	5,015	0.220

The terminology "Lower Range" and "Upper Range" refers to the magnitude of the spring constants arising from the stated soil properties. The smaller values of Young's Modulus and Shear Modulus coupled with the larger value of Poisson's Ratio give rise to lower values for soil spring constants. The larger values of Young's Modulus and Shear Modulus coupled with the smaller values of Poisson's Ratio give rise to higher values for soil spring constants. The values of the spring constants and damping coefficients were computed by Holtec using the soil property values supplied by Geomatrix and applying 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.

Q33. What other PFSF site-specific design features were incorporated into Holtec's seismic analysis of the HI-STORM System for the PFSF?

A33. (AIS) The seismic analysis incorporated the PFSF site-specific dimensions for each storage pad of 67' x 30' x 3', with the casks arrayed, as shown in the figure below, as well as other relevant pad design information:



A single pad was modeled with the effect of the underlying soil foundation included by virtue of the six soil spring/dampers, calculated by Holtec based on soil properties provided by Geomatrix, located at the origin of the X-Y coordinate system at the base of the pad. The effect of soil cement under the pads was included in the modulii values used to model the springs. An effective soil mass or inertia was also included by Holtec in the model for each pad degree of freedom in accordance with formulas provided in Levy and Wilkerson, The Component Element Method in Dynamics..., McGraw-Hill, 1976.

Q34. Using this input information, what seismic analyses did Holtec perform?

A34. (AIS) Various configurations of one (1) to eight (8) casks were modeled using the lower bound, best estimate and upper range soil properties and an upper bound coefficient of friction of 0.8 at the cask/pad interface to emphasize the possibility of cask tipping, and a lower bound coefficient of friction of 0.2 to emphasize the possibility of sliding. The analyses are summarized in Section 8.2.1.2 of the PFSF SAR.

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 from the above table to identify the range of potential sliding. Only one configuration was evaluated at the 0.2 coefficient of friction based upon the results of previous cask stability analyses that Holtec had performed for the PFSF for different earthquakes, which 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.

Q35. What were the results of your analyses?

A35. (AIS) 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 do the analyses predict that there will be any cask tip-over or cask-to-cask impacts. Further, the maximum accelerations experienced by the casks (less than 8 g) are well below the design basis limits (of 45 g) specified by the HI-STORM System FSAR. These results confirm that the forces experienced by the cask and its internals in a design-basis earthquake do not produce stresses that exceed the allowable limits.

Q36. Please describe further the large margin against cask tip-over as shown by your analysis.

A36. (AIS) The following table summarizes the results from the nine Holtec analyses using a coefficient of friction of 0.8. The first column identifies the cases evaluated; the second and third columns show the maximum displacements in the X and Y directions as measured at the top of the casks; the fourth column shows the angle of tilt of the cask, which is measured by the net maximum displacement of the top of the cask in the horizontal X-Y plane (representing the net excursion of the cask from the vertical plane) and the height of the cask. The net maximum displacement in the X-Y plane is computed by a Square-Root-of Sum-of Squares ("SRSS") procedure using the extremes from each direction, which conservatively assumes that the maximum excursions in the two horizontal directions occur at the same time.

SUMMARY OF CASK SIMULATIONS (COEFFICIENT OF FRICTION=0.8)			
Simulation	Max. X-Displacement (absolute value), in.	Max. Y-Displacement (absolute value), in.	Angle of Tip (degrees – based on net top-of-cask displacement and height to top of cask body)
Casks in Position 1 and 2, Best Estimate Properties	2.06	3.24	0.950
Casks in Position 1 and 2, Lower Bound Properties	2.16	3.09	0.934
Casks in Position 1 and 2, Upper Bound Properties	2.58	3.24	1.026
Casks in Position 1to 4 Best Estimate Properties	2.14	3.16	0.945
Casks in Position 1 to 4, Lower Bound Properties	2.08	3.02	0.908
Casks in Position 1 to 4, Upper Bound Properties	2.17	3.23	0.964
Casks in Position 1 to 8, Best Estimate Properties	2.21	2.96	0.915
Casks in Position 1 to 8, Lower Bound Properties	2.04	2.51	0.801
Casks in Position 1 to 8, Upper Bound Properties	1.89	3.18	0.916

The case that produced the maximum displacement (identified by the largest angle of tip) was also evaluated for a coefficient of friction = 0.2. This evaluation

produced maximum displacement of 1.69 inches in the X direction and 1.94 inches in the Y direction.

As can be seen, the maximum angle of tilt indicated by the analysis for the 2,000-year design basis earthquake for the upper bound coefficient of friction of 0.8 is 1.026 degrees. This can be compared to the angle of tilt at which a cask would tip from the movement of its own weight. Using simple geometry and values for the cask diameter and the height of the cask center of mass above the top surface of the pad, the angle of inclination of the cask where the cask has its center-of-gravity directly over a corner of the cask (with the cask tipped up to such an angle) at which the cask would tip over from its own moment with no other force applied is 29.3 degrees. Defining a safety factor against exceeding the so-called "center-of-gravity-over-corner" location by the ratio of c.g-over corner angle to calculated angle of rotation from the vertical, which could signal the possibility of a continued rotation to a tipped-over horizontal position, it is shown that the minimum safety factor for the HI-STORM System for the PFSF design basis earthquake is 28.6, computed as follows:

Safety Factor (overturning) =29.3/1.026 = 28.6

Q37. Please describe further the large margin against cask-to-cask impact as shown by your analysis.

A37. (AIS) Since the maximum excursion predicted at the top of the cask is below 3.25 inches (and this is larger than that predicted for any case where the coefficient of friction is 0.2), a conservative safety factor against cask-to-cask sliding impact may be defined as the ratio of 50% of the cask-to-cask spacing divided by the computed net displacement. The result shows a safety factor of 5.79, computed as follow:

Safety Factor (cask-to-cask impact) = 24"/4.142" = 5.79

Q38. Did Holtec perform other seismic analyses of the HI-STORM System using earthquakes with the PGAs for the PFSF?

A38. (AIS) Yes. Holtec has performed a variety of seismic analyses for various earthquakes. In 1997 Holtec performed a seismic cask stability analysis for the HI-STORM System based on the seismic characterization for the PFSF site in the PFS June 1997 License Application, based on an earthquake with a vertical PGA of 0.69 and a horizontal PGA of 0.67. Then, in 1999, Holtec performed two other seismic cask stability analyses. The first was based on an 1,000-year return period earthquake with vertical and horizontal PGAs of 0.391 and 0.404, respectively, and the second was based on an initial 2,000-year return period earthquake with vertical and horizontal PGAs of 0.55. The results of these earlier analyses showed similarly large safety margins against overturning or sliding and impacting.

Q39. Did Holtec perform any analyses of the HI-STORM System at the PFSF for ground accelerations greater than those for the 2,000-year design basis earthquake?

A39. (AIS) Yes. Holtec performed an analysis 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 PGA of 1.33g and horizontal PGAs of 1.25g and 1.23g. This analysis used a conservative estimate of the coefficient of friction between the base of the cask and the top surface of the pad of 0.8, in order to maximize the possibility of tipping by the cask. The earthquake motion was assumed to be applied directly to the base of the pad so that soil springs were not included in the simulation. Since the rotations were expected to increase to a level where the orientation of the cask could significantly affect the equilibrium equations, a computer algorithm capable of including finite rotations was used for this analysis. Although the loaded cask exhibited larger rotations relative to the pad (approximately 10.89 degrees from the vertical) than seen in the earlier analyses using lower earthquake levels, the results of this analysis still showed the existence of significant margins against tip-over. Using the same definition of safety factor against cask overturning as before, the safety factor against overturning was 2.69, computed as follows:

Safety Factor (overturning) = 29.3/10.89 = 2.69

Thus, even at the 10,000-year earthquake ground motion level, large margins of safety against cask tip-over still exist.

Q40. In addition to these previously performed analyses, have you performed any further analyses of cask tipping and sliding at the PFSF?

A40. (AIS) Yes. In conjunction with the preparation of this testimony, we ran additional simulations to test alleged deficiencies that the State's experts claimed might affect our previous analyses by re-running our analyses using different assumptions than those used in the above described analyses. These additional simulations were done at both the 2,000 and 10,000-year return period earthquakes. The new analyses were run under unrealistic, worst case assumptions, yet all showed that the casks would remain upright and not tip over during a seismic event.

Q41. Based on the seismic analyses that you have performed, what is your conclusion regarding the capability of the HI-STORM System to withstand earthquake events at the PFSF site?

A41. (KPS, AIS) Based on the totality of the analyses performed for this Project by Holtec, which encompassed the entire range of friction coefficients likely at the interface between the casks and pad, and which also encompassed the expected range of cask positioning and number of casks present on the pad, we conclude that under the design-basis 2,000-year return period seismic event, the casks will remain vertical and not tip over, and will not impact each other. Moreover, a very large margin exists such that the HI-STORM System at the PFSF can withstand earthquakes with return periods significantly greater than the 2,000-year design basis earthquake, including earthquakes with 10,000-year return period ground motion, and not tip over.

Q42. Do any independent seismic analyses confirm your conclusions?

A42. (KPS, AIS) Yes. 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. The Sandia analysis considered a single cask on

the pad and included pad flexibility. Instead of using soil springs, the Sandia model used a finite element representation of the soil cement/soil foundation and extended the foundation boundary well beyond the pad boundary. Sandia's results that have been made available to us are for the 2,000-year return period earthquake for both 0.8 and 0.2 coefficients of friction, and for the 10,000-year return period event for the 0.2 coefficient of friction. All of the Sandia analyses we received confirmed that the casks will not tip over and will not impact one another during the postulated events. Moreover, the results obtained by Sandia are in the same general range as those that we have obtained (showing, at most, several inches of displacement for the 2,000-year design-basis ground motions), thus independently confirming the results of our analyses.

C. Cask Drop and Non-Mechanistic Tip-over Analyses for the PFSF

Q43. Did Holtec perform any analyses for PFS concerning either the dropping or postulated tip-over of a loaded HI-STORM System cask?

A43. (AIS) Yes. In accordance with the guidance in NUREG-1536, Holtec performed both cask drop and non-mechanistic postulated cask tip-over analyses of a loaded HI-STORM System cask at the PFSF site. The purpose of the analyses was to demonstrate that the deceleration experienced by the stored fuel in the HI-STORM System cask during each of the postulated vertical drop and tip-over accidents remains below the design basis deceleration of 45 g limit as specified in the HI-STORM System CoC. The pad thickness at PFSF site is 36 inches, which equals the reference pad thickness criteria in the HI-STORM FSAR. The soil foundation, beginning 2 feet below the pad, has an effective soil Young's Modulus no greater than 28,000 psi, which meets the reference Young's Modulus criteria in the HI-STORM FSAR. The first two feet of foundation directly below the pad concrete consist of cement-treated soil having an effective Young's Modulus no greater than 75,000 psi. To ensure that the design basis deceleration limit is met for the specific conditions at the PFSF site, Holtec performed transient finite element analyses to simulate postulated accidents involving the vertical drop and the non-mechanistic tip-over of a loaded HI-STORM System

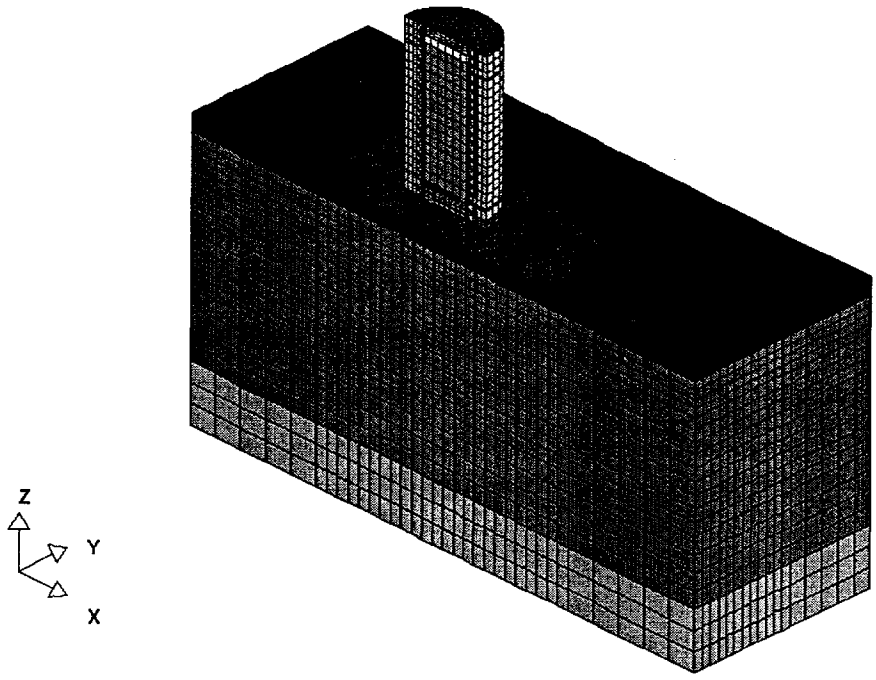
cask using the same methodology and computer codes used in the HI-STORM System FSAR. Holtec used the same methodology and computer codes for these cases as was used in its other analyses.

Q44. Please briefly describe the cask drop analysis that Holtec performed for the PFSF.

A44. (AIS) A loaded HI-STORM System cask was assumed to drop from a specified height, with its longitudinal axis in the vertical orientation, such that its bottom plate hit the pad; two different drop heights were evaluated. The cask steel components were modeled using elastic-plastic material shell and solid elements, the concrete in the cask and in the pad was modeled using a non-linear concrete material model that has been accepted by the NRC, and the soil layers (including the soil cement) were modeled conservatively by linear elastic materials with no permanent energy absorption capability. The parameters of the cask storage pad at the PFSF and the underlying soil layers are summarized below:

Item	Concrete Pad	Soil Cement	Soil Layer 1	Soil Layer 2
Thickness (ft)	3	2	26	7
Compressive Strength (psi)	4,200	---	---	---
Young's Modulus (psi)	---	75,000	6,000	12,000
Poisson's Ratio	0.22	0.2	0.3	0.3
Density (pcf)	140	105	91	115

The finite element model for the drop (only half the structure is modeled by virtue of symmetry) is shown in the figure below:



The calculated deceleration results from the two drop analyses (from a height of 6.5" and 10") were:

Drop height = 10" – longitudinal deceleration experienced by fuel = 45.15 g.

Drop height = 6.5" – longitudinal deceleration experienced by fuel = 36.15 g.

The predicted decelerations are consistent with the design basis decelerations in the Holtec CoC, although the deceleration for a 10" drop is slightly above the 45 g design limit. These decelerations translate into even larger margins of safety against the release of radioactivity, in that to actually breach a canister requires deceleration levels far in excess of those predicted by these analyses.

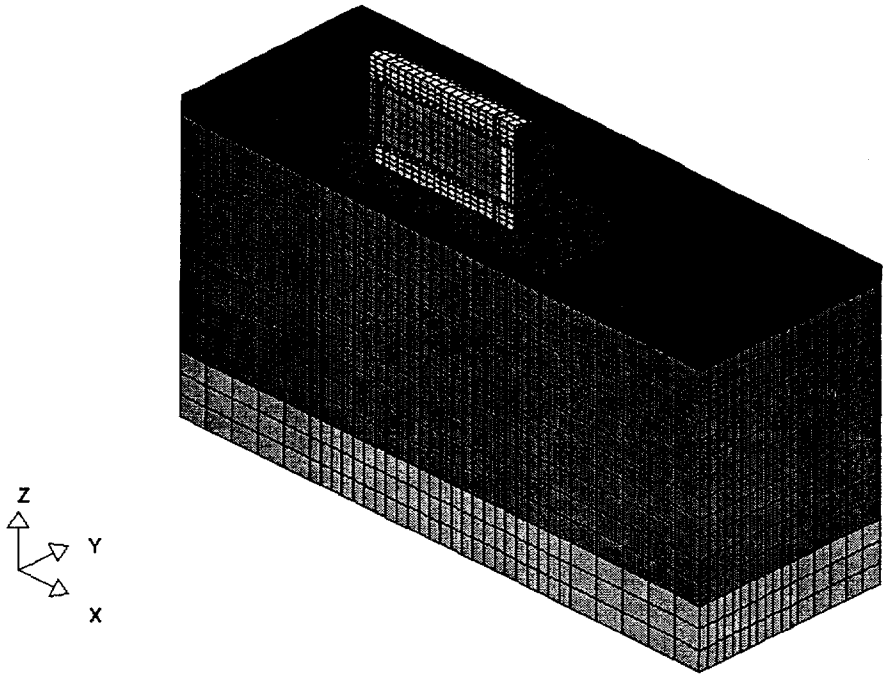
Q45. How were the drop heights selected?

A45. (AIS) The drop heights were selected at random, with the intent of determining the maximum height from which a cask could possibly drop. We understand that PFS is implementing design and procedural measures to limit the maximum height above the ground that a HI-STORM System cask will be lifted by a

transporter to 9 inches. Therefore, the 10 inch drop test represents a conservative upper limit to the potential accelerations to which a cask will be subjected in the event of a drop.

Q46. Please briefly describe the non-mechanistic, hypothetical cask tip-over analysis that Holtec performed for the PFSF.

A46. (AIS) Although it has been demonstrated that casks will not tip over under either the design-basis 2,000-year return period earthquake or a beyond-design basis, 10,000-year return period seismic event, a further "defense-in-depth" analysis has been performed to evaluate the results of a hypothetical cask tip-over event with the attendant impact of the cask on the pad. This analysis is summarized in the PFSF SAR Section 8.2.6. The HI-STORM System storage cask and a representative portion of the pad, soil-cement, and soil substrate were modeled to the extent required to accurately predict the post-impact system response. The primary objective of the hypothetical tip-over analysis was to demonstrate that the decelerations experienced by the fuel contained in the MPC are bounded by the design basis limits for fuel stated in the FSAR. This tip-over analysis showed that the maximum fuel deceleration is below the 45g. The tip-over finite element model used is shown below:



The results from the tip-over analysis, using three different cask concrete compressive strengths, showed that the 45g fuel deceleration limit was not reached. As in the case of the cask drop, staying within the 45 g limit ensures that, in reality, a very large safety margin exists against canister breach and potential releases of radioactivity.

Q47. How do these analyses relate, if at all, to the seismic cask stability analyses performed by Holtec for the PFSF?

A47. (AIS) There is no direct relationship between these hypothetical, postulated cases and the design basis stability analyses, which shows that the casks will remain stable and will not tip over even under ground motions well beyond those from the 2,000-year design basis earthquake for the PFSF. However, the non-mechanistic tip-over analyses evidence the availability of "defense-in-depth" margin with regard to the HI-STORM System to be used at the PFSF. They show that, if for any unspecified reason a cask were to tip over, the cask contents would retain its integrity and no release of radioactivity would occur.

IV. RESPONSE TO THE STATE OF UTAH'S CLAIMS IN SECTIONS C AND D OF THE UNIFIED CONTENTION

Q48. The State of Utah has raised various claims in Sections C and D of the Unified Contention concerning the adequacy of Holtec's cask stability, drop and tip-over analyses. Have you reviewed and analyzed the claimed deficiencies raised by the State in those sections of the Unified Contention?

A48. (KPS, AIS) Yes.

Q49. What claims raised by the State in Sections C and D of the Unified Contention will you be addressing in your testimony?

A49. (KPS, AIS) With respect to Section C, the only claim that we will be addressing is the claim in Section C.3.e concerning the Young's modulus that Holtec used in the cask drop and non-mechanistic tip-over analyses. In Section D, the various issues raised in Section D.1, "Seismic Analysis of the Storage Pads, Casks and Their Foundation Soils," either directly or indirectly relate in whole or in part to the cask stability analyses that Holtec performed for PFS. Accordingly, we will address each of the claims raised in Section D.1, although for certain of the claims (such as the claims in Sections D.1.a and D.1.d concerning non-vertically propagating waves), we rely upon the conclusions expressed in the testimony of Dr. Robert Youngs and Dr. Wen Tseng being filed simultaneously with this testimony.

Q50. What conclusion have you reached regarding the claims made by the State.

A50. (KPS, AIS) In Section C.3.e, the State has claimed that the cask drop and tip-over analyses that Holtec performed for the PFSF are not conservative since, in the State's opinion, the model used an unreasonably low soil modulus to characterize the soil stiffness. Contrary to the State's claim, Holtec used the correct modulus appropriate to a large strain condition in the soil foundation, in accordance with the NRC-approved methodology that has been benchmarked against test data. With regard to the State's contentions in Section D.1 that the stability analyses performed by Holtec are deficient, we will respond to those claims by

demonstrating the inherent conservatisms in our model, and provide a point-by-point refutation of the issues raised by the State.

A. Claim Raised by the State in Section C.3.e of the Unified Contention Concerning the Holtec Cask Drop and Hypothetical Cask Tip-over

Q51. Please describe the claims raised by the State in Section C.3.e of the Unified Contention concerning Holtec's cask drop and non-mechanistic tip-over analyses performed for the PFSF.

A51. (KPS, AIS) The State claims that PFS underestimated the dynamic Young's modulus of the cement-treated soil at the PFSF when subjected to impact during a cask drop or tip-over. Such underestimation, the State claims, significantly understates the impact forces on the cask and canister in those analyses.

Q52. What is the Young's modulus?

A52. (AIS) The Young's modulus is an elastic property of a material that is defined by a simple extension test; it is the ratio of the stress to which the material is subjected to the strain (deformation) that the material experiences as a result of the applied stress. The Young's modulus of a metal is a function of the properties of the metal, but is insensitive to strain level as long as no yielding occurs. The Young's modulus of a non-metallic material may, in addition, be dependent on the level of strain applied.

Q53. What is the significance of the Young's modulus to the cask drop and cask tip-over analyses?

A53. (AIS) As a HI-STORM System cask is dropped (or tips over) onto the concrete storage cask pad, some of the energy caused by the impact will be absorbed by the cement-treated soil and the underlying soil as strain (deformation). Because of the large magnitude of the forces (stress) caused by the impact, the level of strain that will be experienced by the cement-treated soil and the soil will be relatively large, and will depend on the value of the Young's modulus of the cement-treated soil and the soil at the point of impact.

Q54. The State's contention refers to a "dynamic Young's modulus." What does the term mean?

A54. (AIS) The term "dynamic Young's modulus" is somewhat of a misnomer. It really refers to the manner in which the Young's modulus is measured in a test, rather than to whether it represents a "dynamic" condition. A dynamic Young's modulus is one determined by a particular type of test in which a small amount of strain in the soil results from the passage of a wave front generated from a rather large stress (the setting off of explosives). On the other hand, a "static Young's modulus" is one measured in a test in which the type of test performed, such as moving a boring device some distance into the soil, requires relatively little force (stress) but produces a large deformation (strain) on the soil.

Q55. What is the relevance of the "dynamic" Young's modulus that the State claims should be used to the Holtec cask drop and tip-over analyses?

A55. (AIS) None. The proper concepts to apply in those analyses are those of "large strain" and "small strain" Young's modulus. Because the impact of the dropping or tipped-over cask on the underlying cement-treated soil will produce a large strain on the soil directly under the impact location (that strain is calculated in our drop and tip-over analyses as 1.93%), our analysis requires that a "large strain" Young's modulus be used. Such a large strain Young's modulus correlates well with the empirically-determined stress/strain relationships obtained in static tests. Therefore, it is appropriate for the Holtec analyses to be based on large-strain (i.e., "static") values of Young's modulus as opposed to small-strain (i.e., "dynamic") values.

It is important to note that 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) used "static" Young's modulus relationships and showed good correlations between those relationships and the results of actual drop tests of steel specimens on concrete pads on top of soil.

In short, the nature of the cask drop and tip-over analyses conducted by Holtec makes the use of a small strain "dynamic" Young's modulus inappropriate; instead, a large-strain, "static" modulus should appropriately be used. This is

what Holtec did and what the independent evaluations conducted for the NRC have confirmed to be correct.

B. Claims Raised by the State in Section D.1 of the Unified Contention Concerning the Holtec Seismic Cask Stability Analyses for the PFSF

1. Claims Raised in Section D.1.a of Unified Contention – Non-Vertically Propagating Seismic Waves

Q56. Please describe the claim raised by the State in Section D.1.a of the Unified Contention.

A56. (AIS) In Section D.1.a of the Unified Contention, the State claims that "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."

Q57. Do you know whether the seismic waves arriving at the foundations of the pads could arrive at an angle rather than vertically propagating, and if so, what effect, if any, that would have on the movement of the pad and casks?

A57. (AIS) Based on the testimony of Dr. Robert Youngs and Dr. Wen Tseng, we understand that the angles at which seismic waves would impinge the PFSF site are, for all practical purposes, vertical and that the rocking and torsional motion caused by the small angle of incidence from vertical of the waves arriving at the PFSF site would be insignificant. We also note that many of Holtec's analyses of cask stability consider cask arrays which, by design, provide an eccentric loading to the pad. One accepted methodology for bounding the effects of non-vertical seismic waves is, in fact, to deliberately induce a 5% loading eccentricity into the model to account for rocking and torsion effects. The very nature of the cases considered by Holtec introduces eccentricities into the model that are far in excess of those required to model the effects of non-vertical waves.

2. Claims Raised in Section D.1.b of Unified Contention – Pad Rigidity

Q58. Please describe the claim raised by the State in Section D.1.b of the Unified Contention.

A58. (AIS) In Section D.1.b of the Unified Contention, the State claims that the Applicant's calculations incorrectly assume that the pads will behave rigidly during the design basis earthquake and that this assumption of rigidity leads to "[s]ignificant underestimation of the dynamic loading atop the pads, especially in the vertical direction," and to "[o]verestimation of foundation damping."

Q59. Is it appropriate to model the concrete cask storage pad as a rigid body for purposes of Holtec's cask stability calculations?

A59. (AIS) Yes. We believe that the three-foot thick reinforced concrete cask storage pad can be modeled as a rigid body for purposes of Holtec's analysis of cask stability. No body is perfectly rigid. Therefore, whether the inherent flexibility of a body needs to be accounted for in analytical evaluations depends on the nature of the evaluation being performed. To take a simple example, consider a table with three legs, with a load placed somewhere on the table top. To predict what the load in each leg is, and whether the table will fall over, the table may be considered as a rigid body. On the other hand, if we wished to know how much the table top bends when the load is applied (assuming we show that it doesn't overturn), we must now consider the table top as a flexible body supported by the three legs.

The purpose of Holtec's cask stability calculation is to analyze the cask/pad interface in order to establish the interface forces and displacements between the cask and the pad. With respect to the characterization of these forces and displacements, a minor flexibility of the pad would produce only second-order effects that would not affect the validity of the results of Holtec's calculation. In reality, the large rigid casks, even though free standing, effectively confine the pad to a rigid motion under the casks' 11 ft diameter. In the 4 ft free space between casks (comparable to the thickness of the pad), there should be minimal flexible movements ascribed to the pad, since the free section of the plate has a thickness comparable to its free span between casks.

Q60. Has Holtec ever analyzed the potential effect of pad flexibility in its calculations of free standing casks on top of a concrete storage pad?

A60. (AIS) Yes. Holtec has included pad flexibility in its analysis of a pad proposed for the Tennessee Valley Authority's Sequoyah Nuclear Power plant. At the request of the client, Holtec performed analyses assuming that the pad was flexible. The pad was modeled with 16 casks in a square array, with the pad being approximately 64 ft. on each side and only 24 inches thick (compared to 36 inches for PFSF). The model was run for a fully populated pad with 16 casks and for the extreme case of a single cask located in one corner. Subsequently, the analysis was redone removing the flexible pad contributions from the model.

Comparison of the results from both analyses showed that the inclusion of pad flexibility produced only insignificant changes in the pad and cask movements and in the character of the interface force time histories used as input for the structural design and qualification of the pad.

Q61. What conclusions can be drawn from your analyses for the Sequoyah Nuclear Power Plant of the difference between modeling the pad as a flexible body and modeling it as a rigid body?

A61. (AIS) The results of our comparison for Sequoyah confirm that it is appropriate to treat the pad as a rigid body for characterizing the forces and displacements between the cask and the pad in evaluating the stability of the casks. We note that the same approach that was employed at PFSF (i.e., the use of a global dynamic analysis in which the pad is considered to determine the nature of the cask to pad interface forces and to prove cask stability, followed by a finite element analysis of the pad for pad design purposes that assumes the pad to be flexible) has been followed at all sites where Holtec has participated in the seismic/structural analysis of the cask storage pads.

Q62. The State claims that the results of Calculation No. 05996.02 G(P017)-2, "Storage Pad Analysis and Design" by International Civil Engineering Consultants ("ICEC") shows that the storage pads are not rigid and contradicts the assumption of pad rigidity in Holtec's analyses. Do you agree?

A62. (AIS) No. The ICEC calculation is directed toward calculating the detailed stress distribution within the pad subject to the interface force time history results determined from the global dynamic analysis. To determine pad stresses, one

must assume that the pad is flexible since there are no stresses developed unless one includes elasticity. However, what is a necessary assumption for a stress analysis is unnecessary for a global dynamic analysis. As long as the elastic deformations arising from the loads are small, the flexibility effect on the global dynamic solution can be ignored.

Realizing that every man-made structure has some flexibility, it is instructive to consider the following simple analogy: A grandfather clock has, as its basis for keeping time, the oscillation of a simple pendulum. Rigid-body dynamics establishes the relationship between the pendulum length and the time to complete one oscillation. Adjustment of this length allows one to ensure that the time is correct. However, to ensure that the pendulum is not overstressed during operation, the pendulum must also be treated as a flexible body subject to the applied loads from gravity, and the pendulum arm sized accordingly. The same situation applies with respect to the PFSF cask storage pads.

Q63. Please describe the computation of the foundation damping used in Holtec's cask stability analysis as it relates to the issue of pad rigidity.

A63. (AIS) The Holtec dynamic model incorporates the effect of the foundation by using a set of six springs and associated dampers in series with the springs. These springs and dampers were defined, based on the material properties provided by Geomatrix (lower range, best estimate, and upper range properties, based on a weighted average over a 30 ft depth below the pad, including the effect of soil cement). The soil springs and dampers were defined by Holtec using the applicable formulas in ASCE 4-86, which assume that the pad acts like a rigid body.

Q64. Does the assumption of pad rigidity lead to overestimation of foundation damping as claimed in Section D.1.b(ii) of the Unified Contention?

A64. (AIS) No. Based on our evaluation of the effect of pad flexibility for Sequoyah, any effect of pad flexibility on foundation damping would be minimal. This is confirmed by the testimony by Dr. Wen Tseng being filed simultaneously with this testimony. Dr. Tseng testifies that the pad behaves as a rigid body insofar as

it affects foundation damping in the frequency range of interest here. Therefore, treatment of the pad as rigid does not lead to an overestimation of foundation damping as claimed by the State in Section D.1.b(ii).

Q65. The State claims in its Response to Applicant's Eighth Set of Discovery Requests relating to its claim under Section D.1.b(ii) of the Unified Contention that Holtec's use of a 5% Beta damping coefficient is too high. Does Holtec's use of a 5% Beta damping coefficient in any way relate to the State's claim of overestimation of foundation damping based on asserted flexibility of the pad?

A65. (AIS) No. The "Beta damping" factor accounts for the energy loss during a vertical impact between cask and pad. It relates to the damping that Holtec used in modeling the compression-only springs at the interface of the cask and the pad. A damping element in parallel with the compression spring (between the pad's upper surface and the base of the cask) is incorporated to account for this energy dissipation mechanism. Such damping has no relationship to the damping values for the soil underlying the pad.

Q66. The State also claims in its Response to Applicant's Eighth Set of Discovery Requests that the asserted flexibility of the storage pad violates Holtec's assumption "that a uniform coefficient of friction exists between the bottom of the casks and the top of the pad." Do you agree?

A66. (AIS) No. Our assumption as to the coefficient of friction between the casks and the pads does not depend on whether the pads are flexible or rigid. Nor did we assume that the coefficient of friction at the cask-pad interface would be uniform (a single value) as claimed by the State. Rather, the coefficient of friction will vary between two moving objects regardless of whether the bodies are rigid or flexible. To account for the effect of expected variations due to surface effects, we performed our cask stability analyses at both an upper and a lower bound coefficient of friction to envelop the effects of this potential variation. We discuss the State's claims concerning the proper coefficient of friction further in the context of Section D.2.c(iii) of the Unified Contention.

3. Claims Raised in Section D.1.c of the Unified Contention – Evaluation of Potential Storage Pad Motion in Relation to Sliding of the Casks on the Pads

Q67. Please describe the claims raised by the State in Section D.1.c of the Unified Contention.

A67. (AIS) The State claims in Section D.1.c of the Unified Contention 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 the 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 to local deformation of the pad at the contact points with the cask.

Q68. Did Holtec perform an analysis to show the relationship between the potential sliding of the foundation storage pads and the sliding and tipping of casks on the storage pads?

A68. (AIS) Yes. Holtec performed an analysis for PFS of the relationship between the potential sliding of the pads and the sliding and tipping of the casks on the pads. Our analysis is summarized in a August 6, 2001 letter from Holtec to PFS, which PFS forwarded to the NRC Staff by letter of August 7, 2001. The two letters are collectively identified as PFS Exhibit NN.

Q69. Please describe the analysis performed by Holtec.

A69. (AIS) As discussed in the testimony of Mr. Paul Trudeau being filed simultaneously with this testimony, the storage pad will not undergo sliding under the 2,000-year design-basis earthquake. Therefore, sliding of the pads is a beyond design basis event and Holtec's analyses of the effect of sliding of the pads were performed only to demonstrate the conservatisms in the PFS design basis.

Nonetheless, to simulate the potential effect of a postulated sliding of the pad relative to the foundation, the design basis simulation model was altered to replace the three translation soil springs beneath the storage pad (one vertical

spring to simulate tension-compression resistance and two orthogonal lateral springs to simulate the shear resistance from the underlying soil/soil-cement) with a vertical compression-only spring and two orthogonal horizontal friction springs. The vertical compression-only spring represents the resistance to movement due to the normal downward force of the loaded storage pad and the orthogonal horizontal friction springs represent the resistance to movement due to friction between the pad and the soil (for which a coefficient of friction of 0.306 was used). Holtec analyzed three cases, each having eight casks on the pad and assuming a coefficient of 0.80 between the cask and the pad. The only difference between the three cases was the damping associated with the vertical compression only spring and the two orthogonal horizontal frictions springs. Case 1 assumed the damping values used in the original simulation, Case 2 assumed damping values reduced to 10% of the values used in Case 1, and Case 3 assumed damping values reduced to 1% of the values used in Case 1.

Q70. What were the results of your analysis?

A70. (AIS) The results of calculation showed that sliding of the pad dramatically reduces the movement of the cask. For example, whereas the maximum cask lateral excursion, relative to the pad, in the original model simulation was on the order of 3 to 4 inches, for Case 2 of the simulation -- where sliding of the pad of was less than four inches -- the maximum excursion for the casks, relative to the pad, did not exceed 0.02 inches. Thus, sliding of the pad even a few inches reduces the maximum excursion of the cask relative to the pad by more than two orders of magnitude.

Q71. Is this result consistent with what one would expect based on general physics considerations?

A71. (AIS) Yes. As discussed in PFS Exhibit NN one would expect as a general matter that sliding of the pad would reduce the seismic energy transferred to the casks, and therefore decrease the motion of the casks relative to the pad. Indeed, this is the theory behind base isolation design of structures or buildings to protect them from earthquake damage. The base on which the building or structures rests

is designed to freely move with the earthquake such that the forces of the earthquake are not transferred to the building or structure. Therefore, insofar as cask stability is concerned, pad sliding is a favorable occurrence.

Q72. Did you take into consideration the behavior and effect of the soil cement in your pad sliding analyses?

A72. (AIS) No. Since our analysis was a simplified analysis intended only to demonstrate the general effect of sliding of the storage pads on cask motion, Holtec did not take into consideration the effect of soil cement or any other material (e.g., soil) around the pad. Such effects would have not altered significantly the results of the analysis.

Q73. Would the presence of soil cement around the pads result in unbalanced (unsymmetrical) loadings on the pads once the pads undergo sliding movement?

A73. (AIS) There could be some minute effects due to thin cracks in the soil cement, which I understand from the testimony of Mr. Paul Trudeau could occur. However, even assuming (unrealistically) that all these cracks between pads were aggregated into a single gap between the soil cement and one of the pads, the maximum size of the gap, according to Mr. Trudeau, would be on the order of ½ inch. Assuming such a gap, oscillations of the pad under earthquake motions could then involve some mild impacts if the pad were to bump against the soil cement. But, any such impact would be small because earthquake motions rapidly change direction, so the pad and the soil cement often would be moving in the same direction and would not collide. In the analysis Holtec performed, it was considered appropriate to neglect the effect of the soil cement adjacent to each pad as it would likely be negligible.

Q74. To what extent would such unsymmetrical loadings as you just described affect the stability of the pads and casks?

A74. (AIS) If one postulated that gaps of the size I just described were present, and further postulated that the pads did slide under the design basis seismic event, there would be additional lateral restraint forces coming into play to resist further movement of the pad on each cycle. On the one hand, this postulated closure of

the soil cement pad gap would lead to horizontal impacts not included in the current analysis; however, on the other hand, the same non-linear effect would be accompanied by an additional energy absorption not currently included in the analysis. On balance, it is our opinion, based on engineering judgment and experience with a large number of similar simulations involving horizontal rack-to-rack impacts, that the sum total of the effects would result in minimal changes to the results of the existing analyses.

Q75. In his deposition, State witness Dr. Ostadan claimed that Holtec improperly failed to take into account the soil cement in its analysis under design basis conditions, in which Holtec assumes that the pad does not slide. According to Dr. Ostadan, the oscillations of the pad, even though not sliding would be out of phase with the oscillations of the soil cement resulting in the soil cement and the pads bumping against each other as they oscillate.[1] Is this aspect of the State's claim, as articulated by Dr. Ostadan, realistic?

A75. (AIS) No. The potential impacts referred to by Dr. Ostadan would be even less than those discussed above because the movement of the pads under purely oscillatory motion with no sliding would be even less than those occurring if the pad were to slide. Therefore, any loads resulting from the abutment of the pads and the soil cement would continue to be negligible and would not affect the conclusions from the analysis. Indeed, Dr. Ostadan acknowledges that the effect of any pad to soil cement interaction would be small and that he raises the issue because of the allegedly "slim margins" against sliding present in the design.

Q76. In Section D.1.c(ii), the State again takes issue with your treating the pad as rigid, claiming your analysis does not take into account "the flexibility of the pads under SSE loading." Do you understand the State to raise any issues different here than those you already responded to with respect to the State's claims in D.1.b. of the Unified Contention?

A76. (AIS) No.

Q77. In section D.1.c(iii), the State claims that, in evaluating the motion of the casks sliding on the pad, Holtec failed to take into account the variation of the coefficient of sliding friction between the bottom of the casks and the top of the pads due to local deformation of the pad at the contact points with the cask. What is your response to this claim?

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[1] Deposition of Farhang Ostadan ("Ostadan Dep.") (March 8, 2002) at 143.

A77. (AIS) The interface between the cask and the pad consists of the steel surface of the bottoms of the HI-STORM System casks and the concrete surface of the storage pads. In our cask stability analysis for the PFSF, Holtec evaluated the potential for casks to tip over and for casks to impact each other by sliding. We analyzed the stability of the casks at two bounding coefficients of friction, a lower bound coefficient of 0.2 and an upper bound coefficient of 0.8. The analysis at the lower coefficient of friction of 0.2 emphasizes the potential for the casks to slide and impact each other on the concrete pad. The analysis at the higher coefficient of friction of 0.8 emphasizes the possibility for cask tip-over.

The chosen values of 0.2 and 0.8 effectively bracket the expected range of the coefficient of friction for the interaction of a steel-bottomed cask with a concrete pad. Typical upper and lower bounds for the coefficient of friction given by various handbooks for metal on concrete/stone surfaces range between 0.3 to 0.7. See, e.g., Mark's Standard Handbook for Mechanical Engineers 3-22 (Eugene A. Avallone & Theodore Baumeister, III, eds., 10th ed. 1997) (coefficient of friction for iron on stone – 0.3 to 0.7); Harry Parker and James Ambrose, Simplified Mechanics and Strength of Materials 34 (5th ed. 1992) (coefficient of friction for metal on stone, masonry, or concrete – 0.3 to 0.7). The value of the lower coefficient of friction analyzed by Holtec of 0.2 is less than the lower bounds cited by these handbooks, and the value of the higher coefficient of friction analyzed by Holtec of 0.8 is greater than the upper bounds from these handbooks.

Thus, Holtec did not assume that the coefficient of friction would be a single value. Rather, it assumed a lower bound coefficient of friction and an upper bound coefficient of friction such that its analyses would bracket the range of coefficients of friction that one would expect for a free-standing steel surface on a concrete pad. This approach is consistent with the analyses performed by Holtec for spent fuel storage racks in spent fuel pools.

Q78. The State also contends in its Response to Applicant's Eighth Set of Discovery Requests that because of the asserted flexibility of the storage pad that the "sliding resistance will

not be constant due to local deformations of the surface of the pads resulting from inertial loadings imposed by the casks." What is your response to this claim raised by the State?

A78. (AIS) As stated above, Holtec did not assume that the sliding resistance would be a single, constant parameter but chose an upper and lower bound for the coefficient of friction in its analyses to emphasize the potential for sliding or tipping. Use of this procedure has been accepted by the regulating body as appropriate in the many license submittals for Holtec's spent wet storage fuel racks, where we used an upper bound coefficient of friction of 0.80 and a lower bound of 0.20.

Nor will small pad deformations adversely affect the sliding of the casks as asserted by Dr. Ostadan in his deposition. As set forth in the testimony of Dr. Wen Tseng, ICEC has calculated that the maximum local deformations sustained by the pad under the design basis earthquake due to the dynamic forces of the casks are on the order of 1/8 of an inch. Such small deformations would not occur as sharp ridges, but would develop gradually over many feet. Such negligible deformations create neither a depression nor a ridge in the pad that would have any perceptible effect on the sliding of a 19 ft high, 360,000 lb cylindrical cask, with a diameter of 11 ft and bottom surface area of 95 square feet.

4. Claims Raised in Section D.1.d of the Unified Contention – Lateral Variations in the Phase of the Ground Motions

Q79. Please describe the claim raised by the State in Section D.1.d of the Unified Contention.

A79. (AIS) In Section D.1.d of the Unified Contention, the State claims that the "Applicant has failed to consider lateral variations in the phase of ground motions and their effect on the stability of the pads and casks."

Q80. What is your understanding of this claim?

A80. I understand from Dr. Ostadan's deposition and the testimony of Dr. Youngs and Dr. Tseng that this claim is essentially the same claim as raised in Section D.1.a

of the Unified Contention, which, as discussed above, is addressed in the testimony of Drs. Young and Tseng.

5. Claims Raised in Section D.1.e of the Unified Contention – Frequency Dependency of Soil Spring and Damping Values

Q81. Please describe the claims raised by the State in Section D.1.e of the Unified Contention.

A81. (AIS) In Section D.1.e of the Unified Contention, the State claims 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."

Q82. What is your response to this claim?

A82. (AIS) The terminology associated with "frequency dependency" arises from the formulation and solution of a linear problem in the frequency domain (as opposed to a solution in the time domain). The problem of free-standing casks on a pad is a non-linear problem; as such, the only correct methodology to use is a time domain solution. The design basis methodology employed by Holtec for the PFSF cask seismic stability simulations is, (correctly) time-domain based.

The soil spring, masses, and dampers derived by Geomatrix from its analyses incorporate the fundamental frequency of the soil foundation (predominantly 1 to 5 Hz). While there may well be some higher order frequency contributions, their effects on the cask responses will be secondary since the cask response to the earthquake (i.e., amplitude of excursion vs. time) is primarily at or below 5 Hz. Thus, if the soil's spring-mass-damper model used as the design basis input was replaced by a model involving multiple masses, springs, and dampers to incorporate effects of higher order frequency "bumps" in the spectra (if indeed, any such bumps were identified), the response of the casks would not be significantly altered and, certainly, the conclusions concerning overall stability of the casks would remain unchanged.

Q83. In the statements supporting this claim, the State witnesses assert that, "[b]ecause the cask-pad-soil-cement is a non-linear analysis, it is very important to consider all potential variation in the motion of the casks and the pads. If the casks and the pads move out-of-

phase significant instability conditions may arise." To what extent does Holtec's casks stability analysis assume that the casks and pads will move in phase?

A83. (AIS) Holtec's cask stability analysis makes no *a priori* assumption concerning how the casks will move in relation to one another or in relation to the pad. The dynamic simulations performed by Holtec assume only that the casks and the pad are initially at rest at their respective locations, under a 1g gravitational loading, that the cask-to-pad interface has a dynamic coefficient of friction equal to either the upper or lower bound value, and that the coefficient of friction value remains constant through the seismic event's duration. There is no assumption of phasing imposed at the start of the time history simulation and there are no constraints imposed on the cask behavior at any point in the simulation.

Q84. Would out of phase motion between the casks and pads result in underestimating the potential instability of the casks, as claimed by the State?

A84. (AIS) The cask responses in each of the dynamic scenarios exhibit various degrees of phasing between the dynamic responses of each cask; however, this phasing is the solution from the dynamic analysis, not an imposed condition. We note that, even if the responses of adjacent casks were to be constrained to be completely out of phase in the analytical simulation, the magnitudes of the displacements at the top of the cask, resulting from the design basis 2,000-year return period, are such as to ensure large margins of safety against cask overturning and cask-to-cask impact. As discussed above, the maximum displacements are less than 3.25 inches, which is much less than 50% of the approximate 4 to 5 foot spacing between the casks on the pad.

Q85. What do you therefore conclude with respect to the State's claim in Section D.1.e. of the Unified Contention?

A85. In Holtec's opinion, the State's claim has no merit. Even assuming potential underestimation of the effect of out-of-phase motion of the casks, given the large cask-to-cask spacing at the PFSF and the large margins provided by the design against overturning and cask-to-cask impact, any such underestimation could not affect the results of the final analyses.

6. Claims Raised in Section D.1.f of the Unified Contention – Cold Bonding

Q86. Please describe the claims raised by the State in Section D.1.f of the Unified Contention.

A86. (AIS) In Section D.1.f of the Unified Contention, the State claims that 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."

Q87. What do you understand cold bonding to be?

A87. (AIS) We understand cold bonding to be a mechanical process wherein two bodies in contact, under a large pressure at their interface, develop a certain capacity to resist relative sliding. For example, titanium plates are often cold bonded to carbon steel plates by detonating an explosive charge which exerts a large interfacial pressure resulting in a bonding between the two plates. An essential precondition for cold bonding is the existence or application of a large interface pressure.

Q88. Will cold bonding develop over time between the casks and the pad as alleged by Dr. Ostadan?

A88. (AIS) No. The upper bound weight of a cask is 360,000 lb. The average pressure developed at the interface to support this weight is equal to the 360,000 lb of the cask divided by the area of the interface between the cask and the pad – i.e., the area of the bottom of the cask. Based on a 132.5 inch diameter cask, the average pressure at the interface is approximately 26 psi. We recognize the pressure distribution is not uniformly distributed and that higher pressures will exist around the periphery than at the center. But, even if we were to consider the entire load to be supported only over a 12" wide annulus around the periphery, the static contact pressure would rise only to 40 psi. This level of pressure is comparable to a 200 lb man standing on the ball of one foot. It is fair to assume that in such a situation the man would not become bonded to the concrete. In short, the large weight of the cask has no significance here, given the absence of a

large interfacial pressure. There will be no bonding between the steel bottom of the cask and the concrete surface of the pad.

Q89. In responding to PFS's request to identify and fully describe each respect in which the PFS has failed to consider the potential for cold bonding between the casks and the pads, the State responded in part as follows (State's Response to Interrogatory No. 11 in Applicant's Eighth Set of Interrogatories):

> Holtec's design of the casks assumes that the casks will slide on the pad in a controlled in-phase manner during a large earthquake without excessive sliding, pounding or tipping .... However, such a bold design concept could be negated by the potential for cold bonding between the casks and the pad that may develop over time.

Does Holtec in any respect assume, as claimed by the State, that the casks will slide on the pad "in a controlled in-phase manner during a large earthquake without excessive sliding, pounding or tipping?"

A89. (AIS) No. We have previously described how Holtec modeled and performed the cask stability simulations. The Holtec analyses make no assumptions concerning cask phasing. The response of the casks, relative to one another, is an output from the simulations, not an input constraint. Sliding is not controlled in any manner by the solution methodology.

Q90. The State goes on to assert in the same answer to Interrogatory No. 11 answer that "[w]hen two bodies (cask and pad) with such a large load (the cask) are in contact, some local deformations and redistribution of stresses may occur at the points of contact which would create a bond, and thus would not allow the cask to slide on the pad or move smoothly during an earthquake and thus negate the design concept." What is your response to these assertions of the State?

A90. (AIS) The coefficient of friction between two bodies may vary over time due to the direction of relative motion at the interface and other factors. However, the average coefficient of friction obtained from a statistically significant set of measurements will yield a generally acceptable result for engineering analyses of performance and response over time. It is precisely because of the uncertainties involved with coefficients of friction that the PSFS analyses evaluated scenarios using acceptable upper and lower bound values for the coefficient of friction. While using an intermediate value or even some randomly varying value (over

time) will lead to different results, using as we did upper and lower bound coefficients of friction for the design basis solutions does provide appropriate bounding results.

Q91. If the casks do not slide smoothly, will there be greater loadings on the casks than assumed by Holtec?

A91. (AIS) As noted in the previous response, any small perturbations in the cask response due to irregular sliding would be within the range of results encompassed by the design basis simulations.

Q92. In his deposition, Dr. Ostadan claimed that a practical consequence of cold bonding was that the coefficient of friction between the cask-pad interface would be 1.0. Would using a coefficient of friction of 1.0 change the results of your analysis?

A92. (AIS) If we hypothesized as a bounding scenario a coefficient of friction of 1.0 (rather than 0.8), our results could be somewhat altered, but the overall conclusions would not be altered. The reason is that, as a practical matter, the upper bound coefficient of friction that we used of 0.8 is already set high enough to favor tipping of the cask. Potential cask tip-over would be essentially the same at a coefficient of friction of 0.8 as it would at a coefficient of friction of 1.0.

7. Claims Raised in Section D.1.g of the Unified Contention – Failure to Analyze for Pad-to-Pad Interaction in PFS's Sliding Analysis of the Storage Pads

Q93. Please describe the claims raised by the State in Section D.1.g of the Unified Contention.

A93. (AIS) In Section D.1.g of the Unified Contention, the State claims that 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."

Q94. Do your cask stability analyses incorporate the effects of potential pad-to-pad interactions?

A94. (AIS) No. Holtec evaluated the possibility of pad-to-pad interactions and concluded that any such interaction would have only second-order effects that

would not affect the validity of the calculations. Accordingly, Holtec did not incorporate pad-to-pad interaction effects into its analysis.

Q95. How did you evaluate the effect of pad-to-pad interactions, and on what basis did you conclude that they would be second order effects?

A95. (AIS) Based on the calculated pad movements for both the 2,000-year and 10,000-year return period earthquakes, it was our engineering judgment that any resistance from the soil cement between pads would not affect the system response in any material manner.

Q96. What effects may the nearest of the pads to one another (five feet apart) have on Holtec's cask stability analysis?

A96. (AIS) The potential effects for pad-to-pad interaction are essentially discussed in our responses to Section D.1.c. where we discussed the effects of potential loads caused by the pad collisions with the adjoining soil cement.

Q97. Does Holtec treat the soil cement as a reinforced concrete mat in its cask stability analysis?

A97. (AIS) No. The soil cement and the soil layers underlying the soil cement are modeled by six linear springs (three translation and three rotation springs); the magnitudes of these six springs are a function of a soil foundation modulus (averaged over a thirty foot depth) and the geometry of the pad. Formulas to derive the spring constants are obtained from industry standards (e.g., ASCE-4-86) and include the contribution of the soil cement layer under the pad.

Q98. If the cement-treated soil, soil-cement and storage pads for ten rows of pads did not behave as an "integrated unit," would that affect Holtec's cask stability calculations?

A98. (AIS) The cask stability analyses performed by Holtec do not rely on the cement treated soil, or soil cement for 10 rows of pads behaving as an "integrated unit". Therefore, such behavior, or lack of same, would not alter our results.

8. Claims Raised in Section D.1.h of the Unified Contention – Use of One Set of Time Histories

Q99. Please describe the claims raised by the State in D.1.h of the Unified Contention.

A99. (AIS) In Section D.1.h of the Unified Contention, the State claims that the use of one set of time histories in Holtec's cask stability analysis is inadequate because (i) nonlinear analyses such as Holtec's are sensitive to the phasing of input motion and more than one set of time histories should be used, and (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.

Q100. What has been Holtec's experience with the number of sets of time histories used in the non-linear analyses for free-standing nuclear spent fuel components?

A100. (KPS, AIS) As discussed previously, in addition to Holtec's work in the area of dry cask storage, Holtec has also been a major supplier of wet storage (spent fuel racks) technology to the nuclear power industry. Since 1986, Holtec has made a large number of licensing submittals to the NRC and other agencies and had also prepared such documents for utilities evaluating the potential for increasing their wet storage capacity. Holtec's practice has been to follow NRC guidance on the number of sets of time histories that should be used in dynamic analyses of SSCs important to safety.

Q101. What has been the NRC guidance on the number of sets of time histories that should be used in dynamic seismic analyses?

A101. (AIS) The generation of time histories for use in dynamic simulations is discussed in NUREG-0800, Standard Review Plan ("SRP") 3.7.1. Revision 1 of this document, issued in July 1981, simply stated that the response spectra re-generated from the artificial time histories should envelop the design response spectra (with limited exceptions) at the same location for all damping values actually used in the analysis. The practical effect of requiring the design response spectra generated from the time histories to envelope the original earthquake response spectra is that the design response spectra will on average be larger than the earthquake response spectrum. Therefore, this process generally results in amplitudes of the generated design response spectra that are conservative compared to the original earthquake response spectra.

Revision 2 to the SRP was issued in August 1989. This Revision introduced two options for the use of artificial time histories in analysis: Option 1 allowed the use of a single time history (the same as Revision 1), except that in addition to enveloping the original response spectra, a regenerated Power Spectral Density ("PSD") distribution also had to be shown to adequately match a target PSD compatible with the original response spectra. A PSD is a measure of the energy contained in the earthquake as a function of the frequency range, and the requirement to adequately match a target PSD compatible with the original response spectra was intended to insure that all significant energy was captured by the derived artificial time histories and that no important frequency ranges containing peaks in the PSD function were missed.

Option 2 allowed the use of multiple time histories. The SRP recommended a minimum of four time history sets, but specifically provided that each individual set did not have to envelop the target response spectra. Also, Option 2 did not impose any requirement to match a target PSD compatible with the earthquake response spectra.

Although the SRP guidance provided two options, it provided no guidance on when these differing options should be implemented. Neither Option 1 nor Option 2 is restricted to linear or non-linear problems when artificial time histories are considered.

Q102. How did Holtec's practice of generating and using one or more sets of time histories for its non-linear analyses evolve in relation to the change in guidance in the NRC SRP?

A102. (AIS) A partial list of Holtec's licensing submittals and/or plant requested analyses appears below. The list contains, in the final column, whether the seismic inputs involved: a single time history or multiple time histories. As can be seen by examination of this table, Holtec generally followed Revision 1 of the SRP, and then Option 1 of Revision 2 through 1992. However, in the 1993-1994 time period, as a general matter, Holtec followed Option 2 of Revision 2 of the SRP and utilized multiple sets of time histories for its non-linear analyses of spent

fuel storage systems. After 1994, Holtec generally returned to using single sets of time histories.

Plant Name	Date	NRC Docket #	# of Time Histories
Diablo Canyon Unit I & II	1986	50-275 50-323	Single (3 Different Spectra)
St. Lucie Unit No. I	1987	50-335	Single
Byron Units I & II	1987	50-454 50-455	Single
Chin Shan	1988	-	Single
Vogtle	1989	50-425	Single
Millstone Unit I	1989	50-245	Single
Ulchin Unit II	1989	-	Single
Kuosheng	1989	-	Single
Indian Point Unit II	1990	50-247	Single
Laguna Verde	1990	-	Single
J.A. FitzPatrick	1990	50-333	Single
Three Mile Island Unit I	1990	50-289	Single
D.C. Cook	1992	50-315 50-316	Single
Fort Calhoun Station	1992	50-285	Single
Hope Creek	1992	50-354	Single
Zion Station	1993	50-295 50-304	Single
Salem Generating Station	1993	50-272 50-311	Multiple
Sequoyah Unit I	1993	50-327	Multiple

and Unit II		50-328	
Beaver Valley Power Station Unit I	1993	50-334	Multiple
Fort Calhoun Station	1993	50-285	Multiple
Duane Arnold Energy Center	1994	50-331	Single
Duane Arnold Energy Center	1994	50-293	Multiple
Limerick	1994	50-352 50-353	Multiple
Ulchin Unit I Spent Fuel Pool Capacity Expansion	1994	-	Single
Kori-4 & Yonggwang Units I & II	1995	-	Single
Comanche Peak	1995	50-445 50-446	Single
Connecticut Yankee Spent Fuel Pool	1996	50-213	Single
Ulchin Unit 2 Spent Fuel Pool	1996	-	Single
Watts Bar – TVA	1996	50-390	Single
Vogtle	1997	50-424	Single
Diablo Canyon Power Plant	1997	50-275 50-323	Single
Callaway and Wolf Creek	1998	50-483 50-482	Single
Chinshan Unit I & II	1998	-	Single
Waterford 3	1998	50-382	Single
Vermont Yankee	1998	50-271	Single
J.A. FitzPatrick	1998	50-333	Single

Kuosheng Unit I & II	1999	-	Single
Oyster Creek	1999	50-219	Single
Byron/Braidwood	1999	50-456/457 50-454/455	Single
Harris	1999	50-400	Single
Yonggwang	1999	-	Multiple
Millstone Unit 3	2000	50-423	Single
Fermi Unit II	2000	50-341	Single
Edwin I. Hatch Nuclear Plant Unit I & II	2000	50-321/366	Single
Davis Besse Unit I	2001	50-346	Single
Kewaunee	2001	50-305	Single
Nine Mile Point Unit II	2001	50-410	Single
Virgil C Summer	2002	50-395	Multiple

Q103. Was Holtec's change to multiple sets of time histories in the 1993-94 time frame or its return to a single set of time histories in the 1995 timeframe mandated in any respect by the NRC?

A103. (AIS) The changes were not mandated by the NRC in any formal written document. It is our collective recollection that the original change from a single to multiple time histories was motivated both by our client's wishes and the NRC staff suggestions to conform to the latest revision of the applicable SRP. However, in the 1995 timeframe, the NRC staff reviewers dealing with wet storage suggested that we return to the use of a single time history with the added requirements of adequately matching the PSD.

Q104. What are the advantages and disadvantages of the two methodologies as applied to the free-standing spent fuel storage casks modeled by Holtec?

A104. (AIS) The use of a single time history set constructed according to the SRP 3.7.1 guidelines ensures that the time history will generate a set of enveloping response spectra. The requirements of SRP 3.7.1 for use of a single set of time histories

would lead to an "average" re-generated spectra set. On the other hand, the use of multiple histories may capture additional phasing effects. Based on the geometry and size of the pads and the testimony of Dr. Youngs, we do not believe that the phasing issue is of importance at the PSFS site. Our analysis of cask stability is most affected by the input seismic amplitudes. The single time history procedure is more likely to ensure that maximum amplitudes and proper frequency content are captured and utilized in the seismic design of the PFSF.

Q105. What has been Holtec's practice with respect to the number of sets of time histories it used since the NRC provided the option of using single or multiple sets of time histories?

A105. (AIS) Since that time, Holtec has used a single set of spectrum-compatible time histories for its analysis of free standing spent fuel racks and dry cask storage systems, unless directed otherwise by the client.

Q106. Based on the results of your dynamic analyses for the PFSF and your previous experience, can you draw any conclusion concerning the sensitivity of your non-linear cask stability analysis for the PFSF 2,000-year design basis earthquake to the phasing of input ground motions and whether considering additional sets of time histories with different phasing might affect the results of your analysis?

A106. (AIS) On the basis of the above-discussed results of our analyses, one would expect that use of different sets of time history inputs might alter individual results, but not the final conclusions.

Q107. Do you know whether the set of time histories for the current 2,000-year design basis earthquake that you used in your cask stability analyses incorporated what is known or referred to as fault fling?

A107. (AIS) Based on the testimony of Dr. Robert Youngs, we understand that the set of time histories for the 2,000-year design basis earthquake that we used in our cask stability analysis incorporated fault fling.

Q108. Do you have an opinion of whether a different set of time histories incorporating fault fling would affect the results of your cask stability analysis for the PFSF 2,000-year design basis earthquake?

A108. (AIS) Based on the testimony by Dr. Youngs, we would expect different results from different time history sets, independently of the inclusion of fault fling.

However, our opinion is that for the same seismic input strengths, use of one or more sets of time histories, with or without incorporating fault fling, would not alter the basic result. The casks would remain upright and would not impact each other.

Q109. What conclusion do you draw on about the State's claimed need for additional sets of time histories?

A109. (AIS) Holtec's cask stability analyses are based on the use of a time history set that ensures bounding of the design basis response spectra and the power spectral density functions in accordance with SRP 3.7.1, Option 1. The design basis time histories do include fault fling. The level of cask response from the current 2,000-year return period design-basis seismic input ensures that there is a large margin of safety against cask tip-over and/or cask-to-cask impact. While use of different time histories will give different response levels, the margins of safety that exist based on the current design basis results lead us to conclude that there is no merit to the State's claimed need for additional time histories. In Holtec's opinion, any such temporal differences in the cask excursions would be small and would not compromise the conclusion that the casks would remain stable.

Q110. Do you know what process other vendors used for their wet storage submittals?

A110. (AIS) Our knowledge of other cask vendors submittals is limited. However, we have some information on Westinghouse spent fuel rack analyses for San Onofre Units 2 and 3 (circa 1990) and Westinghouse spent fuel rack analyses for Comanche Peak (circa 1994). Both of these analyses used a single set of time histories. For San Onofre, time histories bounding the response spectra were developed without a corresponding comparison of the PSD function. The later submittal, for Comanche Peak, developed time histories that bounded the response spectra and produced re-generated PSD functions in accord with the latest version of SRP 3.7.1.

9. Claims Raised in Section D.1.i of the Unified Contention

Q111. In Section D.1.i of the Unified Contention, the State claims that the because of the alleged errors and omissions and unsupported assumptions asserted in Sections D.1.a through D.1.h of the Unified Contention, "the Applicant has failed to demonstrate the stability of the free standing casks under design basis ground motions" and thus has failed to show that "excessive sliding and collision will not occur or that the casks will not tip over" as required by 10 C.F.R. § 72.122(b)(2). What is your response to the State's claim?

A111. Holtec has examined the cask response at PFSF for different magnitude design basis seismic events and accompanying input soil properties. A multitude of cask arrays on the pad have been considered, which provided both symmetrical and asymmetrical loads on the pad. Under all conditions, including an evaluation of pad sliding on the foundation, the results from the analyses have demonstrated that casks will not overturn nor will adjacent casks impact one another. The methodology employed to obtain the results is based on a time-domain solution of the governing equations of motion and considers each cask on the pad as a free standing body. There are no constraints imposed on the behavior of casks during the seismic event. No assumptions on in-phase or out-of-phase motion are required, and both upper and lower bounds on friction coefficients between casks and pad are employed to ensure that uncertainties in the instantaneous contact behavior at the cask/pad interface would be encompassed by the totality of simulations. Based on the totality of results and on the large margins of safety against tip-over and impact, we conclude that the requirements of 10 C.F.R. § 72.122(b)(2) and NUREG-1536 at 3-6 have been achieved by the analyses performed. We also note that confirmatory independent calculations have been performed by Sandia Laboratories for the NRC. These confirmatory calculations, performed using a finite element code and including pad flexibility and explicit representation of the soil layers, confirmed that for the parameters considered, the levels of cask response from the Sandia analyses and from the Holtec analyses were in good agreement, and that no adverse effects on the stability of the casks would be experienced under design basis earthquake loadings.

Q112. Have you performed any additional analysis to evaluate the various claims raised by the State in Section D.1?

A112. (KPS, AIS) Yes. Holtec performed additional computer simulations to evaluate certain other issues raised by the State. The State's witnesses have challenged the modeling of the soil/soil cement foundation under the pad and the level of damping that can be ascribed to the soil. Our simulation have confirmed that these concerns are unfounded.

Q113. What computer code did you use for these additional analyses?

A113. (KPS AIS) Holtec used the VisualNastran ("VN") code that it had previously used for evaluating the beyond-design basis 10,000-year earthquake. Holtec used the VN code because it conducted most of the additional analyses at the 10,000-year earthquake level. VN is better able to model large rotations of the cask that would be expected to occur under the 10,000-year earthquake event.

Q114. Please describe the issues raised by State that were addressed in the additional analyses.

A114. (KPS, AIS) The State has raised three general issues regarding the previous cask stability analyses that Holtec performed for the PFSF. These are as follows:

1. The State asserts that the 2,000-year design basis earthquake is inadequate in some respects, such as non-vertically propagating waves or lack of sufficient time histories that would increase the strength of that earthquake and adverse affect cask stability.

2. The State argues that pad flexibility significantly affects the level of damping provided by the soil foundation during a seismic event, and that PFS has overestimated the amount of soil damping available to inhibit seismic response of the casks; and

3. The State hypothesizes that the soil frequency response may actually be "in-tune" or, "in resonance" with the major energy producing frequency of the input seismic event and alleges that PFSF has not included this "resonance" potential in its model, leading to an underestimate of the amplification that may be imposed on the pad.

Q115. How do the additional analyses address the State's concerns regarding the adequacy the seismic input for the 2,000-year design basis earthquake?

A115. (KPS, AIS) Our analyses generally used a 10,000-year return period earthquake as the ground motion input so that there are no issues on whether our analyses use a bounding input. We do, however, include some analyses using the 2,000, year return period seismic event in order to demonstrate the dramatic difference in the results when the only change is the input driving function; and, to provide an independent check, using an entirely different computer code, that the level of response predicted from DYNAMO is in fact correct. The new analyses use a bounding seismic event whose strength, as measured by the peak ground acceleration, is far in excess of the 2,000-year return period design basis seismic event and would bound, by virtue of the increased strength, any issues raised by the State concerning the appropriateness of PFS's evaluation of the response to the 2,000-year design basis earthquake.

Q116. How do the additional analyses address the State's claim that Holtec's assumption of pad rigidity results in overestimation of soil damping?

A116. (KPS, AIS) The State's concern is addressed by arbitrarily imposing a low level of soil damping that provides a conservative lower bound on the level of damping actually expected in the soil. For a conservative simulation that minimizes the effect of soil damping, we conservatively choose the soil damping to be a low value of 1% of critical damping (as defined for a 1-degree of freedom mass-spring-damper system); for example, commensurate with an appropriate choice of the spring constant, the soil damper, C, in parallel with that spring is computed from the formula:

$$C = 2 \times (0.01) \times (k_0 \times (W/g))^{1/2}$$

Q117. How do the additional analyses address the State's concerns regarding "potential resonance effects"?

A117. (KPS, AIS) To determine the effects of "in-tune" or "resonance" increases in pad motion, we extended the simulation model used in the previous 10,000 year return period analysis to include the entire 30' x 67' pad, a simulation of soil springs displacement, and the rotation resistance provided by the foundation between the input motion and the pad. The simulation also included multiple

casks on the pad. In this analysis, each cask is modeled as a rigid cylinder weighing 360,000 lb. The pad is modeled as a rectangular solid having a total weight consistent with that of a 3' thick concrete pad, and the effect to the soil substrate is modeled by three linear springs and three rotational springs and associated dampers in parallel with the springs.

A major source of input energy from the seismic event occurs in the vicinity of 5 Hz. Therefore, in many of the beyond-design bases bounding analyses performed in these new simulations, we have used the total vibrating mass (pad plus one or eight casks), and defined linear springs so that the mass-spring system has a resonant frequency of 5 Hz in order to show maximum "in-tune" or "resonance" effects.

The resonant soil properties are defined as follows:

For a given total problem mass (i.e. 30'x67'x3' slab + 8 casks), determine the vertical and horizontal linear soil spring constants to have a resonance at 5 Hz.

$k_0 = (W/g) \times (2\pi f)^2$

f=5, W= weight of entire slab + weight of total number of casks on pad.

With the total stiffnesses proportional to slab displacement chosen, these springs can be distributed over the pad interface area, and then the net rotational resistance about the three centroidal axes of the slab can be defined, providing the definition of the three rotational stiffness values. These stiffnesses are assumed to be positioned at the slab/soil interface.

We are thus able to choose two sets of stiffnesses that ensure a resonance effect for the case of one cask or eight casks on the pad. As noted, damping is chosen at 1% of critical based on the spring constant determined and the vibrating weight.

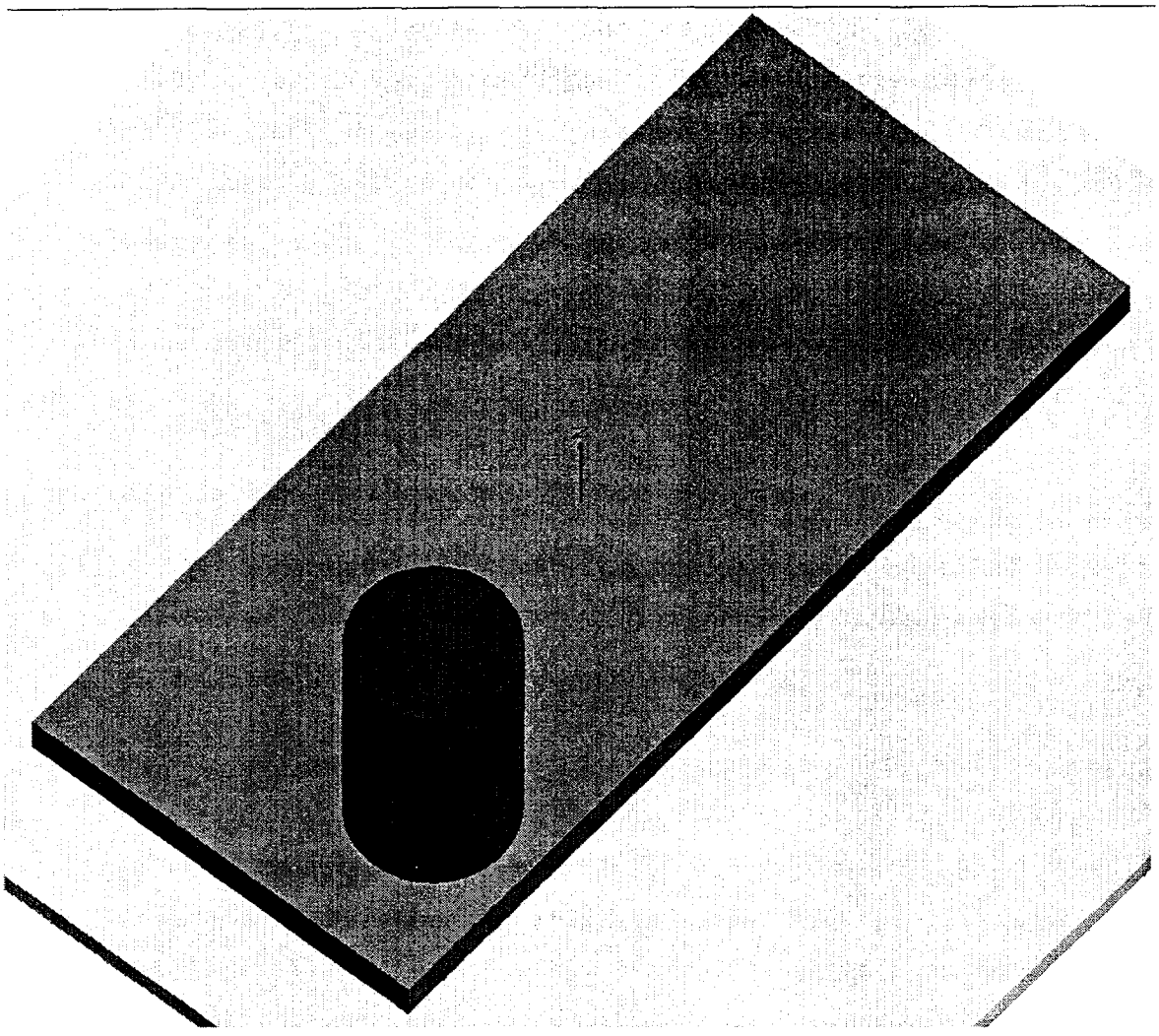
Q118. Please describe each of the analysis performed to address the State's concerns.

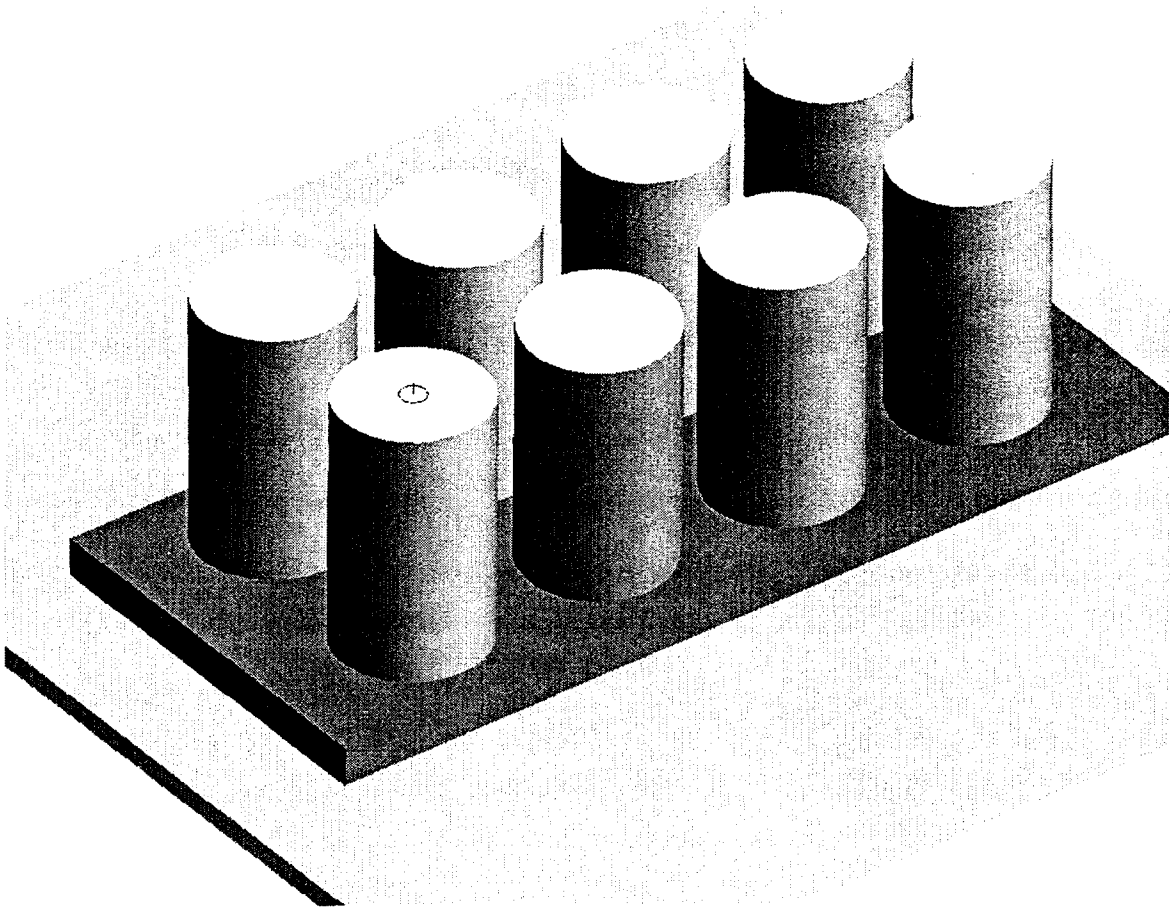
A118. (KPS, AIS) The table below describes the complete set of additional cask stability analyses performed in support of this testimony. For clarity, two 3-D graphics are included from the VN simulation. The graphics show the extreme cases modeled – one and eight casks on the pad. The soil springs between the pad and the reference plane are not depicted in either graphic.

SUMMARY OF VISUALNASTRAN ANALYSES

Case # - Description	Event	Stiffness	Damping	COF	Remarks
1. – 8 casks	2k	Lower Bound design basis	Lower bound design basis	.8	J;lkj;lkj;lkj;l Demonstrate agreement with DYNAMO results
2. – 8 casks	2k	Resonance @ 5 Hz	1%	.8	Evaluate effect of "tuning" soil springs and low damping
3.-1 cask on pad	10k	Based on mass of 1 cask + entire pad oscillating at 5Hz	1%	.8	Lowest stiffness that gives 5 Hz tuning
4. – 1 cask on pad	10k	Based on mass of 1 casks + entire pad oscillating at 5Hz	5%	.8	Check damping effect
5.-3 casks on pad	10k	Based on mass of 1 casks + pad @ 5 Hz	1%	Random between 0.2 and 1.0	Check sliding
6.-3 casks on pad	10k	Based on mass of 1 cask + entire pad oscillating at 5Hz	1%	.8	Intermediate loading with low stiffness
7.- 4 casks on pad	10k	Based on mass of 1 cask + entire pad oscillating at 5Hz	1%	.8	Intermediate loading with low stiffness
8.- 8 casks on pad	10k	Based on mass of 8 casks + entire pad oscillating at 5Hz	1%	.8	Fully populated with tuned stiffness and damping
9.- 8 casks on pad	10k	Based on mass of 8 casks + entire pad oscillating at 5Hz	1%	0.2	Fully populated with tuned stiffness and damping
10. – 8 casks on pad	10k	Based on mass of 8 casks + entire pad oscillating at 5Hz	1%	Random between 0.2 and 1.0	Fully populated with tuned stiffness and damping – evaluation of the effect of real behavior of friction between casks and pads
11. – 8 casks on pad	10k	Geomatrix Lower Bound Values consistent with 10k	Geomatrix Lower Bound Values consistent with 10k	.8	Design basis equivalent of 2k event

Notes: Horizontal shear springs chosen = to vertical spring. Then values are divided by 8 and an individual vertical and two horizontal springs located under each cask so as to define the applicable rotational resistance.





Q119. Please describe the results of your analyses.

A119. (KPS, AIS) The results from each simulation are summarized as a computer animated video, viewable with Windows Media Player. These animated simulated effects form an integral part of the report summarizing each of the models and the resultant effects. These video files are on a single CD-ROM identified as PFS Exhibit MM.

Q120. Please summarize verbally, what these computer modeling simulations demonstrate.

A120. (The animation illustrates the following conclusions of the analysis:

(1) The results of the VN simulation using a 2,000-year return period event and the lower bound set of soil stiffness and damping elements, agree with the results predicted by DYNAMO. To the extent that there may be differences, these are due to the fact that

VN recomputes the equilibrium equations at each instant in time and accounts for the changes in orientation (even though they are small) throughout the entire run duration. DYNAMO, by contrast, uses the original equilibrium equations and does not update them continuously. Thus, the results from VN more accurately display slightly larger rotations than those predicted from DYNAMO if the rotations reach the upper end of "small rotations".

(2) The VN simulations using the 10,000 year return period event experience significant rocking behavior and out of phase motion of the casks when the coefficient of friction is 0.8. At certain instants, some casks impact each other with the net result that one of the two casks involved in the impact, slows down almost completely for a period of time following the contact.

(3) For coefficients of friction of 0.2, the casks move in phase and there are no contacts between casks.

(4) No overturning of any cask was experienced in any of the analyses.

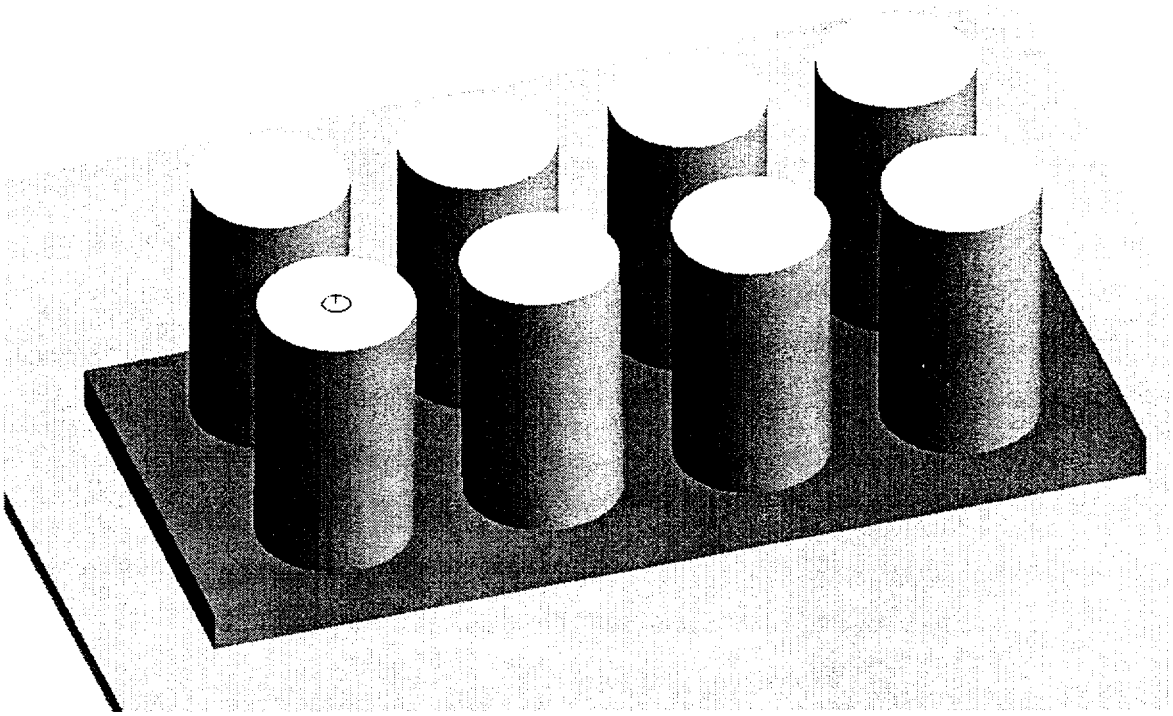
(5) Random coefficients of friction reduced the rocking behavior of the casks.

(6) While there was some effect on the system behavior due to "tuning" the stiffness values to match a input seismic frequency, the major contribution to the large motions was the earthquake strength.

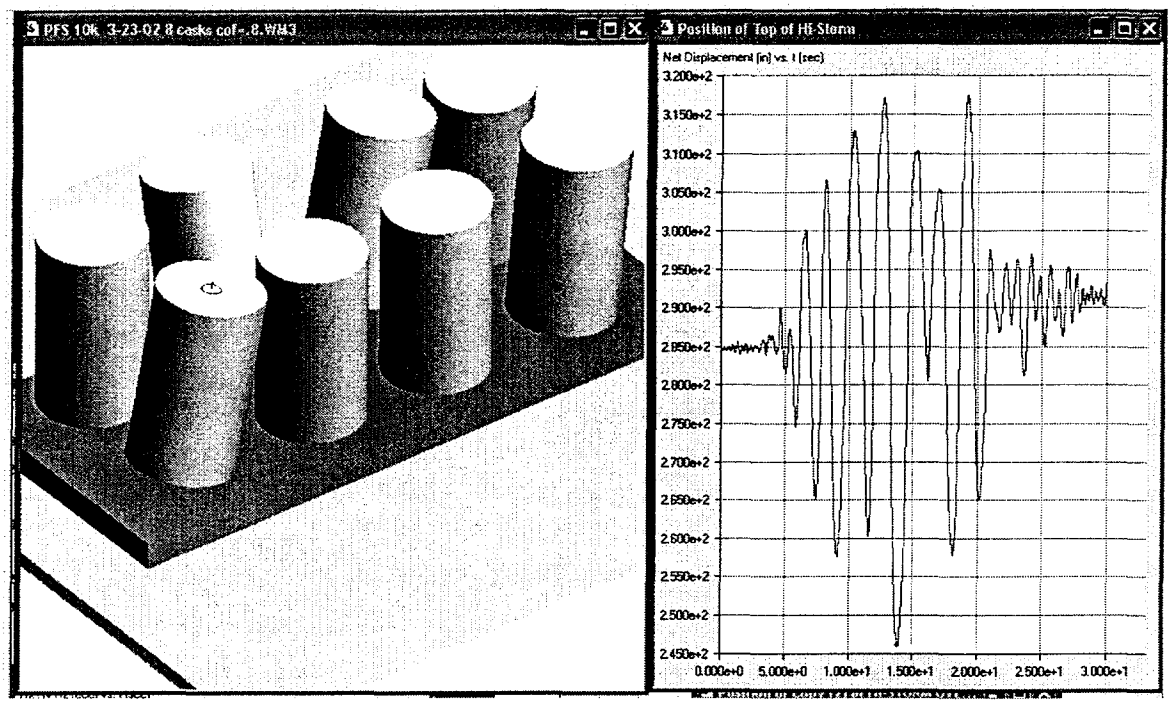
(7) The use of conservatively low soil damping values, while increasing the cask response, does not lead to a condition where severe pad oscillations occur.

(8) Maximum excursions of the pad horizontally are generally below 0.5".

The following figure shows the configuration of the Case 1 (current design basis) at an instant when maximum movement of any cask on the pad is observed. Because the movement is relatively small, only close observation of cask #3 reveals that it has the most deviation from vertical. There is no significant out-of-phase motion apparent throughout the entire duration of the design basis event.



In contrast to the above figure, the following figure shows the nature of the results from Case 8 where the 10,000-year return period seismic event is driving the system and conservatively "tuned" soil stiffness and 1% soil damping is assumed. It is very clear in this figure that the casks are experiencing large motions, with a significant contribution from out-of-phase effects. A plot of the net displacement of Cask #1 (the closest corner cask to the reader) shows the extreme position of this location as a function of time. Despite the orientations observed, at the end of the simulation, all casks are in their original vertical orientation, although perhaps, as can be seen in the graph, in a new location (the final rest position of this cask is approximately 8" from where it started).



Q121. What do you conclude from this additional study?

A121. (KPS, AIS) The additional analyses were performed using input values for earthquake, soil stiffness, and soil damping that was chosen to maximize any deleterious effects (as opposed to using expected real-world values). The results of these analyses shows that none of the State's claims have any merit. It is our opinion that the bounding simulations performed here demonstrate that the casks and the storage pad, under worst-case scenarios, show no significant detrimental effects that would lead to cask tipover. Accordingly, these recent analyses reconfirm our conclusion that the HI-STORM System will exhibit satisfactory performance at the design basis earthquake, and demonstrate capability of the HI-STORM System to withstand much larger earthquake events, up to and beyond the 10,000-year return period earthquake.

V. OTHER EVALUATIONS OF CASK STABILITY AT THE PFSF

A. Overview and Summary

Q122. Please identify what other analyses you have reviewed concerning the stability of the HI-STORM 100 casks at the PFSF.

A122. (KPS, AIS) We have reviewed and evaluated a cask stability analysis performed on behalf of the State of Utah by Dr. Moshin Khan of Altran Corporation, entitled "Analytical Study of HI-STORM 100 Cask System for Sliding and Tip-Over Potential During High-Level Seismic Event." The report is identified as Altran Technical Report No. 01141-TR-001, Revision 0, prepared for the Office of the Attorney General, State of Utah, dated December 11, 2001 ("Altran Report"). We also reviewed an earlier version of this Report dated November 30, 2001 filed by the State of Utah as part of its December 7, 2001 Opposition to PFS's Motion for Summary Disposition of Utah L, Part B (now Section E of the Unified Contention). We have also reviewed a report prepared on behalf of the NRC Staff by Sandia Laboratories, and other technical consultants, entitled, "Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage Facility" ("Sandia Report"), dated March 8, 2002.

Q123. Have you performed other activities in connection with your evaluation of the Altran Report?

A123. (KPS, AIS) In addition to reviewing the Altran Report, we also attended the March 5, 2002 deposition of Dr. Khan at which he explained various aspects of his analysis. We also performed various calculations to test what results his model would provide in standard problems whose solution is well known, to test the validity of the model used by Dr. Khan in the analysis described in the Altran Report. We have also reviewed pertinent information in the Finite Element Analysis (FEA) literature concerning the modeling of contact problems. Finite Element Analysis is a numerical approach to the solution of complex problems in structural analysis (and other fields). It required the development of computers to make the technique viable. Essentially, the continuum is broken into a large number of manageable elements where the displacement shape may be assumed. Continuity equations ensure that the elements are tied together properly, and the computer solves the large number of equations that ensue.

Q124. What are your conclusions from your evaluation of Dr. Khan's methodology and the results set forth in his report, as further elaborated on at his deposition?

A124. (KPS, AIS) Based on our review of Dr. Khan's work and the additional items performed as noted above, we conclude that Dr. Khan's work comes to erroneous conclusions because he has not achieved the correct, converged solution for many of his simulations, and has utilized unrealistic and unsupportable inputs for the simulations. Because his input values are unrealistic, they lead to non-converging solutions from which he draws improper conclusions on the behavior of the HI-STORM System casks.

Q125. What were the results of your review of the Sandia Report?

A125. (KPS, AIS) We concurred with the reasonableness of the model described in the March 8, 2002 report submitted to the NRC by Sandia Laboratories. We reviewed the results obtained by Sandia Laboratories for the cases and seismic events considered; on the basis of our review, we concluded that, although there are differences in the models used in the Sandia and Holtec analyses, the conclusions were in agreement. In fact, we view the Sandia results as confirmation that Holtec's assertions on the absence (or lack of effect) of pad flexibility and the applicability of soil springs in the dynamic analyses are reasonable and proper.

B. REVIEW AND EVALUATION OF ALTRAN REPORT ON CASK STABILITY AT THE PFSF

Q126. Please describe your major areas of disagreement with the Altran Report, as elaborated on by Dr. Khan at his deposition.

A126. (KPS, AIS) The major areas of our disagreement are: (1) Dr. Khan uses a model for his analysis that – unlike Holtec's model – has not been validated to show that it correctly models, and provides good solutions to, standard problems for which the correct solutions are known; (2) Dr. Khan fails to follow established guidance for developing inputs for key parameters used in his model and instead assumes values for key input parameters that provide unrealistic and physically impossible answers to real life situations; (3) Dr. Khan misinterprets results from his analyses for which his model has clearly failed to produce a correct solution (i.e., very large horizontal movements, way out of proportion to the strength of the input)

and claims his results to represent accurate solutions. Because of these errors in Dr. Khan's analysis, his results are meaningless and therefore, the conclusions he draws from them are faulty; and (4) Dr. Khan's criticisms of Holtec's model are invalid and based on a misunderstanding of the inputs used in the Holtec model.

1. Lack of a Validated Model

Q127. Please describe the models that Dr. Khan used to evaluate HI-STORM System cask stability at the PFSF.

A127. Dr. Khan uses three models. His initial model is a simple mass weighing 360,000 lb that can slide and uplift. Dr. Khan used this simple mass model in an attempt to benchmark his analysis code, SAP2000, by running the model on both ANSYS (another general purpose industry computer code) 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 stimulates 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]. The last two models were run on SAP2000, which is a general purpose structural program that can be adapted to stimulate a wide range of problems. For these last two models, 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.

Q128. Had Dr. Khan ever constructed such a model before?

A128. No. Dr. Khan acknowledged that this was the first time that he had ever attempted to model the movement of a large free standing object, such as the HI-STORM System. See Deposition of Dr. Moshin Khan, March 5, 2002 (Khan Dep.") at 143 (identified as PFS Exhibit PP.) In addition, instead of using a specialized computer code that was tailored for the features of the PFSF cask/pad/soil configuration, Dr. Khan attempted to adapt a general purpose

structural program, SAP2000, to model the free-standing HI-STORM System cask on a storage pad, something he also had never done before.

Q129. What steps did Dr. Khan take to attempt to validate his model?

A129. (KPS, AIS) 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. He did not attempt to validate any of his three models in any other manner. In particular, he did not attempt to compare the solutions derived from simulations using his models with known classical solutions, as required by the NRC. (As noted earlier, Holtec has performed thorough, successful validations of its DYNAMO code and has had the code and its results approved by the NRC in numerous dockets).

Q130. Did Dr. Khan's running his simple mass model on two different general purpose computer codes prove the validity of simple mass model, or that of the other two models that he used?

A130. No. It 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 model output. As Dr. Khan readily acknowledged at his deposition, the same wrong input parameters to both would lead to equally erroneous result for both. Khan Dep. at 77. While his two solutions show good agreement with each other, the modeling itself is clearly erroneous, and leads to results that defy physical reality. Using his model with some of the key parameters applicable to the PFSF cask stability analysis -- coefficient of friction of 0.2 and a mass of 360,000 lb -- the mass should begin to slide at a horizontal load equal to F=0.2 x 360,000 lb. = 72,000 lb. However, his simple model predicts that if we apply a force of 71,000 lb., just below that force required to initiate sliding of the block, this 360,000 lb. mass (equal to the mass of a fully-loaded HI-STORM System cask) would move - without sliding -- more than 2/3 of an inch. There is no physical mechanism for this phenomenon to occur in the real world. Because his model is the same for both computer codes, his validation succeeds only in showing that both computer codes give the same

spurious answer. In short, the "validation" Dr. Khan claims to have accomplish fails to validate the adequacy of his model or demonstrate the suitability of his analysis of the stability of the Holtec HI-STORM System cask.

Q131. In the joint declaration describing his model and criticisms of the Holtec model filed by the State in Opposition to PFS's Motion for Summary Disposition on Part B of Utah L (now Section E of the Unified Contention), ("Utah Joint Declaration") Dr. Khan states that both "SAP2000 and ANSYS have been benchmarked with known analytical solutions to provide adequate results for dynamic analyses." Is the comparison between SAP2000 and ANSYS that you just described sufficient to validate or benchmark Dr. Khan's model for analyzing the dynamic motion of a free-standing spent fuel storage cask on a storage pad?

A131. (KPS, AIS) No.

Q132. Why not?

A132. (KPS, AIS) For the same reason as we stated above, comparing results from two computer codes simply proves that the code algorithms produce similar results to similar inputs. In the final analyses, even if the code's algorithims are appropriate, the codes will only give an answer that is as good as the input provided. To properly validate a friction model for a free standing structure, it is necessary to check the model you propose against a known analytical solution or against experimental results. The ANSYS FEA Code, for example, provides a suite of verification problems to demonstrate that the ANSYS Code can reproduce the solutions to well-known problems. Indeed, ANSYS provides verification for modeling contact stiffness that shows how to correctly solve such a problem. Dr. Khan did not follow this ANSYS guidance; instead, the simple mass model he used was not verified and predicts an incorrect and non-sensical solution for a simple problem. Had Dr. Khan studied the simple problem considered in the ANSYS verification manual, he most likely would have realized his error in utilizing unreasonably "soft" stiffness values.

Q133. Unlike Dr. Khan's model, has Holtec's model been validated and benchmarked for analyzing nonlinear dynamic solutions?

A133. (KPS, AIS) Yes, as stated above, the Holtec program was validated, using various benchmarking problems, in a manner consistent with ASME NQA-2a-1990, Part 2.7, "Quality Assurance Requirements of Computer Software for Nuclear Facility Applications." The Validation Manual for the Holtec Code DYNAMO (also referred to as "DYNARACK") was prepared many years ago and has been continuously updated, most recently in 1998. The validation is equally applicable to both wet and dry storage applications.

Q134. What computer code validation requirements does ASME NQA-2a-1990 impose?

A134. (KPS, AIS) ASME NQA-2a-1990 mandates that a computer code be benchmarked against classical solutions and peer computer codes to the extent possible using appropriately selected test problems so as to establish the suitability and stability of the code for the genre of problem being analyzed. In accordance with the ASME requirements in this respect, DYNAMO has been specifically validated using problems that test its ability to predict the dynamic behavior of free-standing bodies in the presence of friction. Of pertinent interest here is one of the test problems used to benchmark DYNAMO, which deals with static and sliding friction and is a published paper in the Journal of Applied Physics, Volume 21, Number 9, September, 1953 (Static and Sliding Friction in Feedback Systems, by J. Tou and P.M. Schultheiss) (which is identified as PFS Exhibit QQ). As shown from the portion of the Validation Manual for DYNAMO identified as PFS Exhibit RR , DYNAMO correctly predicts the solution for this classical test problem. Dr. Khan's model does not.

2. Failure to Follow Authoritative Guidance in Developing Contact Stiffness Input Parameters and Choosing Contact Stiffness Input Parameters, Resulting in Unrealistic And Physically Impossible Solutions to Real Life Situations

Q135. You stated earlier that Dr. Khan failed to follow authoritative guidance with respect to key input parameters and chose key input parameters that provide unrealistic and physically impossible solutions to real life situations. What key input parameters were you referring to?

A135. (KPS, AIS) The key parameters that we were referring to were the values for choice of contact stiffnesses between the HI-STORM System storage casks and the concrete storage pads on which they rest. There are two such stiffness parameters, a vertical contact stiffness parameter and a horizontal contact stiffness parameter. Dr. Khan's major criticism of Holtec's cask stability analysis is Holtec's choice of these contact stiffness parameters. However, the values that Dr. Khan recommends for these parameters are both contrary to authoritative guidance and produce results that are contrary to the laws of physics.

Q136. Would you please explain what is meant by contact stiffness?

A136. (KPS, AIS) 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 to approach the other a unit distance. The parameter is measured in the pounds of force required to cause one body to approachthe second body by one inch. For example, assume that you have two pads made of undefined materials and you place on each pad a loaded HI-STORM System cask weighing 360,000 lbs. Assume for Pad Material 1 that the HI-STORM System cask would move towards the pad by 1.0 inches, at and that for Pad Material 2, the HI-STORM System cask would move toward the pad by only 0.01 inches. With respect to Pad Material 1 you would say that the vertical contact stiffness of the material would be 360,000 lbs. per 1 inch, or 0.36×10^6 lbs. per inch. For Pad Material 2, the vertical contact stiffness would be 36×10^6 lbs. per inch, since placement of the cask on the pad caused a movement of only .01 inch. The numbers for both of these examples can be derived from this simple formula:

$K = W/d$ where W is the vertical load applied (in this example, the weight of the cask), d is the average deformation under the cask (assumed rigid for the purpose of this discussion), and K is the contact stiffness (in this case, based on known weight and measured information on the deformation of the cask "into" the pad.

Q137. How is the vertical contact stiffness used in modeling the motion of a large free standing object, such as the HI-STORM System cask?

A137. (KPS, AIS) It is used to define the stiffness of the vertical-only "compression springs" at the interface of the cask and the pad that are used in the dynamic modeling of cask motion on the pad.

Q138. What vertical contact stiffness did Holtec use in its modeling of the HI-STORM casks for the PFSF and in what respect does Dr. Khan's differ?

A138. (KPS, AIS) In the design basis analysis for the 2,000 year return period earthquake using Holtec's computer code DYNAMO, Holtec used a vertical contact stiffness of 454,000,000 lbs per inch or 454 x 10^6 lbs/in. Dr. Khan, however, claimed to be doing a parametric study on the effect of choice of contact stiffness on the solution, and ran his models using a range of contact stiffnesses. According to Dr. Khan, Holtec's choice of a vertical contact stiffness of 454 x 10^6 lbs/inch is too high. He claimed instead that, "[b]ased on [his] experience, it is [his] opinion that a more appropriate contact stiffness value for unanchored casks is 1 x 10^6 lbs/inch." Utah Joint Declaration ¶ 67. However, as already noted, Dr. Khan acknowledged that he did not have any experience in modeling the motion of large free standing bodies, and his choice of contact stiffness is contrary to ANSYS guidance on choosing an appropriate contact stiffness.

Q139. Where does one find the authoritative guidance that you claim that Dr. Khan failed to follow in developing contact stiffnesses for modeling purposes?

A139. (KPS, AIS) Such authoritative guidance is typically found in user manuals for the various computer codes that can be used for modeling. In fact, one of the computer codes used by Dr. Khan, ANSYS, has extensive guidance on how to develop contact stiffness for modeling purposes.

Q140. What about Dr. Khan's claim in his deposition that ANSYS doesn't provide detailed guidance on choosing of contact stiffness?

A140. (KPS, AIS) Dr. Khan is wrong. The Verification Manual provided by ANSYS contains a number of sample problems covering friction and contact issues. Additionally, the ANSYS Advanced Contact and Bolt Pretension, Training Manual and Workshop Supplement (Version 5.6) contains more than 100 pages devoted almost entirely to friction and contact problems, several of which are

reproduced and identified as PFS Exhibit SS. It is made eminently clear there that in order to achieve realistic modeling, the choice of contact and friction springs should not imply a "measurable" penetration or elastic movement prior to sliding. If this occurs, then the stiffness should be increased.

Q141. Please elaborate on this guidance provided by ANSYS.

A141. (KPS, AIS) ANSYS in essence says that "although physical contradicting bodies do not interpenetrate" some "finite amount of penetration" is required to mathematically model the contact between bodies. It therefore states that "[m]inimum penetration gives best accuracy" and that, "[t]herefor, the contact stiffness should be very great." However, it notes that too stiff a value may cause difficulty in having model converge to a solution and determining the stiffness value "usually requires some experimentation." It clearly states, however, that "if you can visually detect penetration . . . the penetration is probably excessive" and one should "[i]ncrease the stiffness and restart." Thus, the general guidance provided by ANSYS is that minimum penetrations, denoting large contact stiffnesses, give the best accuracy.

Q142. Given that Dr. Khan used ANSYS to run his models, did he follow this guidance from ANSYS on how to develop an appropriate contact stiffness?

A142. (KPS, AIS) No. He was apparently unaware of, or disregarded, the guidance provided by ANSYS. When questioned at this deposition, Dr. Khan testified that "ANSYS never provided any guidance on sliding, how to calculate the stiffness for a sliding problem," and that "there is no guidance from ANSYS how to solve a nonlinear sliding problem with large horizontal motions." Khan Dep. at 168-69.

Q143. Is Dr. Khan's choice of 1×10^6 lbs/inch for modeling the seismic response of HI-STORM 100 at PFSF in accordance with the guidance from ANSYS?

A143. (KPS, AIS) No. Dr. Khan's choice violates the fundamental precept of the ANSYS guidance, i.e., that there should be no visible interpenetration of the two objects. Using the same formula that we set forth above, the penetration or deflection can be computed as follows:

$$D \text{ (deflection or penetration)} = \frac{\text{Weight in lbs. (W)}}{\text{Contact stiffness in lbs/inch (K)}}$$

Applying this formula to Dr. Khan's professed "appropriate choice" of contact stiffness leads to a contact interpenetration of approximately 3/8 of an inch, just due to placing the cask on the top surface of the pad. This is computed as follows:

D = (360,000 lb.) / (1,000,000 lb./inch) = 0.36 inch

We have previously calculated the pressure placed by a fully loaded HI-STORM cask on the pad to be 26 psi, which is less than a man standing on the ball of one of his feet. To say that the cask placing that little pressure on the concrete pad would interpenetrate the pad by 3/8 of an inch defies physical reality and common, everyday experience. Objects do not sink into concrete pads just by being placed on them. Dr. Khan's choice or contact stiffness is also directly contrary to the guidance provided by ANSYS that "if you can visually detect penetration . . . the penetration is probably excessive."

Q144. Did Holtec develop the contact stiffness that it used in its cask stability analysis in a manner consistent with the guidance from ANSYS and other available authoritative sources?

A144. (KPS, AIS) Yes. Holtec seeks to use contact stiffness values that produce very small interpenetrations, but yet permit the code to achieve a converging solution. While we may draw upon known physical solutions to obtain a specific value of contact stiffness (i.e., examine some relevant classical solutions), any choice of stiffness we make in real cases must give meaningful results. For example, the Holtec choice of stiffness of 454,000,000 lb./inch used in the DYNAMO model was based on a result from a classical solution of a rigid body on a half space. However, the real reason we used that value is not that it comes from a classical solution, but that the static penetration of a HI-STORM System cask into the

concrete predicted using that value for stiffness is d=360,000 lb/454,000,000 lb./inch = 0.00008," an acceptable, realistic prediction. In our latest analyses for the beyond-design basis 10,000-year return period earthquake, we used an equally valid rationale for the choice of contact stiffness; namely, for a simple vertical vibration of the cask, we set the stiffness so that it was consistent with the assumption that the lowest frequency of vibration was 33 Hz. This requirement yielded a vertical stiffness value of 40,130,000 lb/inch. This different value, however, also met the test of "no visible penetration" as formulated in the ANSYS guideline manual, for it yielded an interpenetration d=360000 lb/40,130,000 lb./inch = 0.009", a value sufficiently low to be deemed to be acceptable.

Q145. You appear to have made your choices of vertical contact stiffness values on the basis of some physical principle. Is there any guidance on the appropriateness of doing so?

A145. As stated earlier, the underlying rationale is one of providing no "visible" interpenetration when you place the bodies in contact; to the extent that the value can be chosen from the solution of a physically relevant problem that satisfies the primary test, that is a "plus".

Q146. You stated earlier that there was also a horizontal contact stiffness parameter. What does this parameter measure?

A146. (KPS, AIS) This parameter measures the force at the point of contact between two bodies in the horizontal direction that causes a relative deflection of 1 inch in the horizontal direction between two originally coincident points on the interface.

Q147. Does Dr. Khan's model use reasonable values of horizontal contact stiffness?

A147. (KPS, AIS) No. Dr. Khan assumes that the force in the horizontal direction required to cause a relative deflection of 1 inch in the horizontal direction is 100,000 lbs/in. and that the cask will slide at a coefficient of friction of 0.20. If you apply a force greater than the 20% of the weight of the cask, or 72,000 lb, the cask will slide; a force below 72,000 lb should impart no visible relative movement in the horizontal direction. But if we use Dr. Khan's horizontal

stiffness value and apply just 71,000 pounds of force on the cask in the horizontal direction, the cask should not slide, yet Dr. Khan's model predicts a "visible" horizontal deflection of 0.71 inches, which again defies physical reality.

Q148. Have you done any other evaluations of the capability of Dr. Khan's model to correctly predict solutions to classical problems?

A148. (KPS, AIS) Yes. We have evaluated the capability of Dr. Khan's model to correctly predict the classical problem discussed by J. Tou and P.M. Schultheiss in the Journal of Applied Physics, Volume 21, Number 9, September, 1953 (Static and Sliding Friction in Feedback Systems). We had previously noted that in benchmarking DYNAMO that DYNAMO had correctly predicted the solution for this classical test problem. The classical solution and the Holtec simulation results are included in PFS Exhibits QQ andRR.

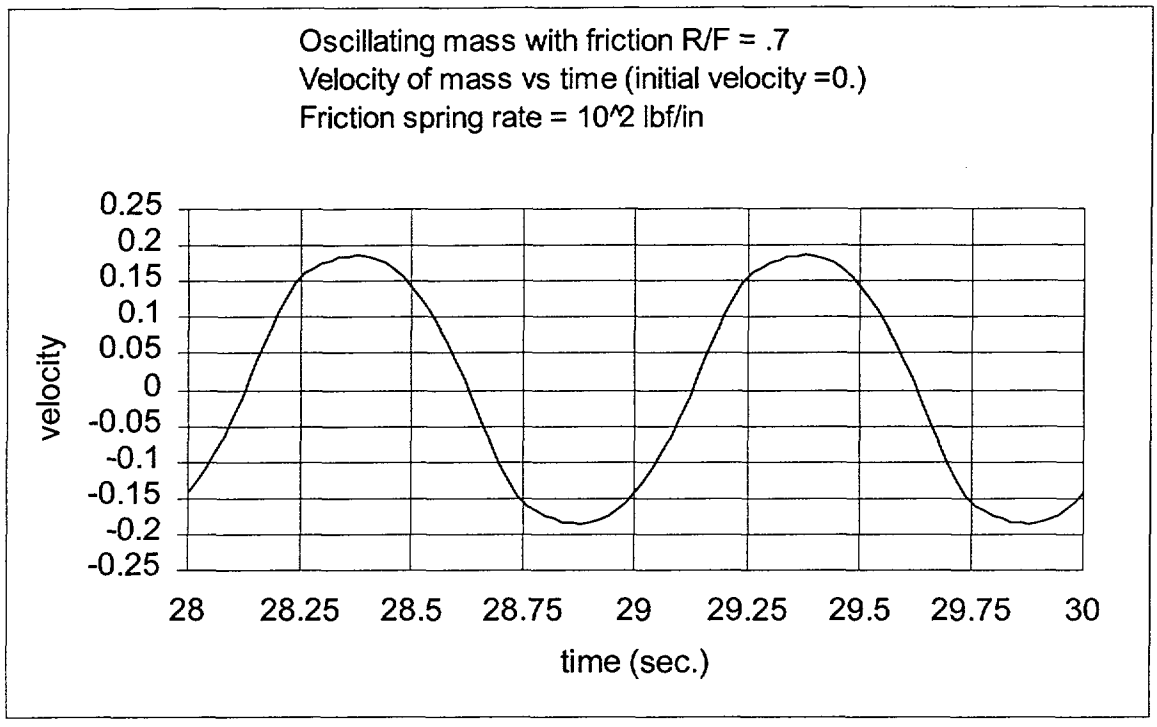
Q149. Please describe the classical problem which is discussed by Tou and Schultheiss.

A149. In this problem, a rectangular box is placed on a flat surface which permits a frictional resistance force to be developed as the mass oscillates on the flat surface. An external sinusoidal force is applied to the mass. Depending on the ratio of maximum frictional force that can be developed to the maximum amplitude of the applied sinusoidal force, different effects may be observed. For maximum friction force amplitude to applied force amplitude ratio less than 0.536, it is shown in the reference classical solution that the response of the mass is approximately sinusoidal with discontinuities in the acceleration. However, if the same ratio is greater than 0.536, then the motion is sporadic, with "dead bands" occurring in time, where the motion halts (and later resumes). Finally, when the ratio of friction resistance to applied force exceeds 1.0, then no motion, save an initial transient, occurs.

Q150. Please describe how you went about evaluating the capability of Dr. Khan's model to predict the solution of this problem.

A150. In the validation performed by Holtec, we modeled the mass, the frictional surface, and the applied sinusoidal force. To ensure that we correctly modeled the

"stick-slip" nature of frictional resistance, we assumed a large value for the horizontal spring (10,000,000 lb./inch) that simulated the behavior prior to sliding (since, the problem was fairly simple, the use of this very large value to simulate an "infinite stiffness" gave us no convergence problems). Our results reproduced the phenomena predicted by the classical solution (see PFS Exhibit RR). To demonstrate the inappropriateness of the low value for horizontal spring rate suggested by Khan, we took the Holtec DYNAMO Code and modified the input so that Dr. Khan's choice of input data was used. Since he feels that a ratio of weight to friction spring rate of 360,000/100,000 = 3.6 is appropriate, we used the DYNAMO Code and used a friction spring rate of 107.33 lb/in (note that the benchmark application uses a mass of 1 lb-sec^2/inch, which is a weight of 386.4 lb; therefore, to get the same ratio that Dr. Khan suggests is appropriate for the friction spring, requires that k = 386.4/3.6). The remainder of the parameters were set so that the solution should produce "dead bands". The figure below represents what we call the "Khan Solution" and plots the velocity of the mass vs. time. Since no dead bands are evident, Dr. Khan's choice of parameters, applied to this problem, produces a solution that clearly does not agree with the theoretical results (PFS Exhibits QQ andRR).



Q151. What is your conclusion therefore with respect to Dr. Khan's choice of contact stiffness values?

A151. Dr. Khan violated the first and foremost principle in simulating contact friction: namely, choose stiffness values that are high enough so that no visible penetration or elastic movement, prior to sliding, is predicted. Dr. Khan's vertical stiffness value of 1×10^6 lbs/inch and his horizontal stiffness value of 1×10^5 lbs/inch produce nonsensical results for simple, easily understood physical problems. Dr. Khan's proposed input parameters also predict a static vertical interpenetration of 0.36" and a movement of 0.71" prior to sliding, again unreasonable and at odds with reality. A computer code whose application in test cases gives unreasonable results is likely to run into convergence problems when applied to real life situations.

Q152. Do you see any convergence problems manifesting themselves for the contact stiffnesses that Dr. Khan professes to be "appropriate?"

A152. (KPS, AIS) Yes. Clear evidence that Dr. Khan's model, at his proposed contact stiffness values, runs into convergence problems can be seen by close

- examination of some results in Table 2 of the Altran report, in particular cases 2, 4, 6 and 10. These cases are set forth in the Table below, which extract the relevant data from the Altran Report.

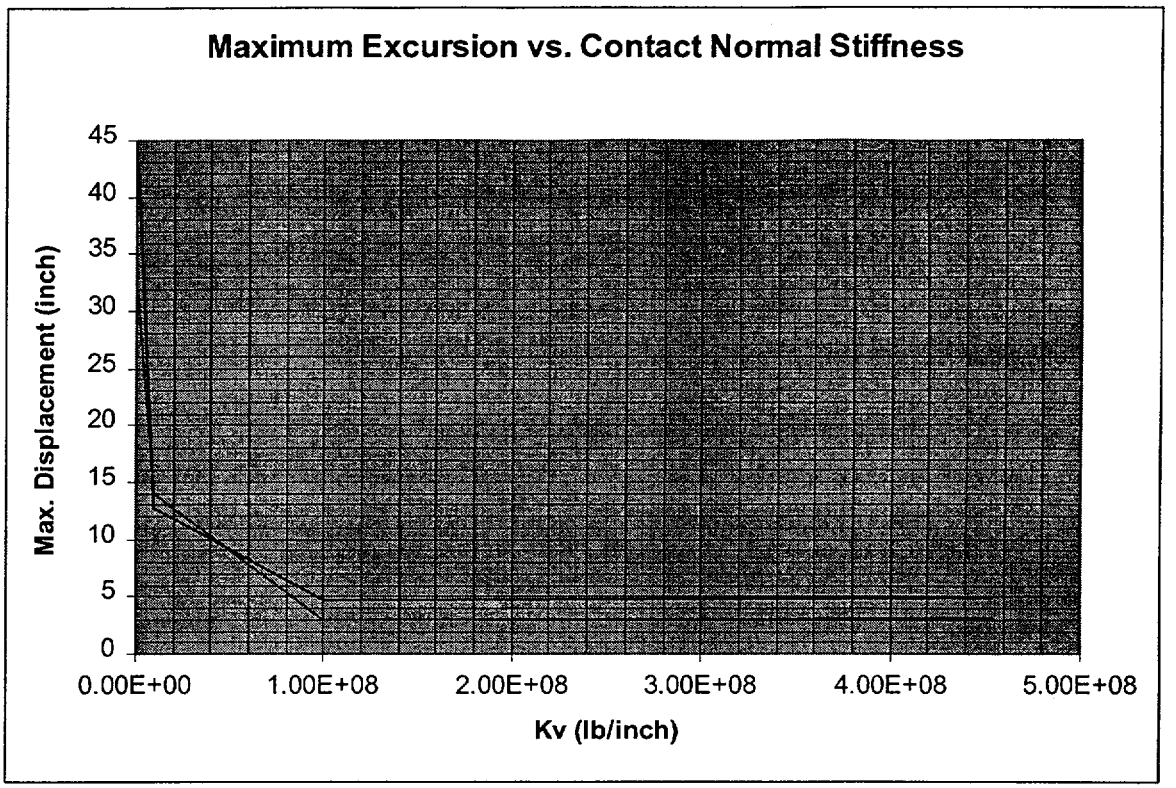
Information Excerpted from Table 2 of Altran Report					
		Stiffness for Non-Linear Elements		Relative Cask Displacements	
Study Run #	Coefficient of Friction	Vertical Stiffness (lb/inch)	Horizontal Stiffness (lb/inch)	Horizontal Displacement (inch)	Vertical Displacement (inch)
2	.8	1,000,000	100,000	42.74	31.35
4	.8	10,000,000	100,000	12.70	14.03
6	.8	100,000,000	100,000	4.74	3.05
10	.8	454,000,000	100,000	4.83	3.06

These cases are of interest since the only difference between them is the value for the vertical stiffness. Thus, the Khan solution of these cases is supposed to show the effect of changing only the vertical stiffness.

Q153. Please describe what this Table shows?

A153. (KPS, AIS) We focus on these cases because the assumptions for them differ only in the choice of vertical stiffness at the contact interface (although they all use the horizontal friction stiffness having an unrealistically invalid low value of 100,000 lb./inch, as previously discussed above). A plot of the results from Dr. Khan's analysis is given below (the two lateral excursions (the last two columns of the Khan excerpted data) are plotted against vertical stiffness value (the third column of the extracted data)) . The key point is not that the displacement results

are different, but rather, that they "settle down" (converge) to a value that is independent of the exact stiffness chosen.



Q154. What do you conclude from this graphic display of the results of the table?

A154. (KPS, AIS) This table and the graph show that Dr. Khan's results are insensitive to changes in contact stiffness values after some plateau is reached, which would generally correspond to the lack of visual penetration of the two objects. In reality, this aspect of Dr. Khan's results serve as a validation of the correctness of Holtec's stiffness value at 100,000,000 lb/inch, and show that the results are insensitive to the choice of stiffness after a certain plateau is reached, as they should be. As noted earlier, the results with lower stiffness values also fail the "visible" interpenetration test (these initial values are not reported in the Khan analysis) and thus, do not conform to the guidance provided by ANSYS; therefore, it would be obvious to a practitioner more familiar with this kind of problem that the assumptions should be suspect.

Q155. In his deposition, Dr. Khan argued that the examples involving static conditions, such as those you have discussed above, were irrelevant to modeling dynamic motion where the contact stiffness between the casks and the pads would be constantly changing. What is your response to Dr. Khan's argument?

A155. (KPS, AIS) As previously discussed, the model should be able to provide realistic answers to all such situations, as does Holtec's.

Q156. How does Holtec's computer code model dynamic motion situations?

A156. (KPS, AIS) The dynamic change of contact stiffnesses between the pad and the cask, due to changing contact area, is modeled by having a series of springs between the pad and the cask over which the contact stiffness is divided. For example, Holtec's DYNAMO model employs 36 springs between the cask and the pad around the circumference of the cask, which means that each spring represents a contact stiffness of 454×10^6 lbs/inch divided by 36, or 12.6×10^6 lbs/inch. Thus, if part of the cask lifts off during an earthquake, the instantaneous contact stiffness between the cask and the pad will change and will only include those points actually in contact at that instant.

Q157. Dr. Khan also suggests that use of a high contact stiffness, such as that used by Holtec, is inappropriate because "high stiffnesses absorb significant amount of energy" before either sliding or tipping occurs. What is your response to this assertion by Dr. Khan?

A157. (KPS, AIS) Dr. Khan is simply wrong, and misconstrues the laws of physics governing linear springs. The energy absorbed by a linear spring is given by a simple relation $E = 0.5 \times K \times d^2$ where "K" is the stiffness of the spring and "d" the compression of the spring, which in the model here, where the springs represent local contact stiffness at an interface, is also the deflection or interpenetration at the cask-pad interface. For a given value of compression force W, since $W = K \times d$, the energy absorbed by the spring can be expressed as: $E=0.5 \times W^2/K$. This is recoverable energy (since we deal only with linearly elastic springs) which means that the spring will "give back" the energy that it absorbed during the compression cycle when it decompresses (prior to separation) Therefore, as K gets larger for a given W, the energy absorbed by the spring is

less, rather than more (since K appears in the denominator of the energy relation, a larger K means less energy for the same value of W), directly contrary to Dr. Khan's assertion.

Q158. Dr. Khan also claims that although high contact stiffness values are generally used in mathematical simulations, the high stiffness values artificially treat the solution as linear without amplifying it in the upward direction and give non-unique or invalid results. Do you agree with Dr. Khan's assertions?

A158. (KPS, AIS) We agree with Dr. Khan's first assertion that "high contact stiffnesses are generally used." Indeed, that is precisely the guidance provided by ANSYS that "contact stiffness should be very great" because "[m]inimum penetration gives best accuracy." ANYSY recommends lower values only if "too stiff of a value causes convergence difficulties, but the lower values should still pass the test of "no visible penetration". His second assertion that the use of high contact stiffness values "gives non-unique or invalid results" is flatly wrong and contrary to accepted modeling practice, as demonstrated by the ANSYS provisions just quoted. As stated, the objective in choosing an appropriate contact stiffness value is to pick one in the range where your results are not sensitive to the precise choice of the contact stiffness value chosen. In the range of contact stiffness values proposed by Dr. Khan, his own results shows that this fundamental precept is violated.

Q159. What is your conclusion regarding Dr Khan's claims concerning an appropriate contact stiffnesses to use in modeling cask stability?

A159. (KPS, AIS) In our opinion, Dr. Khan's report does not support any of the claims made by the State. Dr. Khan's choice of model parameters for a number of his simulations do not satisfy the basic test required of all contact and friction analyses; namely, that they do not predict excessive penetration nor excessive movement prior to sliding. Dr. Khan has failed to validate his model; indeed, we have shown in our responses that Dr. Khan's choices do not give agreement with simple exact solutions.

3. Dr. Khan's Misinterpretation of other Key Holtec Input Parameters

Q160. In what other respects does Dr. Khan misinterpret the input parameters used by Holtec in its cask stability analysis?

A160. (KPS, AIS) Dr. Khan misinterprets and misapplies the 5% beta damping value that Holtec used in cask stability analysis. According to Dr. Khan, the 5% beta damping is a structural damping, and Dr. Khan further argues that 5% structural damping is much too high for two bodies assumed to be rigid in the Holtec analysis -- the cask and the pad -- and argues that the beta damping value should be on the order of 1%. Dr. Khan fails to understand that the damping used in Holtec's model does not represent structural damping (since rigid bodies have no structural damping); rather, the damping included in the cask-to-pad contact elements represents impact damping, and reflects the physical fact that there is energy lost when the cask impacts the target concrete and then rebounds. The simple discussion and problem, excerpted from a Holtec report on another project and identified as PFS Exhibit TT, illustrates this point. In the simple example, when a known mass is dropped from a fixed height, it is physically observable that it does not return to its initial height. It can be shown that the difference in height is related to a quantity defined as the "coefficient of restitution". In simple terms , if H0 is an initial drop height for the mass, and H1 is the measured height to which the mass returns, after impact, then the coefficient of restitution, "e", is defined by the equation

$$e^2 = H1/H0$$

Alternatively, the coefficient of restitution is equally definable in terms of the relative velocity of approach, "Va", and the relative velocity of separation, "Vs". Recognizing that for the case of a vertical drop of the mass, the approach velocity is "down", and the separation velocity is "up," the coefficient of restitution is also defined as:

$$Vs/Va = e$$

These two definitions are interchangeable. We can simulate the physical phenomena by defining a mass-spring-damper system and studying its behavior during the time period when impact begins, and when impact ends. The solution of this simple problem can be done analytically and provides a solution for the velocity ratio solely in terms of the critical damping constant. Thus, a unique relation between critical damping value and coefficient of restitution can be defined. PFS Exhibit TT. contains details of the development.

Thus, contrary to Dr. Khan, the 5% damping used by Holtec is not structural damping of the cask (even though there would be considerable structural damping of the canister and canister internals which Holtec conservatively ignores in its model). Rather, it is the damping or dissipation of energy resulting at the contact points between the cask and the pad. The use of 5% damping for dampers at the contact interface implies a coefficient of restitution, "e" approximately 0.85. In physical terms, if we drop the cask from a height of 12", then classical impulse-momentum considerations predict that it would rebound to a height of H = (.85)(.85)(12") = 8.67" The use of dampers, with an appropriate percentage of critical damping, in parallel with the contact stiffness, is the appropriate way to model this phenomena.

Q161. In his deposition, Dr. Khan claimed that it was inappropriate to assume impact damping at the cask-pad interface. Khan Dep. at 124-134. What is your response to Dr. Khan's claims on this point?

A161. (KPS, AIS) Dr. Khan, in his deposition, refused to consider the possibility of a cask moving up and down and dissipating energy by impact damping during the period when it contacts and then rebounds from the target. As we have just shown, the loss of energy in a vertical impact problem can only be simulated in a numerical analysis by including a damper in the model. Dr. Khan's impression would be that the spring absorbs the energy (p.126, line 17) but fails to mention that it gives it all back when it expands. Dr. Khan does not consider an automotive shock absorber as a damper but implies that a car's vibration is slowed and ended because it being stopped by a rigid surface. He continues by

claiming, erroneously in our opinion, that the spring is "dissipating the energy through stiffness".

Q162. In a similar vein, in the Utah Joint Declaration, Dr. Khan claims that friction should be the primary energy dissipation mechanism, not damping or any other form of dissipation of energy associated with the spring at the cask-pad interface. What is your response to this claim by Dr. Khan?

A162. (KPS, AIS) At the interface, the friction effect predominates when horizontal sliding predominates, and the damper in parallel with the normal contact spring will be the only energy dissipator when there is no sliding. Under no circumstances, will linear springs permanently remove energy from the problem.

Q163. Dr. Ostadan has also claimed that the 5% damping used by Holtec is too great and has suggested that the damping that you have illustrated by your example of a bouncing ball is resistance damping attributable to the damping effect of the soils and foundations. What is your response to this claim raised by Dr. Ostadan?

A163. (KPS, AIS) Dr. Ostadan has misinterpreted the modeling in the Holtec simulation. There is damping to account for the effect of the cask impacting a target, and there is also damping associated with the soil response. If the cask was fixed to the pad to the pad, you would have only soil damping; on the other hand, if the soil were perfectly rigid, you would still have to model the observable fact that when an object is dropped, it does not rebound to its same height. As noted in our previous response, a damper in parallel with a contact spring is necessary to characterize this behavior. The damping referred to by Dr. Ostadan is the damping associated with the soil spring under the pad whereas the damping that we are discussing here is associated with the spring between the cask and the pad. Thus, we have two different stiffnesses and specific damping associated with each stiffness.

Q164. What is your conclusion regarding the State's claims that use of 5% damping by Holtec in its modeling of cask stability is inappropriate?

A164. (KPS, AIS) The use of 5% damping for energy dissipating dampers in parallel with contact stiffness elements leads to a reasonable and conservative estimate of the rebound if we imagine dropping the cask from a fixed height and calculating

the rebound. The same methodology has been reviewed and accepted by the NRC in the wet storage licensing submittals. Its application to the cask analyses is reasonable and appropriate.

4. Dr. Khan and the State's Other Witnesses Inappropriately Rely upon the Results of Dr. Khan's Model From Inadequate Model and Erroneous Input Parameter as Realistic Solutions

Q165. What are your conclusions regarding the information provided in the Altran Report in Table 3.

A165. The results in the cited table, in our opinion, are completely erroneous. The reason for this is primarily that they all use a low value of horizontal stiffness which cannot be expected to give agreement with any known exact solutions. In addition, some of the simulation results compound the error by also assuming improper vertical stiffness. We note that all of the results, quoted by Khan and used by the other State experts, that lead to approximately 30 ft. lateral movements, and the casks "jumping" into the air, have as their inputs, the discredited low values for vertical and horizontal stiffness. Therefore, the results, besides being physically unbelievable, suffer from bad input data. There is no evidence of any trend in the tabular results. Therefore, we cannot even begin a rational dissection of the results in Table 3 as we did with Table 2 of the report as there is no two sets of results that can be "trusted" as being based on good input data.

Q166. Please explain why the examples of large objects tipped over or otherwise disturbed by large earthquakes referred to by the State's experts do not support the results of Dr. Khan's model or otherwise show that excessive sliding or tipping of the Holtec casks during a large earthquake event is likely.

A166. The examples cited by the State's witnesses of large objects turning over do not support any conclusions reached by Dr. Khan or by Dr. Ostadan concerning the response of HI-STORM casks. Simply stated, the ratio of object height to object supported width in the State's examples is much larger than the same ratio applied to the HI-STORM cask. Given the same earthquake strength, objects with a larger

height/width ratio are more prone to overturning. For HI-STORM, the ratio is approximately 1.8. For the State' examples, the corresponding ratios would appear to be much higher (based on estimates from the photographs).

VI. TESTIMONY CONCERNING SECTION E OF THE UNIFIED CONTENTION

VII.

Q167. What is your understanding of the State's claims in Section E of the Unified Contention?

A167. (KPS, AIS) Section E challenges the granting of the exemption from the requirements of 10 CFR § 72.102(f) to allow PFS to employ a probabilistic seismic hazard analysis using a 2,000-year return period earthquake as the design basis for the PFSF. The State asserts that PFS should be required to either use a probabilistic methodology with a 10,000-year return period or comply with the existing deterministic analysis requirements of section 72.102(f), or, alternately, using a return period significantly greater than 2,000 years.

Q168. Is the HI-STORM System able to withstand earthquakes greater than the 2,000-year return period design basis earthquake used for the PFSF?

A168. (KPS, AIS) Yes. As discussed earlier, the design of the HI-STORM System has many conservatisms that would allow it to survive and continue to fulfill its safety function under far greater ground motions than those produced by the 2,000-year design-basis earthquake. First, the cask stability analysis performed by Holtec demonstrates that a HI-STORM System storage cask can withstand much larger seismic events than the 2,000 year design basis earthquake without significant pure sliding motion or tipping over. Second, even if a cask were to sustain an impact due to sliding, or a cask were otherwise to impact another without tipping over, that impact would be significantly less severe than the impacts posited in the hypothetical cask tip-over analysis. Third, assuming that a cask were to tip over, the velocity of the impact due to the tipover would be in the same range as that in the hypothetical cask tip-over analysis that we performed, which shows that canister's confinement integrity would not be threatened. Fourth, even if one were to assume that a tip-over would have a larger velocity than that postulated in

the hypothetical cask tip-over analysis, the huge margins in the design of the cask and canister system would prevent the release of radioactive material.

Q169. Please summarize the results of the various cask stability analyses that Holtec has performed for the PFSF.

A169. (KPS, AIS) Under design basis earthquake loadings, the maximum calculated cask displacement is less than 3.25 inches, which leaves large margins (at least over three feet of clearance) before the casks were to impact each other, and a large margin exists against cask tip-over. In a 10,000-year return period earthquake, large margins still exist against cask tip-over, the factor of safety against tip-over at the 10,000 year earthquake is still on the order of 2 to 3 as measured against the center-of-gravity over corner location. Further, even under unrealistic, "worst-case" assumptions as to damping and other factors, the casks do not tip-over in a 10,000-year earthquake. Under some of the scenarios that we studied, some of the casks may impact each other, but the impacts occur at relatively low speeds with no damage to the casks or loss of stability; the net effect of the collision is that one of the cask loses most of its energy and its motion and shortly comes to a halt. No impacts due to sliding were observed under any of the scenarios that were run using the 10,000-year earthquake, even those based on "worst case" assumptions. Even if sliding impacts were to occur, the velocities of the impacts would be much lower than the velocity of impact determined in the hypothetical cask tip-over event. The conclusion, therefore, is that even at the 10,000-year earthquake ground motions, large margins exist against cask tip-over or any cask-to-cask impacts that might threaten the confinement integrity of the MPC canister.

Q170. Assuming hypothetically that a cask were to tip over in a beyond-design basis earthquake, would the confinement capability of the MPC canister be threatened?

A170. (KPS, AIS) No. In reviewing the computer-generated movie files showing the behavior of the casks in an earthquake, we observed that casks tend to tilt from the vertical, resulting in a plane of precession for certain durations in the course of the earthquake event. The cask experiences an oscillatory rocking motion, while

precessing, with periodic returns to the vertical position, until the rocking finally ends when the earthquake subsides. This behavior supports the assumption of a zero initial angular velocity if the cask ever begins to tip over. Observation of the simulated motion experienced by the PFSF casks during the 10,000-year event and other non-PFSF simulations of cask tipover leads us to conclude that, if the strength of the seismic event were increased to the point where the cask did tip over, the initiating angular velocity propelling the cask towards the ground is quite small. Furthermore, the precession characteristics of the motion of the cask enables it to remain stable even while the center of gravity of the cask is well past the corner. As a result of the precession motion, with superimposed rocking, the initial height of the cask center of gravity is apt to be much lower than the statically computed tipover scenario (where tipover begins as soon as the center of gravity crosses the vertical plane containing the axis of overturning rotation). With less distance to fall, and a negligible initial angular velocity propelling the tip-over, a cask tipping away from the precession motion is expected to have substantially less kinetic energy of collision than one tipping from zero velocity with center of gravity of over corner. Moreover, even if one were to assume that a tip-over would have a larger velocity than that posited in Holtec's hypothetical cask tipover analysis, the huge margins in the design of the MPC canister system would prevent the release of radioactive material. This has been demonstrated by the canister's capability to withstand a 25 ft. straight drop, unprotected by a cask onto a hard concrete surface, with a still significant margin, after impact, before reaching the failure strain limit of the material.

Q171. Based on the conservatisms you have described, do you have an opinion regarding the magnitude of a beyond design basis earthquake that the HI STORM 100 storage cask system could withstand?

A171. (KPS, AIS) Yes. As discussed above, the cask storage system can experience a 10,000-year return period earthquake without cask tip-over or significant sliding. Moreover, there are significant additional margins of safety within the storage cask system in the unlikely event of an actual cask tipover event. Thus, it is clear that the HI-STORM System can experience and withstand, without the release of

radioactive material, not only a 10,000-year return period earthquake, but also earthquakes of substantially larger magnitude.

Q172. Does that conclude your testimony?

A172. Yes it does.

KRISHNA P. SINGH, Ph.D., PE

EXECUTIVE ENGINEER
HOLTEC INTERNATIONAL

EDUCATION

University of Pennsylvania
Ph.D. in Mechanical Engineering (1972)
GPA: 4.0 Out of 4.0

University of Pennsylvania
M.S. in Mechanical Engineering (1969)
GPA: 4.0 out of 4.0

B.I.T. Sindri, Ranchi University
B.S. In Mechanical Engineering (1967)
(Ranked in the top 1% of Engineering Graduates)

AREAS OF PROFESSIONAL CONCENTRATION

Application of ASME, ACI, and NUREG-0612 Codes. Mechanical and civil/structural design of weldments and reinforced concrete systems. Applied heat transfer and fracture assessment of dry storage systems.

PROFESSIONAL EXPERIENCE

HOLTEC INTERNATIONAL
Marlton, New Jersey

1986-Present	President and CEO

JOSEPH OAT CORPORATION
Camden, New Jersey

1979 - 1986	Vice President of Engineering
1974 - 1979	Chief Engineer
1971 - 1974	Principal Engineer

R.I.T. ALLAHABAD
India

1967 – 1968	Assistant Professor of Applied Mechanics

PROFESSIONAL CERTIFICATIONS

Registered Professional Engineer - Pennsylvania (1974-present)
Registered Professional Engineer - Michigan (1980-present)

PROFESSIONAL SOCIETY MEMBERSHIPS/ACTIVITIES

Elected Fellow of the ASME (1987); Member ANS (1979-Present); Member, ASME (1973-Present); Chairman, TEMA Vibration Committee (1979 - 1986); Chairman, PVP Committee Of the ASME, Nuclear Engineering Division (1988-92); Member, ASME O&M Committee (1991 to present); Member ASCE (1977-83), Member, Heat Exchange Institute (1976-86).

PATENTS

"Heat Exchanger for Withstanding Cycle Changes in Temperature" (with M. Holtz and A. Soler), U.S. Patent No. 4,207,944 (1980).

"Radioactive Fuel Cell Storage Rack" (with M. Holtz), U.S. Patent No. 4,382,060 (May, 1983).

"Apparatus Suitable for Transporting and Storing Nuclear Fuel Rods and Methods for Using the Apparatus", U.S. Patent No. 5,898,747 (April, 1999)

"Apparatus Suitable for Transporting and Storing Nuclear Fuel Rods and Methods for Using the Apparatus", U.S. Patent No. 6,064,710 (May 16, 2000)

"Duct Photon Attenuator for Installation in a Ventilated Overpack Used to Store Spent Nuclear Fuel" (with Everett L. Redmond, John C. Wagner, and Stephen Agace) (Patent Pending)

"Below Grade Cask Transfer Facility" (with Stephen Agace) (Patent Applied For)

"Seismic Cask Stabilization Device" (with A.I. Soler) (Patent Applied For)

"Ventilated Overpack for Storing Spent Nuclear Fuel" (with Stephen Agace) (Patent Applied For)

BOOKS AND ARCHIVAL VOLUMES (authored or edited):

1. "Mechanical Design of Heat Exchangers and Pressure Vessel Components", (authored with A. I. Soler), Arcturus Publishers, Cherry Hill, New Jersey, 1100 pages, hardbound (1984).

2. "Theory and Practice of Heat Exchanger Design" (sole author), Arcturus Publishers (ca. 2000).

3. "Feedwater Heater Workshop Proceedings", edited with Tom Libs, EPRI 78-123 (1979).

4. "Feedwater Heater Technology: State-of-the-Art", sole author, EPRI - cs - 4155 (1985).

5. "Analytical Correlations of Fluid Drag of Fuel Drag of Fuel Assemblies in Fuel Rack Storage Locations", sole author, EPRI Project RP-2124.

6. "Thermal/Mechanical Heat Exchanger Design", (edited) ASME, PVP - Vol. 118 (1986).

7. "Time Dependent and Steady State Characterization of the CAES Recuperator", (principal author) EPRI TR-104224 (July 1994).

8. "Pressure Vessels, Heat Exchangers and Piping", Proc. ASME, IEEE Joint Power Generation Conference, (editor) NE-14 (1994).

EXPERT WITNESS AND TECHNOLOGY APPRAISAL SERVICES FOR NUCLEAR PLANTS AND NATIONAL LABORATORIES

Most of the expert witness activities pertain to spent fuel storage technology and PWR steam generator design.

1. Pacific Gas & Electric Company vs. National Sierra Club (1986-87).

2. Florida Power & Light Company vs. Stuart Intervenor Group (1990).

3. Duquesne Light Company vs. Westinghouse (1993-1994).

4. Portland General Electric vs. Westinghouse (1993-1994).

5. Houston Light and Power vs. Westinghouse (1994-1995).

6. Pacific Northwest Laboratories, Rockwell International, and U.S. DOE vs. RSI (1994).

7. Northern States Power vs. Westinghouse (1996)

8. Commonwealth Edison Company vs. Westinghouse (1997)

ACADEMIC ACTIVITIES

Chair, Advisory Committee On Mechanical Engineering and Mechanics, University of Pennsylvania (1993-1999)

Professor (Adjunct) in Mechanical Engineering and Mechanics, University of Pennsylvania (1986-92), Offered Graduate and Undergraduate Courses in Heat Transfer Equipment and Pressure Vessel Technology.

CONTINUING EDUCATION COURSES OFFERED TO PRACTICING GRADUATE ENGINEERS

1. I.I.T. Bombay, One Week Course on Heat Exchanger Design (1979).

2. Duke Power Company, Charlotte, NC (1982, 1983, 1986, 1990) - In-house Training Course on Heat Exchanger Design and Testing.

3. National Italian Reactor Authority, Genoa, Italy - On Condensers, Steam Generators, and Moisture Separator Reheaters (1985).

4. Mississippi Power & Light Company, In-House Course on Moisture Separator Reheaters and Surface Condensers (1987).

5. Center for Professional Advancement (1988, New Brunswick, NJ; 1990, Caracas, Venezuela; 1991, Houston, Texas; 1992, Amsterdam, Holland).

SPENT FUEL STORAGE TECHNOLOGY

- Developer of the industry's first multi-purpose canister design (ca. 1993), later licensed by the USNRC under Docket 71-9261 for transport and Docket 72-1008 for storage. Patent for a unique spent fuel basket design granted by the U.S. Patent Office in April, 1999 (U.S. Patent No. 5,898,747).

- Co-developer of Cask Transfer Facility Specification and Design.

- Developed the nonlinear methodology for cask drop analysis within §50 jurisdiction in support of Shorehams defueling project (ca. 1994). Participated in dynamic (drop) analysis of TN-12 and IF-300 casks.
- Developer of the multi-layer transport overpack design in 1993, subsequently licensed as the HI-STAR 100 dual-purpose overpack.
- Performed brittle fracture analysis of MPC lid welds in Holtec MPC systems.
- Participated in the development of Holtec's thermal evaluation methodologies for dry storage systems.
- Developer of the thermosiphon action MPC design.
- Developed dozens of company position papers and generic reports for Holtec International for cask system design and analysis.
- Author of over 200 industry reports on dry and wet storage technologies.
- Developer of detuned honeycomb rack design used by Holtec International in over sixty rerack projects.
- Led licensing of over fifty O.L. amendment requests for reracking spent fuel pools.
- Over a dozen technical papers in dry and wet storage of spent nuclear fuel.

TECHNICAL CONSULTING

Technical consulting services to over fifty national and international organizations, including: Electric Power Research Institute (EPRI); Pressure Vessel Research Council (PVRC); Tubular Exchanger Manufacturers Association (TEMA); Department of Energy (DOE) (Idaho Operations); Department of Energy (DOE) (Chicago Operations); American Electric Power Corporation; Baltimore Gas and Electric; Carolina Power & Light; Commonwealth Edison Company; Detroit Edison Company; Duke Power Company; Entergy Operations; GPU Nuclear; Iowa Electric Light and Power; New York Power Authority; Niagara Mohawk Power Corporation; North Atlantic Energy Services; Northeast Utilities; Northeast Nuclear Energy; Pacific Gas and Electric Company; PECO Energy; Southern Nuclear Operating Company; and Tennessee Valley Authority.

PUBLICATIONS

1. "A Method for Solving Ill-Posed Integral Equations of the First Kind", (with B. Paul), Computer Methods in Applied Mechanics and Engineering 2 (1973) 339-348.

2. "Numerical Solutions of Non-Hertzian Elastic Contact Problems", (with B. Paul), Journal of Applied Mechanics, Vol. 41, No. 2, 484-490, June, 1974.

3. "On the Inadequacy of Hertzian Solution of Two Dimensional Line Contact Problems", Journal of the Franklin Institute, Vol, 298, No. 2, 139-141 (1974).

4. "How to Locate Impingement Plates in Tubular Heat Exchangers", Hydrocarbon Processing, Vol. 10, 147-149 (1974).

5. "Stress Concentration in Crowned Rollers", (with B. Paul), Journal of Engineering for Industry, Trans. ASME, Vol. 97, Series B, No. 3, 990-994 (1975).

6. "Application of Spiral Wound Gaskets for Leak Tight Joints", Journal of Pressure Vessel Technology, Trans. ASME, Vol. 97, Series J, No. 1, 91-93 (1975).

7. "Contact Stresses for Multiply-Connected Regions - The Case of Pitted Spheres:, with B. Paul and W. S. Woodward, Proceedings of the IUTAM Symposium on Contact Stresses, August 1974, Holland, Delft University Press, 264-281, (1976).

8. "Design of Skirt-Mounted Supports:, Hydrocarbon Processing, Vol. 4, 199-203, April 1976.

9. "Predicting Flow Induced Vibration in U-Bend Regions of Heat Exchangers - An Engineering Solution". Journal of the Franklin Institute, Vol. 302, No. 2, 195-205, August 1976.

10. "A Method to Design Shell-side Pressure Drop Constrained Tubular Heat Exchangers", with Mr. Holtz, Journal of Engineering for Power, Trans. of the ASME, Vol. 99, No. 3 July 1977, pp 441-448.

11. "An Efficient Design Method for Obround Pressure Vessels and Their End Closures", International Journal of Pressure Vessel and Piping, Vol. 5, 1977, pp 309-320.

12. "Analysis of Vertically mounted Through-Tube Heat Exchangers", Journal of Engineering for Power, Trans. ASME, Vol. 100, No. 2, April, 1978, pp 380-390.

13. "Study of Bolted Joint Integrity and Inter-Tube-Pass Leakage in U-Tube Heat Exchangers: Part I - Analysis", Journal of Engineering for Power, Trans. ASME, Vol. 101, No. 1, pp 9-15 (1979).

14. "Study of bolted Joint Integrity and Inter-Tube-Pass Leakage in U-Tube Heat Exchangers, Part II - Applications", Journal of Engineering for Power, Trans. ASME, Vol. 101, No. 1, pp 16-22 (1979).

15. "On Thermal Expansion Induced Stresses in U-Bends of Shell-and-Tube Heat Exchangers", (with Maurice Holtz); Trans. ASME, Journal of Engineering for Power, Vol. 101, No. 4, October, 1979, pp. 634-639.

16. "Heat Transfer Characteristics of a Generalized Divided Flow Heat Exchanger", Proceedings of the Conference on Industrial Energy Conservation Technology, Houston, Texas, pp 88-97 (1979).

17. "An Approximate Analysis of Foundation Stresses in Horizontal Pressure Vessels", (with Vincent Luk), Paper No. 79-NE-1, Trans. ASME, Journal of Engineering for Power, Vol. 102, No. 3, pp 555-557, July, 1980.

18. "Generalization of the Split Flow Heat Exchanger Geometry for Enhanced Heat Transfer", (with Michael Holtz), AIChe. Symposium Series 189, Vol. 75, pp 219-226 (1979).

19. "Analysis of Temperature Induced Stresses in the Body Bolts of Single Pass Heat Exchangers", ASME Winter Annual Meeting, Paper No. 79 QA/NE-7, New York, NY, 1979.

20. "Optimization of Two-Stage Evaporators for Minimizing Rad-Waste Entrainment", (with Maurice Holtz), Journal of Mechanical Design, Trans. of the ASME, Vol. 102, No. 4, pp 804-806 (1980).

21. "A Comparison of Thermal Performance of Two and Four Tube Pass Designs for Split Flow Shells", (with M. J. Holtz), Journal of Heat Transfer, Trans. of the ASME, Vol. 103, No. 1, pp 169-172, February, 1981.

22. "A Method for Maximizing Support Leg Stress in a Pressure Vessel Mounted on Four Legs Subject to Moment and Lateral Loadings". International Journal of Pressure Vessels and Piping, Vol. 9, No. 1, pp 11-25 (1981).

23. "Design, Stress Analysis and Operating Experience in Feedwater Heaters",, (with Tom Libs), Proceedings of the Conference on Industrial Energy Conservation Technology, Houston, pp 113-118 (1980).

24. "On the Necessary Criteria for Stream Symmetric Tubular Heat Exchanger Geometries", Heat Transfer Engineering, Vol. 3, No. 1 (1981).

25. "Some Fundamental Relationships for Tubular Heat Exchanger Thermal Performance", Trans. ASME, Journal of Heat Transfer, Vol. 103, pp 573-578 (1981).

26. "Transient Swelling of Liquid Level During Pool Boiling in an Emergency Condenser", (with J. P. Gupta). Letters in Heat and Mass Transfer, Vol. 8, No. 1, pp 25-33, Jan/Feb., 1981.

27. "An Approximate Method for Evaluating the Temperature Field in Tubesheet Ligaments Under Steady State Conditions", (with M. Holtz), Journal of Engineering for Power, Trans. ASME, Vol. 104, pp 895-900 (1982).

28. "Feasibility Study of A Multi-Purpose Computer Program to Optimize Power Cycles for Operative Plants", (with Y. Menuchin and N. Hirota), Proceedings of the Conference on Industrial Energy Conservation Technology, Houston, (1981).

29. "Design Parameters Affecting Bolt Load in Ring Type Gasketed Joints", (with A. I. Soler), Trans. ASME, Journal of Pressure Vessel Technology, Vol 105, pp 11-13 (1983).

30. "A Design Concept for Minimizing Tubesheet Stress and Tubejoint Load in Fixed Tubesheet Heat Exchangers", (with A. I. Soler), Trans. ASME (C. 1982).

31. "Dynamic Coupling in a Closely Spaced Two-Body System Vibrating in Liquid Medium: The Case of Fuel Racks", (with A. I. Soler), Proceedings of the Third International Conference on "Vibration in Nuclear Plant", Keswick, England, May, 1982, pp. 815-834.

32. "Effect of Nonuniform Inlet Air Flow on Air Cooled Heat Exchanger Performance", (with A. I. Soler and Lee Ng), Proceedings of the Joint ASME-JSME Heat Transfer Conference, 1983, pp. 537-542.

33. "Seismic Response of Free Standing Fuel Rack Constructions to 3-D Motions", (with A. I. Soler), Nuclear Engineering and Design, Vol. 80, (1984), pp. 315-329.

34. "A Method for Computing Maximum Water Temperature in a Fuel Pool Containing Spent Nuclear Fuel", Heat Transfer Engineering, Hemisphere, Dec. (1986).

35. "On Minimization of Radwaste Carry-Over in a N-stage Evaporator", (with Maurice Holtz and Vincent Luk), Heat Transfer Engineering, pp. 68-73, Vol. 5, No. 1-1 (1984).

36. "Feedwater Heater Procurement Guidelines - Some New Performance Criteria", Symposium on State-of-the-art Feedwater Heater Technology, EPRI (c. 1984).

37. "Method for Quantifying Heat Duty Derating due to Inter-Pass Leakage in Bolted Flat Cover Heat Exchangers", Heat Transfer Engineering, pp. 19-23, Vol. 4, No. 3-4 (1983).

38. "Foundation Stresses under Support of Freestanding Equipment Subjected to External Loads", (with K. P. Singh and I. Gottesman), International Journal of Pressure Vessels and Piping, Vol. 20, No. 2 (1985) pp. 127-138.

39. "On Some Performance Parameters for Closed Feedwater Heaters, Journal of Pressure Vessel Technology, Trans. ASME (1987).

40. "A Design Procedure for Evaluating the Tube Axial Load Due to Thermal Effects in Multi-Pass Fixed Tubesheet Heat Exchangers", (with A. I. Soler), Journal of Pressure Vessel Technology, Trans. ASME (1987).

41. "An Elastic-Plastic Analysis of the Integral Tubesheet in U-Tube Heat Exchangers - Towards an ASME Code Oriented Approach", Int. Journal of Vessel and Piping (c. 1987).

42. "Feedwater Heaters", Heat Transfer Equipment Design, R. Shal et. al (editor), Hemisphere (c. 1988).

43. "Surface Condensers", Heat Transfer Equipment Design, R. Shal et. al (editor), Hemisphere (c. 1988).

44. "FLow Induced Vibration", Heat Transfer Equipment Design, R. Shal et. al (editor), Hemisphere (c. 1988).

45. "Mechanical Design of Heat Exchangers", Heat Transfer Equipment Design, R. Shal et. al (editor), Hemisphere (c. 1988).

46. "A Rational Method for Analyzing Expansion Joints":, (with A. Soler), ASME, Journal of Pressure Vessel Technology (c. 1988).

47. "An Analysis of the Improvement in the Thermal Performance of Surface Condenser Equipped with Tweener Supports", ASME Joint Power Generation Conference, Miami (Oct. 1987).

48. "Pressure Vessels - Design & Operation", Chemical Engineering, pp 62-70, Chemical Engineering, July 1990, McGraw Hill, N.Y.

49. "Spent Fuel Storage Options: A Critical Appraisal", Power Generation Technology, pp 137-140, Sterling Publications, U.K. (1990-91).

50. "Design Strength of Primary Structural Welds in Free-Standing Structures", with A.I. Soler and S. Bhattacharya, Journal of Pressure Vessel Technology, Trans. ASME (c' 1991).

51. "Seismic Qualification of Free-Standing Nuclear Fuel Storage Modules - The Chin Shan Experience", Nuclear Engineering International, U.K. (March, 1991).

52. "Transient Response of Large Inertia Cross Flow Heat Exchangers", with Y. Wang, A.I. Soler and K. Iulianetti, ASME 91-JPGC-NE-27 (1991).

53. "Some Results from Simultaneous Seismic Simulations of All Racks in a Fuel Pool", with A.I. I. Soler, INMM Spent Fuel Management Seminar X, Washington, D.C., January, 1993.

54. "A Case for Wet Storage", INMM Spent Fuel Management Seminar X, Washington, D.C., January, 1993.

55. "Application of Transient Analysis Methodology to Heat Exchanger Performance Testing" with I. Rampall and Benjamin H. Scott, ASME Joint Power Generation Conference, October, 1994.

56. "Predicting Thermal Performance of Heat Exchangers Using In-Situ Testing and Statistical Correlation", with K. Iulianetti and Benjamin H. Scott, ASME Joint Power Generation Conference (1994).

57. "An Overview of the HI-STAR Technology", INMM Conference, Washington, DC, January, 1997.

58. "A Structural Assessment of Candidate Fuel Basket Designs for Storage and Transport of Spent Nuclear Fuel", with Max Delong, INMM Conference, Washington, DC, January, 1998.

59. "Seismic Response Characteristics of HI-STAR 100 Cask System on Storage Pads", with Mark G. Smith and A.I. Soler, INMM Conference, Washington, DC, January, 1998.

60. "Analysis of Mechanical Impact Events in Spent Fuel Storage Equipment", with Charles Bullard and Jin Yop Chung (1997).

61. "Predicting the Structural Response of Free-Standing Spent Fuel Storage Casks Under Seismic Events", with Alan I. Soler and Mark G. Smith, 16th Conference on Structural Mechanics in Reactor Technology (SmiRT 16), Washington, DC August 12-17, 2001.

ALAN I. SOLER, Ph.D.

EXECUTIVE ENGINEER
HOLTEC INTERNATIONAL

EDUCATION

University of Pennsylvania
Ph.D. in Mechanical Engineering (1962)

California Institute of Technology
M.S. in Mechanical Engineering (1959)

University of Pennsylvania
B.S. in Mechanical Engineering (1958)

AREAS OF PROFESSIONAL CONCENTRATION

Dynamics of casks and fuel racks, impact, mechanical design of cask and MPCs, failure analysis of reinforced concrete structures, cask transporter design, NUREG-0612 compliance, crane design and stress analysis.

PROFESSIONAL EXPERIENCE

HOLTEC INTERNATIONAL
Marlton, New Jersey
1986 – Present — Executive Engineer

UNIVERSITY OF PENNSYLVANIA
Philadelphia, Pennsylvania
1966 - 1991 — Professor of Mechanical Engineering and Applied Mechanics

INGERSOLL-RAND RESEARCH CENTER
Princeton, New Jersey
May 1964–Sept. 1965 — Member of Technical Staff

DYNASTRUCTURES, INC., CONSULTANTS IN APPLIED MECHANICS
Philadelphia, Pennsylvania
May 1962–May 1964 — Research Specialist

ACADEMIC HONORS

Tau Beta Pi
Sigma Tau
Society of Sigma Xi

PROFESSIONAL SOCIETY MEMBERSHIPS/ACTIVITIES

Member, ASME, Fellow ASME, 1986
Treasurer, University of Pennsylvania Chapter, Sigma Xi, 1968-70.
General Arrangements Committee Member, 1969, ASME Vibrations Conference.
ASEE Local Activity Coordinator, 1968-1974.

Member, Rotordynamics Subcommittee, ASME Design Division, 1973-1974.
Local Arrangements Committee, 1971 Summer ASME Applied Mechanics Meeting.
Recording Secretary, ASME Applied Mechanics Division, Publication Committee, 1971-1972.
-Applied Mechanics Representative to ASME Power Division Subcommittee on Environmental Policy, 1974-1976.
Member, Turbine and Auxiliaries Committee, ASME Power Division, 1974-76, Papers Review
Member, Task Group on Heat Transfer Equipment, ASME, working group #1 (tubesheets), 1975-1998.
Member - Subcommittee on Pressure Vessels and Piping, Nuclear Engineering Division, ASME, 1976-1987, Chairman, 1984-1987.

TECHNICAL CONSULTING

Consultant to Solid Mechanics Group, Ingersoll-Rand Research Center, Princeton, New Jersey, September 1965 - December 1966.
Consultant to Condenser Engineering Department, Ingersoll-Rand Corporation, Phillipsburg, New Jersey, September 1965 – 1982. Consultant to Structural Mechanics Associates, November 1958 - January 1969.
Visiting Scientist, Mechanical Engineering Research Division, Livermore Laboratories, Livermore, CA, Summer 1973, 1974 (AEC "Q" Clearance).
Member of Consulting Group, Thermac Associates, 1975 - 1986.
Consultant to Joseph Oat Corp. - Manufacturers of Nuclear Heat Exchangers. Camden, New Jersey, 1975 - 1986.
Consultant to Heat Exchange Institute - Nuclear HEX, 1978-1979.
Consultant, Inc., Wilson Div., Reading, PA, 1979-1980.
Consultant, NADC, Willow Grove, PA, 1984-1986.

PATENTS

Patent #3,382,918, May 1968, Reinforcing Structure for Direct Flow Steam Dome for Condensers (with Mr. R. J. Stoker and Dr. B. Paul of Ingersoll-Rand Corporation).

DRY SPENT FUEL STORAGE TECHNOLOGY

1992-Present: Lead Analyst in Mechanical/Seismic/Structural analysis in support of Holtec=s Dry Storage submittals for dual-purpose casks (HI-STAR 100 for Storage and Transport) and for METCON casks (HI-STORM 100 for Storage).

1994: Performed cask tip-over and drop analysis to support §50.59 effort for defueling Shoreham Station using IF-300 casks.

1995: Principal Analyst for evaluating cask drop events for Connecticut Yankee.

1997: Co-developer of the dynamic formalism to predict peak cask deceleration from cask tip-over and drop event on ISFSI pads.

1996: Principal designer of HI-STAR 100 Impact Limiter.

1998: Developer of the "penetration area principle" to predict impact limiter response under cask drop events; method was verified using quarter-scale tests.

1999: Designer and principal analyst for Holtec International's autonomous "Cask Transfer Facility" (CTF).

HIGH DENSITY FUEL RACK STRESS ANALYSIS

- Principal developer of Holtec's rack dynamic analysis code DYNARACK. This code is widely recognized as the most sophisticated program for high density rack seismic analysis.
- Performed seismic analysis of high density racks for 36 Nuclear Power Plants in the period 1980 to present.
- Pioneered dynamic analysis techniques of elevated pool slabs. Qualified the elevated pool slabs of Quad City Units 1 and 2, Grand Gulf and Oyster Creek using dynamic reinforced concrete analysis (all approved by the USNRC).

LICENSING SUPPORT

- Provided licensing support on over forty high-density rack applications to the USNRC (in the past twenty years).
- Appeared as expert witness (support) for Pacific Gas & Electric in Diablo Canyon reracking license review (1987).

PUBLICATIONS/PRESENTATIONS

1. "On the Lobar and Longitudinal Vibrations of Solid Propellant Rocket Motors", (with H. B. Kingsbury and J. R. Vinson) Proceedings of the 6th Solid Propellant Rocket Conference, AIAA, Washington, D.C. (February 1965).
2. "On the Solution to Transient Coupled Thermoelastic Problems by Perturbation Techniques", (with M. A. Brull) presented at the Summer Applied Mechanics Meeting of ASME (June 1965) and published in the Journal of Applied Mechanics (June 1965).
3. "A New Perturbation Technique for Differential Equations with Small Parameters", (with M. A. Brull), Quarterly of Applied Mathematics XXIV, No. 2 (July 1966) and presented at the 5th National Congress on Applied Mechanics, Minneapolis, Minnesota (June 1966).
4. "On Rolling Contact and the Theorem of Angular Momentum", (with S. C. Batterman), Journal of Engineering Education 67, 9 (May 1967).
5. "Higher Order Effects in Thick Rectangular Beams", International Journal of Solids and Structures 4, (July 1968) pp. 723-739.
6. "On the Vibrations and Stability of Moving Bands", Journal of the Franklin Institute (October 1968).
7. "Higher Order Theories for Structural Analysis Using Legendre Polynomial Expansions", presented at ASME Winter Annual Meeting, Los Angeles, CA (November 1969), and published in Journal of Applied Mechanics (December 1969).

8. "One Dimensional Viscous Magnetofluidynamic Flow in an Annulus", (with S. Schwietzer), presented at the AIAA Fluid and Plasma Dynamics Conference, San Francisco, California (June 1969), and published in Journal of the Franklin Institute 289, No. 6 (June 1970).

9. "On the Solution of Finite Deformation Problems of Beams Using Rate Equations", (with J. Lehner), Journal of Applied Mechanics, (March 1970) pp. 207-210.

10. "Approximate Theory for Locally Loaded Plant Orthotropic Beams", (with H. Tsai), International Journal of Solids and Structures 6, (1970) pp. 1055-1068.

11. "Approximate Solution of the Finite Cylinder Problem Using Legendre Polynomials'< (with J. Fellers), AIAA Journal 8, No. 11 (November 1970) and presented at the 6th U.S. Congress on Applied Mechanics (June 1970).

12. "On Analysis of Cable Network Systems Using Galerkin's Method", (with H. Afshari), Journal of Applied Mechanics, (September 1970) pp. 606-612.

13. "On the Buckling of Rings", (with S. C. Batterman), ASCE Engineering Mechanics Journal (December 1970).

14. "Dynamic Response of Sincle Cables with Initial Sag", Journal of the Franklin Institute (October 1970).

15. "Analysis of Cable Dynamics and Optimum Towing Strategies for Tethered Submersibles", (with B. Paul), presented at the Ocean Engineering Symposium, University of Pennsylvania (November 19-20, 1970), and published in Journal of Marine Technology 6, 2 (April 1972) pp. 34-41.

16. "Circumferential Forces and Moments in Edge Loaded Conical Shell Elements", Journal of Applied Mechanics (March 1972) pp. 290-291.

17. "Pre-twisted Curved Beams of Thin-Walled Open Section", Journal of Applied Mechanics (September 1972) pp. 779-786.

18. "Thermal Stresses and Initial Deformation of Heated Condenser Tubes", Journal of Engineering for Power (April 1973) pp. 84-91.

19. "New Results on Applications of Multi-Segment Stepwise Integration to First Order Equations", (with G. J. Hutchins), Journal of Computer Methods in Applied Mechanics and Engineering (1972) pp. 307-316.

20. "Dynamics of Cables and Cable Systems", Shock and Vibration Digest 5, 3 (March 1973) pp. 1-9.

21. "Cable Network Vibrations Using Galerkin's Method of Polynomial Approximating Functions", (with H. Afshari), Journal of Applied Mechanics (June 1973) pp. 622-624.

22. "Analysis of Moderately Thick Shells of Revolution", (with G. J. Hutchins), Journal of Applied Mechanics (December 1973) pp. 955-961.

23. "Project Cyclops - A Design Study of a System for Detecting Extraterrestrial Life", contributing author, NASA Report CR114445 (October 1972).

24. "Vibration of Cable Gridworks with Small Initial Deformation", (with H. Afshari), Journal of Applied Mechanics (December 1973), and presented at Winter ASME Meeting, Detroit, Michigan (November 1973).

25. "Transverse Elastic Buckling of Plane Pipe Gridworks", (with H. Afshari, Journal of Structures, ASCE (April 1974).

26. "On Seal Forces in Removable End Closure in Very High Pressure Test Chambers", ASME Journal of Pressure Vessel Technology (February 1975).

27. "Limit Design of Condenser Hotwell Floors", ASME Journal of Engineering for Power (October 1975) pp. 628-633.

28. "Stability of Rotor-Bearing Systems with Generalized Support Flexibility and Damping and Aerodynamic Cross-Coupling", (with R. E. Warner), presented at ASME Lubrication Conference, Toronto (October 1974), and published in the ASME Journal of Lubrication Technology (July 1975) pp. 461-472.

29. "Tubesheet Design in U-Tube Heat Exchangers Including the Effect of Tube Rotational Restraint", published in Journal of Engineering for Industry 98, 4 (November 1976) pp. 1157-1160 and presented at Design Engineering Conference, Chicago, IL (April 1976).

30. "Effective Bending Properties for Stress Analysis of Rectangular Tubesheets", (with W. Hill), published in ASME Journal for Power 99, 3 (July 1977) pp. 365-370, presented at 1976 ASME Annual Meeting.

31. "Stress Analysis of a U-Tube Heat Exchanger Tubesheet with an Integral Channel and an Unperforated Rim", presented by Pressure Vessel and Piping Division, ASME Mexico City Conference (September 1976) (76-PV-58).

32. "Analysis of Beam Columns on Elastic Plastic Foundations with Application to Power Plant Condenser Support Plate Design", (with C. Shahravan), published in ASME Journal of Engineering for Power, 100 (January 1978) pp. 182-188.

33. "Analysis of Closely Spaced Double Tubesheets under Mechanical and Thermal Loading", presented at 1977 Joint Power Generation Conference, ASME, Los Angles, California (77-JPGC-NE-21).

34. "The Tubesheet Analysis Method in the New HEI Condenser Standards", (with M.D. Bernstein), presented at the 1977 Joint Power Generation Conference, ASME, Los Angeles, California, published in ASME Journal for Power 100 (April 1978) pp. 363-368.

35. "Design Curves for Stress Analysis of U-Tube Heat Exchanger Tubesheet with Integral Channel and Head", (with J. E. Soehrens) Journal of Pressure Vessel Technology 100 (May 1978) pp. 221-233.

36. "Design of Condenser Hotwell Floor for Pressure Loading", presented at ASME 1978 Annual Meeting, ASME Advances in Reliability and Stress Analysis H00119 (1979) pp. 203-215.

37. "A Preliminary Assessment of the HEI Tubesheet Design Method - Comparison with a Finite Element Solution", presented at ASME 1978 Winter Annual Meeting, ASME Advances in Reliability and Stress Analysis H00119 (1979) pp. 127-146.

38. "Analysis of Bolted Joints with Nonlinear Gasket Behavior", ASME Journal of Pressure Vessels 102 (August 1980) pp. 249-256.

39. "Stress Analysis of Rectangular Tubesheets for Condensers", Paper 80-C2/NE-14 presented at ASME Nuclear Engineering Conference, San Francisco, California (August 1980).

40. "A Finite Element Model for Thick Beams", (with D. Barrett) Computer Methods in Applied Mechanics and Engineering 25 (1981) pp. 299-313.

41. "A Design Concept for Minimizing Tubesheet Stress and Tubejoint Load in Fixed Heat Exchangers", (with K. P. Singh) 1982 ASME Pressure Vessel and Piping Conference, Orlando, Florida; Int. Journal for Pressure Vessel Technology, Trans. ASME (c. 1982).

42. "Dynamic Coupling in a Closely Spaced Two Body System Vibrating in a Liquid Medium: The Case of Fuel Racks", (with K. P. Singh) 1982 SMIRT Conference, Keswick, England (May 1982).

43. "A Finite Element Model for Thickwalled Axisymmetric Shell", (with D. J. Barrett), ASME Journal of Pressure Vessel Technology 104, (August 1982) pp. 215-222.

44. "Design Parameters Affecting Bolt Load in Ring Type Gasketed Joints", (with K. P. Singh), Journal of Pressure Vessel Technology, Trans. ASME (1984).

45. "Effect of Non-Uniform Inlet Air Flow on Air-Cooled Heat Exchanger Performance", (with K. P. Singh and T. L. Ng) presented at Joint ASME-JSME Transfer Conference, Hawaii (March 1983) and published in Conference Proceedings.

46. "A Method for Computing Maximum Water Temperature in a Fuel Pool Containing Spent Nuclear Fuel", (with K. P. Singh) presented at Fourth International Conference on Pressure Vessels and Piping, Portland, Oregon (June 1983), Nuclear Technology, ANS (c. 1984).

47. "Seismic Response of Free Standing Fuel Rack Constructions to 3-D Floor Motions", (with K. P. Singh) presented at the Fourth International Conference on Pressure Vessels and Piping, Portland, Oregon (June 1983) and published in Nuclear Engineering and Design 80, (1984) pp. 315-329.

48. "Analysis of Tube-Tubesheet Joint loading Including Thermal Loading", (with Xu Hong) published in Journal of Applied Mechanics (June 1984), and presented at 1984 Pressure Vessels and Piping Conference.

49. "Analysis and Design of Pressure Vessel Bolted Flanges with Non Linear Gasket Materials", 11th Conference on Production Research and Technology - Computer Based Factory Automation, Conference Proceedings, Carnegie Mellon University, Pittsburgh, PA (May 1984).

50. "Foundation Stresses under Support of Freestanding Equipment Subjected to External Loads", (with K. P. Singh and I. Gottesman), International Journal of Pressure Vessels and Piping, Vol. 20, No. 2 (1985) pp. 127-138.

51. "Finite Elements for Thick 3-D Shells", (with A. Khaskia), International Journal of Pressure Vessel Technology, 1985.

52. "Tube-to-Tubesheet Rolled Joints: Part I - Analysis Including Strain Hardening and Temperature Dependent Properties", (with S. Weinstock), Proceedings of ASME 1985 Pressure Vessel and Piping Conference H00329, New Orleans, LA.

53. "Tube-to-Tubesheets Rolled Joints: Part II - Experimental Analysis", (with K. Reinis), Proceedings of ASME 1985 Pressure Vessel and Piping Conference H00329, New Orleans, LA.

54. "An Elastic Plastic Analysis of the Integral Tubesheet in U-Tube Heat Exchangers - Towards an ASME Code Oriented Approach", (with K. P. Singh), Proceedings of ASME 1985 Pressure Vessel and Piping Conference H00329, New Orleans, LA.

55. "A Design Procedure for Evaluating the Tube Axial Load due to Thermal Effects in Multi-Pass Fixed Tubesheet Exchangers", (with K. P. Singh), ASME Journal of Pressure Vessel Technology (c. 1986).

56. "Tubesheet Analysis - A Proposed ASME Design Procedure" (with S. Caldwell and K. P. Singh), ASME Karl Gardner Memorial Symposium Proceedings (c. 1986). Channel and an Unperforated Rim, presented by Pressure Vessel and Piping Division, ASME.

57. "Some Results From Simultaneous Seismic Simulations of all Racks in a Fuel Pool", with K.P. Singh, INMM Spent Fuel Management Seminar X, Washington, D.C., January, 1993.

58. Application of Transient Analysis Methodology to Quantify Thermal Performance of Heat Exchangers, I. Rampall, K.P. Singh, A. Soler, and B. Scott, Heat Transfer Engineering, 1997.

59. "Seismic Response Characteristics of HI-STAR 100 Cask System on Storage Pads", with K.P. Singh and Mark G. Smith, INMM Conference, Washington, DC, January, 1998.

April 1, 2002

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

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

APPLICANT'S PREFACE TO THE TESTIMONY OF PAUL J. TRUDEAU ON SECTION D OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESS

Paul J. Trudeau

Paul J. Trudeau is a Senior Lead Geotechnical Engineer at Stone & Webster, Inc., a Shaw Group Company ("S&W") in Stoughton, Massachusetts. Mr. Trudeau 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, and 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.

II. TESTIMONY

A. SCOPE

Mr. Trudeau will address the allegations raised by the State in Section D of Unified Contention Utah L/QQ concerning PFS's seismic analysis of the storage pads, casks, and their foundation soils and the seismic analysis of the Canister Transfer Building and its foundation. In this testimony, Mr. Trudeau will respond to the allegations raised by the State in Sections D.1.b(i) (with respect to the potential lack of rigidity of the storage pads and its effect on the stability analysis of the pads), D.1.c(i) (with respect to the potential effect of soil cement around the pads once the pads undergo sliding motion), D.1.g (with respect to the effect of potential pad-to-pad interaction on the sliding analysis of the pads), and D.2.c (with respect to the potential out of phase motion between the CTB and the soil cement placed around the building's foundations).

B. EFFECT OF PAD RIGIDITY ON SEISMIC STABILITY ANALYSES

Mr. Trudeau will describe the seismic stability analyses performed by PFS with respect to the potential failure mechanisms for the pads and the CTB (sliding, overturning, or bearing capacity failure), and will demonstrate that significant conservatisms have been incorporated in those analyses and in the design of the foundations of the structures. He will also show that large factors of safety are provided by the design against such failures.

With specific reference to the State's allegation that the stability analyses incorrectly assume that the storage pads behave rigidly under design basis earthquake loads, Mr. Trudeau, drawing on the testimony of other witnesses, will testify that the assumption of pad rigidity is proper and that the earthquake dynamic loads have not been underestimated in the stability analyses. He will further demonstrate that the use of peak ground acceleration in the seismic analyses is appropriate.

C. UNSYMMETRICAL LOADINGS ON PADS

Mr. Trudeau will refute the State's claim that the presence of soil cement and cement-treated soil adjacent to the storage pads will introduce unsymmetrical loadings on the pads once the pads undergo sliding motion in an earthquake, and will show that the Newmark sliding block analyses for the pads is not rendered invalid for failure to consider unsymmetrical loadings.

D. PAD-TO-PAD INTERACTION

Mr. Trudeau will testify that the sliding analysis of the storage pads is not deficient due to a failure to analyze for potential pad-to-pad interactions because such interactions will be insignificant.

E. POTENTIAL CRACK FORMATION DUE TO OUT OF PHASE MOTION OF THE CTB RELATIVE TO THE SOIL-CEMENT CAP

Mr. Trudeau will examine the potential formation of cracks in the soil cement that is to be placed around the foundations of the Canister Transfer Building due to out of phase motion between the soil cement and the building and will show that such cracks, if forming, will have little or no impact on the soil cement's ability to provide passive resistance against sliding of the CTB.

April 1, 2002

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

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

TESTIMONY OF PAUL J. TRUDEAU ON SECTION D OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESS BACKGROUND

Q1. Please state your full name.

A1. Paul J. Trudeau.

Q2. By whom are you employed and what is your position?

A2. I am a Senior Lead Geotechnical Engineer at Stone & Webster, Inc., a Shaw Group Company ("S&W") in Stoughton, Massachusetts.

Q3. Please summarize your educational and professional qualifications.

A3. My professional and educational experience is described in the *curriculum vitae* attached hereto. As indicated there, I have twenty-nine years of experience in geotechnical engineering. My experience includes 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, and 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.

Q4. What is the basis of your familiarity with the Private Fuel Storage Facility?

A4. S&W is the Architect/Engineer for the Private Fuel Storage Facility ("PFSF") under contract with Private Fuel Storage, L.L.C. ("PFS" or "Applicant"). As such, it coordinates the facility design activities, including the studies needed to characterize the PFSF site and establish its suitability. My particular areas of concentration on the PFSF project are the analysis of soils – settlement, bearing capacity, and stability of foundations – as well as the conduct of soils investigations, laboratory testing of soils to measure static and dynamic properties, and the performance of computer-aided analyses of the behavior of soils and structures under static and dynamic loading conditions.

Q5. What is the purpose of your testimony?

A5. The purpose of my testimony is to respond to allegations raised by the State of Utah in Section D of Unified Contention Utah L/QQ with respect to the seismic analysis of the storage pads, casks, and their foundation soils and the seismic analysis of the Canister Transfer Building and its foundation. I am also filing separate testimony on the allegations raised by the State in Section C of Unified Contention Utah L/QQ. That testimony addresses: (1) the characterization of subsurface soils at the PFSF site through subsurface investigations, sampling and analyses; (2) the stress/strain behavior of the soils under design basis earthquake conditions; and (3) the use of soil cement and cement-treated soil to enhance the seismic behavior of the soils beneath and adjacent to the foundations of the safety-related structures at the PFSF.

II. SEISMIC STABILITY ANALYSES PERFORMED BY S&W FOR THE PFSF

Q6. What are the main stability analyses that you have conducted regarding the performance of safety-related structures at the PFSF during seismic events?

A6. Part of my duties as lead geotechnical engineer is to perform, or direct the performance of, analyses of the response of the PFSF structures to the forces imparted by postulated seismic events. In particular, I was responsible for the preparation of Stone & Webster 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"), and 05996.02-G(B)-13, Rev. 6, *Stability Analyses of Canister Transfer Building* (July 26, 2001) ("CTB Stability Calc. Rev. 6"). Copies of relevant excerpts from these two calculations are included as PFS Exhibits UU and VV.

Q7. Would you please describe how seismic stability analyses such as those are conducted?

A7. In the seismic stability analyses, we seek 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 excessive settlement or rotation of the structure's foundation.

Q8. You use the term failure. Is the intent of the analyses to determine whether the structure in question will actually undergo sliding, overturning or bearing capacity failure?

A8. No. The intent of the analyses is to establish what margin or "factor of safety" ("FS") is provided by the design of the structure's foundations against each of the failure modes. It is typical in the industry to use FS = 1.1 as the desired safety factor against each of the three failure modes that I mentioned for load combinations that include seismic loads from the design basis earthquake. For example, Section 3.8.5 of NUREG-0800, the Standard Review Plan ("SRP") for Nuclear Power Plants, indicates that the factors of safety against overturning and sliding are acceptable if they exceed 1.1 for load combinations that include seismic loads due to the design basis earthquake.

Q9. If, for example, a factor of safety of 1.1 against sliding is not demonstrated, does that mean that the structure will actually slide in a seismic event?

A9. No. 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 might occur. Even then, our analyses include additional conservatism in various parameters, such that even if the calculated factor of safety was less than 1, the structures likely would not slide during the seismic event. In addition, because of the cyclic nature of the seismic loading, each of the peak accelerations we use to estimate the dynamic loads from the earthquake exists only for one, very brief moment in time – typically less than 0.005 seconds – and then the earthquake accelerations reverse direction. Therefore, even if the forces due to the peak acceleration of the earthquake exceeded the resisting forces, a fraction of a second later the accelerations would decrease, and the corresponding inertial forces would decrease as well, such that the structure would not experience significant horizontal displacement. In addition, even for an earthquake as large as the design basis earthquake for the PFSF, there will be only one point in time where the acceleration will equal the maximum value – at every other point in time, the accelerations will be much less than the peak value – yet the analyses assume that the forces due to these peak accelerations act continuously for purposes of computing the factor of safety.

Q10. Do you analyze, for each type of failure mode, various combinations of earthquake loadings?

A10. Yes. In addition to a reference "static" case ("Case I" in PFS Exhibits UU and VV), in which only the weight of the structure and its effect on the soils beneath the foundation are determined, we run, for each seismic failure mode, three families of cases: one (labeled "Case II") for static loads plus dynamic horizontal forces due the earthquake; another (labeled "Case IIIA," "Case IIIB," and "Case IIIC") for static plus various combinations of horizontal and vertical uplift forces due to the earthquake; and another family (labeled "Case IVA," "Case IVB," and "Case IVC") for static plus various combinations of horizontal and vertical compression forces due to the earthquake.

Q11. Do you also perform variations of each case in which some of the assumptions or parameters are varied?

A11. Yes. In addition to a "base case" that reflects the design intent with respect to the soils and foundations, we also perform hypothetical, "what if" analyses, in which other behavioral modes are explored.

Q12. Does performance of those hypothetical "what if" analyses mean that they are regarded as constituting credible scenarios for the behavior of soils and structures in an earthquake?

A12. No. The hypothetical analyses may be performed for a variety of reasons, such as, for example, determining what additional margins may be present in the design for which credit is not taken. However, performance of a hypothetical analysis does not necessarily mean that it is regarded as credible.

Q13. What was the "base case" you analyzed with respect to the sliding stability of the cask storage pads?

A13. That case is described and analyzed on pages 15 through 28 of Cask Storage Pad Stability Calc. 6(B)-04, Rev. 9 (PFS Exh. UU). It is based on engaging the shear strength of the soils beneath the pads to provide resistance against sliding forces. To ensure that the full shear strength of the soils is available to provide resistance against sliding, an "engineered mechanism" will be provided through the replacement of the top layer (1 to 2 feet) of soil below the cask storage pads with a cement-treated soil mixture having a minimum compressive strength of 40 psi, which provides a shear strength that is nearly twice as strong as the underlying clayey soils. The details of the design, testing, and construction of this cement-treated soil layer are described in my testimony on Section C of Unified Contention Utah L/QQ.

Q14. What conservative assumptions are made in the base case?

A14. In addition to replacing the soils within one to two feet beneath the pads with cement-treated soil that provides nearly twice the shear resistance as the *in situ* clayey soils beneath the pads, the design intent is also to replace the top 3 ft. of soil below grade in the areas around the cask storage pads with a 2 ft.-4 in. thick layer of soil cement with a minimum compressive strength of 250 psi, topped with 8 in. of compacted aggregate. The purpose of this soil cement placed adjacent to

the pads is to provide a firm foundation for supporting the cask transporter that will move storage casks onto the pads. This soil cement installation will provide significant, additional, resistance against sliding of the pads in an earthquake; however, the base case conservatively does not take credit for the strength of the soil cement installed around the pads to resist these sliding forces. Thus, the base case analysis conservatively ignores the cohesive strength of the soil cement in calculating the dynamic active earth pressures that must be resisted to preclude sliding. In addition, it ignores the passive resistance provided by the soil cement adjacent to the pad, and it ignores the shearing resistance available between the sides of the pad parallel to the direction of sliding and the soil cement adjacent to the pads. The analysis also conservatively uses shear strengths of the clayey soils based on static strengths measured in direct shear tests, despite the well-known phenomenon that such clayey soils exhibit increases in shear strength of as much as 100% when subjected to rapid loadings, such as those imparted by the design basis earthquake.

Q15. Are similarly conservative assumptions also made in the base cases for the other potential failure mechanisms?

A15. Yes. Similarly conservative assumptions (such as the use of static shear strength for the soils) are also made in the bearing capacity and overturning failure cases.

Q16. Have you sought to estimate how much the factors of safety would increase in the various stability calculations if, for example, more realistic values of the shear strength of the soils were used?

A16. Yes. I performed several simple calculations to estimate how much the factors of safety against failure would increase if the shear strength of the clayey soils was increased 50% from the strengths obtained in the static strength tests to account for the well known phenomenon that the dynamic strength of clayey soils under rapid rates of loading comparable to the cycling applicable for earthquakes is 50% to 100% greater than the strength measured in static shear tests. The results are as follows:

For the pads (bearing capacity failure):

As shown in SAR Table 2.6-7 (also p. 107 of Calc. G(B)-04-9, PFS Exh. UU), of the cases that combine the earthquake components in accordance with the 40-40-100 rule recommended by ASCE 4-86 (p. 12 of G(B)-04-9, PFS Exh. UU), Load Case IVB had the lowest FS against a bearing capacity failure based on inertial forces (p. 69 of Calc. G(B)-04-9, PFS Exh. UU: FS = 2.1 using the static shear strength, c = 2,200 psf). Increasing the soil shear strength by 50% to 3,300 psf to account for the dynamic strength of this clayey soil, increases this FS to 3.63. Conversely, the earthquake accelerations would have to be increased by a factor of 1.74 (i.e., to a horizontal acceleration of 1.24g and a vertical acceleration of 1.21g) to reduce the FS to 1.1, and by a factor of 1.79 (i.e., a horizontal acceleration of 1.27g and a vertical acceleration of 1.24g) to reduce the FS to 1.0.

For the pads (sliding failure):

For the sliding stability of the pads, the critical case will be for 10 pads sliding in the north-south direction. Pages 32 and 33 of Calc. G(B)-04-9 illustrate that using the static shear strength of the clay soils, the factor of safety against sliding of an entire column of pads in the north-south direction is 1.51. If we increase the clay soil strength by 50% to account for the normal increase of strength for clayey soils to dynamic loadings such as these, the factor of safety for this case increases to 2.2. Conversely, the pad + soil cement + cement-treated soil inertial forces and the maximum cask dynamic forces from the 2,000-yr return period earthquake would have to be more than doubled (i.e., the horizontal earthquake acceleration would have to be increased to 1.44g) for this case to obtain a factor of safety against sliding equal to 1.1.

For the CTB (bearing capacity failure):

As shown in SAR Table 2.6-10 (also p. 48 of Calc. G(B)-13-6, PFS Exh. VV), of the cases that combine the earthquake components in accordance with the 40-40-100 rule recommended by ASCE 4-86, Load Case IVB had the lowest FS against

a bearing capacity failure based on inertial forces (p. 41 of Calc. G(B)-13-6, PFS Exh. VV: FS = 6.25, with a shear strength c = 3,180 psf.) (The soil shear strength c = 3,180 psf was adjusted from the c = 2,200 psf for these soils based on the CPT results, as described on p. 9 of the calculation.) Increasing the soil shear strength by 50% (to c = 4,770 psf) to account for the dynamic strength of these clayey soils increases the FS to 10.1. Conversely, the earthquake accelerations would have to be increased by a factor of 4.34 to reduce the FS to 1.1, and by a factor of 4.39 to reduce the FS to 1.0.

For the CTB (sliding failure):

As shown in p. 23 of G(B)-13-6, PFS Exh. VV, c = 1.36 ksf is the applicable static residual shear strength of the soil for the CTB sliding case that used the full passive resistance of the soil cement around the building. The factor of safety against sliding for that shear strength value is 1.26. Increasing the shear strength of the soil by 50% (c = 2.04 ksf) to account for the dynamic strength of the clayey soils, increases the FS against sliding to 1.61. Conversely, the earthquake accelerations would have to be increased by a factor of 1.46 to reduce the FS to 1.1, and by a factor of 1.61 to reduce the FS to 1.0 for c = 2.04 ksf.

Q17. Are there other conservatisms incorporated into the design practices and the codes and standards used in performing this and the other stability analyses?

A17. There are several major elements of conservatism in nuclear industry design practices and applicable codes and standards that are reflected in the stability analyses conducted by PFS. These conservatisms include those in the utilization of "lower bound" (as opposed to best estimate or mean) values of the soil properties, in analysis assumptions, and the definition of "failure". Such conservatisms form part of the intentional and recognized safety margin inherent in the NRC seismic evaluation process discussed in the testimony of Dr. Allin Cornell being filed simultaneously with this testimony. These conservatisms imply that the foundations will have much greater factors of safety against failure

than the analyses predict, and would not actually fail until the earthquake ground motions become far larger than the design basis motions.

Q18. What were the results of the base case analyses?

A18. The analyses show that the minimum factor of safety against sliding of the storage pads in the event of a design basis earthquake is 1.27 (versus a target of 1.1), ignoring, as indicated above, the passive resistance available due to the soil cement adjacent to the pad. This value is based on the dynamic loads acting in the east-west direction. Those acting in the north-south direction are somewhat lower, resulting in a factor of safety against sliding of a single pad in the north-south direction of 1.36. This means that the storage pads will not slide in the event of a design basis earthquake. It should be noted that the calculated factor of safety against sliding between the base of the concrete pad and the underlying cement treated soil layer is 1.98, meaning that the limiting factor in the resistance to sliding is the bond between the cement-treated soil and the native soil underneath, not the bond between the cement-treated soil and the concrete pad above it.

Q19. What other sliding cases did you analyze for the storage pads?

A19. We also considered a case in which we take credit for the passive resistance provided by the 2 ft.-4 in. layer of soil cement to be placed around the pads, in order to demonstrate the beneficial effect of placing this soil cement adjacent to the pads. Our calculations for that case, which include only the forces acting on the pad, not those on the underlying cement-treated soil, are presented on pages 29 and 30 of Cask Storage Pad Stability Calc. Rev. 9 (PFS Exh. UU), show that the minimum factor of safety against sliding in the north-south direction without including the passive resistance of the soil cement is 1.52, and that this factor of safety increases to 2.35 when the passive resistance due to the soil cement adjacent to the pad is included. It also demonstrates that the factor of safety against sliding in the east-west direction is increased to 3.3 when the passive

resistance of the soil cement is included; thus, the critical direction for sliding of the pads is the north-south direction.

The sliding stability of an entire column of 10 pads in the north-south direction also was considered. In this case, the resistance to sliding of the entire column (running N-S) of pads exceeds that of each individual pad because there is more area available to engage more shearing resistance from the underlying soils than just the area directly beneath the individual pads. The extra area is provided by the 5-ft long x 30-ft wide plug of soil cement that exists between each of the pads in the north-south direction. This analysis assumes that the soil cement east and west of the long column of pads provides no resistance to sliding, conservatively assuming that the soil cement somehow shears along a vertical plane at the eastern and western sides of the column of 10 pads running north-south. The resulting factor of safety increases from 1.36 for an individual pad in the north-south direction to 1.50 for an entire column of 10 pads.

We also considered a hypothetical sliding stability case, presented on pages 36 to 45 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, in which the cohesive portion of the strength of the clayey soils along the interface with the cement-treated soils underneath the pads is completely ignored. In this hypothetical case, resistance to sliding is provided only by the frictional portion of the shear strength of the clayey soils beneath the cement-treated soil layer underneath the pads, and it is based on an obviously conservative value of the friction angle for the underlying soils. Not surprisingly, the pads are shown to slide in an earthquake under these assumptions, whether a single pad or a row of pads is considered.

This analysis also includes an estimation of the horizontal displacements that will be experienced by a row of 20 pads under the assumptions described above. The estimation is based on a method described in the technical literature for assessing the displacement of dams and embankments during earthquakes. This analysis yields horizontal displacements of the pads on the order of 2 to 6 inches. Again,

these displacements apply only to a hypothetical case based on extremely conservative assumptions.

Another hypothetical analysis (pages 46-51 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU) was conducted in which it was assumed that the storage pads rest directly on cohesionless soils, instead of on cement-treated soil and the clays that exist at the PFSF site. For that case, based on a conservative, lower-bound friction angle of 30 degrees for the cohesionless soils that were postulated to exist directly at the base of the pads, horizontal displacements of the pads on the order of 1.9 to 2.2 inches are predicted.

Q20. What weight should be given to the various hypothetical cases you just described?

A20. These cases are important in that they illustrate various conditions that bound the characteristics of the PFSF site soils and their performance in a design basis earthquake. However, the case that represents the design basis of the pads, which in itself incorporates a number of conservative assumptions, demonstrates that the design of the foundations of the cask storage pads provides a more than adequate factor of safety against sliding of the pads and the casks they support in an earthquake.

Q21. What analyses did you perform of the bearing capacity of the cask storage pads?

A21. The bearing capacity analyses, which are presented on pages 52-98 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, consider both static load cases and two different sets of dynamic loads. One set of dynamic loads was that resulting from the inertial forces applicable to the peak ground accelerations from the design basis ground motion. The other set of dynamic loads was based on the maximum dynamic cask driving forces obtained by the designer of the pads for cases in which the pad supports 2, 4, and 8 casks.

Q22. What results did you obtain?

A22. For the case of dynamic loads based on inertial forces from the design basis ground motion, the lowest factor of safety against bearing capacity failure was

1.17 (Case II, p. 59 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU), and was obtained under the very conservative assumption that 100% of the earthquake loads act in both horizontal directions at the same time. More realistic cases, in which the loads were distributed among the three dimensions in accordance with procedures set forth in industry standards, yielded factors of safety against bearing capacity failure exceeded 2 (Case IVB, p. 69 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU).

In the second set of analyses, the dynamic loads were based on those developed by the pad designer for varying numbers of casks loaded onto the pads. Those analyses were based on the conservative assumption that the maximum dynamic forces will all occur at the same time at each node in the model used to represent the cask storage pads, which, therefore, represents an upper bound of the dynamic forces that can be applied to the pads. A minimum factor of safety against bearing capacity failure of 1.6 (Case IVB, p. 97 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU) was obtained, applying to the case in which 8 casks are loaded onto the pad.

Q23. What analyses did you perform of the overturning stability of the cask storage pads?

A23. Overturning analyses were based on the dynamic loadings from the design basis ground motion. The analyses showed that the factor of safety of the storage pads against overturning is 5.6, well in excess of recommended margins.

Q24. Would you please summarize the results of the stability analyses of the storage pads under design basis earthquake loadings?

A24. The analyses that we performed of the sliding stability, bearing capacity, and overturning stability of the foundations of the storage pads show that significant margins are available for those foundations in the event of a design basis earthquake. These factors of safety, which incorporate a number of conservative assumptions, assure that the pads and the storage casks will remain stable under the loads imparted by the design basis earthquake. Moreover, the results of the base cases plus the conservatisms built into the stability analyses (as demonstrated

just by increasing the shear strength of the soils to more realistic values) make it safe to predict that the storage pads will not experience failure under the loadings from an earthquake far more severe than the design basis earthquake.

III. RESPONSE TO STATE CLAIMS IN SECTION D RELATING TO SEISMIC STABILITY ANALYSES OF STORAGE PADS AND CASKS

Q25. In Section D of Unified Contention Utah L/QQ, the State alleges several deficiencies in the PFS seismic stability analyses for the storage cask pads and the CTB and its foundation. Are you familiar with those allegations?

A25. Yes.

Q26. What is your general response to the State's allegations?

A26. The claims raised by the State are either incorrect or seek to find fault with some of the hypothetical cases that are included in the seismic stability analyses but which do not represent the design basis case; therefore the claims are irrelevant. They are also inconsequential in that the deficiencies alleged to exist, even if present, would not materially affect the validity of the analyses.

Q27. In Subsection D.1.b(i) of Unified Contention Utah L/QQ, the State asserts that 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 because the Applicant's calculations incorrectly assume that the pads will behave rigidly during the design basis earthquake. The assumption of rigidity is alleged to lead to significant underestimation of the dynamic loading atop the pads, especially in the vertical direction. Is this claim correct?

A27. No. As discussed in the testimony of Dr. Wen-Shou Tseng filed simultaneously herewith, the storage cask pad deflections under design basis earthquake loads are very small and the pads can be considered as essentially rigid for analytical purposes (although Dr. Tseng's organization, International Civil Engineering Consultants, Inc. or ICEC, conservatively treated the pads as flexible for purposes of their structural design). Because the pads are essentially rigid, the premise to the State's assertion that our stability analysis are faulty is incorrect. In addition, it can be demonstrated that the dynamic loads have not been underestimated in these analyses.

Q28. State witnesses have testified that the estimate you used of the seismic loadings on the pads in the horizontal and vertical direction use the peak ground acceleration of the design basis motion, which underestimates the accelerations to which the pads and storage casks will be subjected. Is there a significant difference between the peak ground acceleration and the accelerations to which the pads will be subjected?

A28. No. The difference, if any, is not significant, because the appropriate response spectrum curve to be used for determining these acceleration values should be based on the damping applicable for the pad + casks + soil system. This damping should include both radiation damping and material damping; however, the bulk of the energy is dissipated due to radiation damping in this case. The radiation damping is calculated based on a relatively simple formulation, and for the best-estimate soil properties, it can be shown to be approximately 50% for vertical vibration of the pad + casks + soil system. This number varies only slightly for the lower-bound (52%) and upper-bound (48%) soil properties. For such high degrees of damping, the amplification that would occur for the pad + casks + soil system would be much lower than would apply based on the response spectrum plot for 5% damping referred to by the State [Trudeau/Chang Deposition 11/15/00 at 172:22] as a demonstration that a huge amplified response is applicable for the pad foundations. The fundamental frequency for this case is approximately 6.21 Hz, corresponding to a fundamental period of 0.16 sec. The response spectrum for the vertical earthquake time history for 50% damping indicates that the maximum acceleration should be 0.757g, which is a slight amplification over the 0.695g used to calculate the inertial forces applicable for the pad + soil cement in these analyses. This is not the smoothed design response spectrum, however. As shown in Table 1 in Calc. 05996.02-G(PO18)-3-1 for a period of 0.16 sec, the response spectrum of the PFS vertical time history overestimates the design response spectrum by approximately 13%. If this adjustment is taken into consideration, the applicable vertical acceleration for 50% damping would be 0.757g ÷ 1.13, or 0.67g. This value is less than the value of 0.695g that was used to calculate the inertial forces applicable for the pad + soil cement in these analyses.

At any rate, the response spectrum technique is a very conservative way of arriving at the dynamic loads applicable for the pads and underlying cement-treated soils in this case. An independent verification of the dynamic loads used in the sliding stability analyses presented in the Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, can be obtained from a review of the time histories of the forces used by Holtec in its soil-structure interaction analysis of the cask storage pad. Time histories of the forces at the base of the pad generated by Holtec (without the soil mass attached to the pad) show that the peak horizontal force at the base of the pads during the entire earthquake record was 3,310 kips, acting in the east-west direction at 4.675 seconds into the time history. (Holtec's time history of forces from their SSI analysis of the casks + pad + virtual soil mass underlying the pad show that these peak forces are less when the virtual soil mass attached to the pad is included.) The peak horizontal force acting in the north-south direction was less than this, equaling only 2,540 kips at 5.445 seconds into the time history. Therefore, the critical direction for sliding is in the east-west direction. The factor of safety against sliding of the pad for this worst-case loading from Holtec's SSI analysis is calculated as follows for the 30 ft x 67 ft pad:

$$FS_{Sliding} = \frac{\Sigma \text{ Resisting Forces}}{\Sigma \text{DrivingForces}}$$

$$FS_{\text{Sliding E-W w/oPassive}} = \frac{2.1\,\text{ksf} \times 30\,\text{ft} \times 67\,\text{ft}}{3{,}310\,\text{k} + 65.3\,\text{k}} = 1.25$$

(In this equation, 2.1 ksf is the shear strength of the soil, 3,310 kips is the earthquake's peak horizontal sliding force, and 65.3 kips is the force due to dynamic active earth pressure acting on the pad in the same direction as the earthquake's acceleration.)

The minimum factor of safety against sliding of the pad at any point in time resulting from the time history of forces from Holtec's SSI analysis, 1.25, is

nearly the same as the minimum factor of safety against sliding of 1.27 calculated on p. 23 of the Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, for the design basis case. Therefore, the use of the peak horizontal ground accelerations in determining the sliding forces of the pad and underlying cement-treated soil does not significantly underestimate the dynamic loads acting on the storage cask pad foundation, if it underestimates them at all.

Further, the minimum factor of safety against sliding applies to only a single point in time in the entire time history. At every other point in the time history, the factors of safety against sliding exceed this value. Plotting the factor of safety against sliding vs time based on the time history of forces from Holtec's SSI analysis without the virtual soil mass included, demonstrates that the average factor of safety against sliding is approximately 10 throughout the duration of the earthquake, greatly exceeding this minimum value, as shown in PFS Exh. WW.

Q29. By how much would you expect the seismic loadings would change if you used the natural frequency of the pads in the analyses?

A29. The time history of forces, described in my previous answer, which were developed by Holtec in their SSI analysis of the pad + casks, provides a more rigorous and correct determination of these dynamic forces than you would obtain from the use of the response spectrum at the appropriate damping value. The calculation of the factor of safety against sliding based on this time history of forces at the base of the pad + casks demonstrates that there is only a very slight reduction in the minimum factor of safety against sliding when these loads are used, from 1.27 to 1.25, compared to the use of inertial forces of the pad and cement-treated soil based on the peak horizontal ground accelerations.

Q30. Subsection D.1.c(i) of Unified Contention L/QQ asserts 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 in that Applicant's evaluation ignores 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. State witnesses have asserted that one of the consequences of this deficiency is that the Newmark sliding block analysis for the storage casks did not

consider the potential for unsymmetrical sliding and underestimated the displacement of the storage pads. How do you respond?

A30. In considering this hypothetical scenario, it is important to understand that the pads have a greater resistance to sliding along their base than does the soil cement. As indicated on p. 39 of the Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, "the soil cement cannot even resist sliding of itself during the earthquake **if only the frictional portion of the strength is assumed to be available** along its base." Thus, if the pads slide, so will the soil cement – they will move in concert – and the pads will not be impacting the soil cement. In this situation, it is proper to ignore the presence of the soil cement in estimating displacements of the pads. It is unreasonable for the State's witness to assume that the soil cement will have more resistance to sliding than the pads.

The Newmark sliding block analysis is included in the pad stability calculation for the hypothetical case where it is assumed that the shear strength available to resist sliding at the interface between the cement-treated soil and the in situ clayey soils is based only on the frictional portion of the clay strength, completely ignoring the cohesive strength of the clay. For this obviously conservative scenario, the factor of safety against sliding was less than 1, indicating that the pads might be expected to slide due to the earthquake. An estimation of the amount of sliding that might occur was made based on the method proposed by Newmark[1] for estimating displacements of dams and embankments during earthquakes.

Newmark defines "N·W" as the steady force applied at the center of gravity of the sliding mass in the direction which the force can have its lowest value to just overcome the stabilizing forces and keep the mass moving. If the surface is horizontal, then it is just as easy for the block to slide to the left as it is to the right. In this case there is symmetrical resistance to sliding, and this is the case

[1] Newmark, N. M., 1965, "Effects of Earthquakes on Dams and Embankments," *Fifth Rankine Lecture*, Geotechnique, Institution of Civil Engineers, London, 15(2), pp139-60.

that applies for the pads at the PFSF, because the site is essentially horizontal. For a block resting on an inclined plane, such as applies to a model of the slope of an embankment or a dam, the situation is different. It is much easier for the block on the slope to move downhill than it is to move uphill, because gravity helps in moving the block downhill. The force required to move the block uphill must overcome both the resistance to sliding of at the base of the block on the slope and gravity. In this case, the resistance to sliding is considered to be unsymmetrical, because it is more difficult to move the block back up the hill than to move it down the hill.

The soil cement at one side of the cask storage pad provides the same resistance to sliding as at the other; therefore, this clearly is a case of symmetrical sliding as defined by Newmark.

It is also worth remembering that this is not the design basis case for the pads. 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, and on the commitment to demonstrate by testing that this shear strength can be achieved and that it is achieved by construction. 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. Since the pads do not slide, the question is moot.

Q31. In paragraph D.1.g of Unified Contention L/QQ, the State asserts that PFS 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. What is your understanding of the bases for the State's claim?

A31. My understanding is that the State is claiming that the stability analysis of the storage pads failed to consider potential of pad-to-pad interaction, but assumed all pads in a quadrant move together as an integrated foundation. The State believes this is an erroneous assumption.

Q32. Is it?

A32. No. PFS's design basis for the pads provides a factor of safety against sliding that exceeds 1.1; therefore, the pads do not slide, but rather, they will move with the underlying soil during the earthquake. The only possible interaction between the pads is dependent on the shear deformation of the pad above its base and the soil cement plug between the pads. However, the concrete pads and the soil cement plug between the pads are both very rigid with respect to the seismic shear loading. For example, the SHAKE analyses included in Calc. 05996.02-G(PO18)-2, Rev. 1 include soil cement at the top of the profile. The results for the lower-bound, fault-parallel case indicate that the effective shear strains in the clayey soil layer underlying the soil cement averaged 0.13%. This case produced the highest shear strains in this clay layer of all of the various soil property and earthquake component cases analyzed. However, even for this case, the effective shear strains in the soil cement were only 0.0034%, which is insignificant when considering movements required to effect pad-to-pad interactions. Therefore, shear distortions within the soil cement and concrete pads due to the upward propagation of seismic waves should be very small. It is, therefore, anticipated that the pad and soil cement plug between the pads will deflect in phase with the underlying soils, meaning that the interaction between the pads will be insignificant.

IV. RESPONSE TO STATE CLAIMS IN SECTION D RELATING TO SEISMIC STABILITY ANALYSES OF CANISTER TRANSFER BUILDING AND CASKS

Q33. In paragraph D.2.c of Unified Contention L/QQ, the State asserts that 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. How do you respond to the State's claim?

A33. The State claims that various mechanisms can lead to the formation of cracks in the soil cement that surrounds the building: shrinking and curing of the soil cement during the placement process, differential settlement between the building foundation and the surrounding soil cement, bending stresses in an earthquake, motion between the building foundation and the surrounding soil cement. I disagree that earthquake bending stresses will lead to the formation of new cracks,

or that differential settlement between the building foundation and the soil cement layer will lead to crack formation. At any rate, as I discussed before, these are all thin, vertical, random cracks that do not affect the ability of the soil cement to provide the passive resistance to sliding relied upon in the design.

Q34. Why will not new cracks be formed due to earthquake bending stresses?

A34. The effect of bending stresses on the soil cement surrounding the CTB mat will be to alternately open and close the tops and bottoms of any shrinkage cracks that may have occurred in the soil cement in the area, not to form new cracks.

Q35. And why will there be no new cracks due to differential settlement?

A35. Because, as the CTB foundation mat is loaded, the soils within the profile adjacent to the mat also will experience increases in stresses, as the loading gets distributed over a wider area deeper in the soil profile. This stress distribution results in settlement of the soil cement areas adjacent to the mat which will approximate those at the edge of the mat, so that there will not be an abrupt differential settlement noted at the joint between the edge of the mat and the soil cement. These settlements will gradually decrease with increasing distance from the edge of the mat. The resulting settlement profile will be dish-shaped, concave downward, extending some distance away from the edge of the mat, so no cracks will form due to differential settlement. The concave downward shape of the settlement profile will result in closing of the lower portion of the nearly vertical shrinkage cracks. This lower portion of the soil-cement profile provides a greater percentage of the resistance due to increased passive pressure at depth; therefore, this settlement is beneficial in improving the ability of the soil cement to provide passive resistance.

Q36. Why would there be no effect on the passive resistance of soil cement around the CTB if new cracks are formed or existing cracks reopen?

A36. Because the passive resistance of soil cement is not diminished by the presence of a crack. The effect of cracks opening as seismic waves pass through the soil-cement layer is, at most, to cause the building to displace a small distance to close

each crack, and then the full passive resistance of the soil cement to sliding is restored.

Q37. Does that mean that the CTB might actually slide some distance?

A37. Theoretically, the CTB might move a small distance – measured in fractions of an inch to inches – in order that the cracks in the soil cement be closed and full passive resistance be restored. Were that to happen, however, there would be no safety-related consequences, because there are no connections between the CTB and any other safety-related systems, structures, or components that would be adversely impacted by such horizontal movement.

Q38. Have concerns been expressed by the State regarding potential failure mechanisms for the CTB other than sliding?

A38. Yes. State witnesses have raised concerns about potential overturning of the CTB in a seismic event. However, my understanding is that the concerns refer to some of the assumptions made in the overturning calculations that are part of the stability analysis of the CTB, not with the calculation results, which show that there is a significant factor of safety in the CTB design against overturning (FS_{OT} = 1.95, p. 15 of CTB Stability Calc. Rev. 6, PFS Exh. VV). While I disagree with the concerns, I agree with the conclusions expressed by the State's witnesses that overturning of the CTB during a design basis earthquake is not a realistic concern.

Q39. Is bearing capacity failure of the CTB a concern?

A39. No. To my knowledge, neither the State nor any of its witnesses has raised bearing capacity as a failure mechanism of concern for the CTB. This is not surprising, since our calculations show that for all cases analyzed, the factor of safety against bearing capacity failure of the CTB is 5.5 (Load Case II, SAR Table 2.6-10 and p. 48 of CTB Stability Calc. Rev. 6, PFS Exh. VV) or greater. Thus, bearing capacity failure of the CTB is not a credible scenario.

Q40. Would you please summarize the results of the stability analyses of the CTB under design basis earthquake loadings?

A40. The analyses that we performed of the sliding stability, bearing capacity, and overturning stability of the CTB show that adequate factors of safety are available for those foundations in the event of a design basis earthquake. These factors of safety, which incorporate a number of conservative assumptions, assure that the CTB will not be subject to failure under the loads imparted by the design basis earthquake. Moreover, the results of the base cases plus the demonstrated conservatisms built into the stability analyses (as demonstrated just by increasing the shear strength of the soils to more realistic values) make it safe to predict that the CTB will not experience failure under the loadings from an earthquake significantly more severe than the design basis earthquake.

Q41. Does that conclude your testimony?

A41. Yes, it does.

Paul J. Trudeau

Senior Lead Geotechnical Engineer

Years Experience (as of February 2002)

At Stone & Webster: 29 With other Firms: 0

Department/Division/Location

Civil and Transportation/Division 52/Boston

Professional History

Stone & Webster, Boston, Massachusetts – 1973 to Present
Massachusetts Institute of Technology – Cambridge, Massachusetts – 1971 to 1973
Stone & Webster, Boston, Massachusetts – 1971 to 1972
Worcester Polytechnic Institute, Worcester, Massachusetts – 1967 to 1971

Areas of Expertise

- Geotechnical Engineering and Design
- Use of Computers In Geotechnical Analyses and Designs
- Managing Geotechnical Investigations
- Geotechnical Instrumentation
- Performing Cross-Hole Shear Wave Velocity Surveys
- NRC Regulatory Compliance, Review, and Implementation for Nuclear Power Plants and Independent Spent Fuel Storage Installations

Awards

Desmond Fitzgerald Medal awarded by the Boston Society of Civil Engineers for "Shear Wave Velocity and Modulus of a Marine Clay," Journal of the Boston Society of Civil Engineers, January 1974.

Computer Hardware/Software Capabilities

Mr. Trudeau has considerable experience using PC and mainframe computer programs for performing geotechnical analyses. He is extremely proficient at developing spreadsheets using Microsoft Excel for solving complex engineering calculations and also is an expert FORTRAN programmer and in programming IBM JCL. He also has considerable experience in using MicroStation for generating report-quality sketches and figures and in using InRoads for plotting contours, subsurface profiles, and determining earthwork quantities.

He is adept at developing batch programs, as well as programming in dBASE, AWK, perl, and developing shell scripts in Unix. He routinely uses these techniques for automatic placement of graphics at correct locations and scales in MicroStation design files for generation of geotechnical figures, such as boring location plans, subsurface profiles, contour maps, and other figures for reports.

Department/Division Assignments

Geotechnical Division Computer Coordinator

Training

40 hours of instruction in Waste Site Worker Protection and 8 hours of instruction in Supervisory Training to comply with OSHA 1910.120(e)(2&3)

Experience Summary

Mr. Trudeau has over 29 years of experience in the engineering industry. Currently, as a Senior Lead Engineer in the Civil and Transportation Department of Stone & Webster, he is Lead Geotechnical Engineer on several Independent Spent Fuel Storage Facility projects. In prior years, as the Geotechnical Division Computer Coordinator, he was responsible for the development, documentation, and maintenance of more than 80 geotechnical computer programs sponsored by the Geotechnical Division of Stone & Webster and for providing consulting for geotechnical computer applications.

Since joining Stone & Webster in 1973, he has served as a Lead Geotechnical Engineer for Independent Spent Fuel Storage Installations (ISFSI) at the Duane Arnold Energy Center in Palo, Iowa, the Private Fuel Storage Facility in Skull Valley, UT, and at Maine Yankee's nuclear plant in Wiscasset, ME; for numerous combined-cycle power plants; and for the Bellefonte Nuclear Plant, Shoreham Nuclear Power Station, Falcon Seaboard Gas Pipeline, TVA Widows Creek Steam Plant, and various projects at the Hanscom Air Force Base. He has also served as a Geotechnical Engineer on several nuclear and fossil power plant projects. In these roles, he was responsible for performing geotechnical investigations, preparing geotechnical analyses, developing geotechnical design criteria for other disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical, and for preparing geotechnical sections of Preliminary and Final Safety Analysis Reports and Environmental Reports. This work was performed in accordance with quality assurance programs that satisfied the quality assurance requirements of Appendix B of 10CFR Part 50 and NQA-1.

He was also responsible for reviewing geotechnical analyses and reports prepared by others on these projects, and for preparing testimony and for testifying at depositions and public hearings. He has also completed 40 hours of instruction in Waste Site Worker Protection and 8 hours of instruction in Supervisory Training to comply with OSHA 1910.120(e)(2&3) and is certified to work on hazardous waste sites.

Mr. Trudeau's field experience includes performing cross-hole shear wave velocity tests in Maine, Connecticut, and Texas; geotechnical boring supervision at Jamesport, Shoreham, and Shoreham West on Long Island in New York and at Wards Island in New York, New York; and a compaction control investigation and intake canal revetment repair at Shoreham Unit No. 1. He has performed inspections of the haul road for transport of 300-ton steam generators at the North Anna Nuclear Power Station in Virginia and has inspected the route proposed for transport of the 800-ton reactor pressure vessel from the Shoreham Nuclear Power Station to Chem-Nuclear's disposal facility in Barnwell, South Carolina. In addition, he has served as Lead Scientist/Field Supervisor of environmental borings that were drilled for site assessment studies performed for New York City Department of Environmental Protection at their Jamaica, Wards Island, and 26th Ward water pollution control plants.

Mr. Trudeau's laboratory experience includes performing index property tests, consolidation tests, resonant column (Hardin Oscillator) tests, and static and dynamic triaxial tests. He was instrumental in selection, installation, testing, and debugging of Stone & Webster's geotechnical laboratory data acquisition system. His educational experience encompasses many aspects of civil engineering, including soil mechanics and foundations, computer programming (FORTRAN), soil dynamics, earthquake engineering, geotextiles, and structures.

Education

Master of Science in Civil Engineering, MIT, Cambridge, Massachusetts – 1973
B.S. in Civil Engineering, Worcester Polytechnic Institute, Worcester, Massachusetts – 1971

Licenses, Registrations, and Certifications

Professional Engineer: Massachusetts – 1977
Maine – 1999
Iowa – 2002

Professional Affiliations

Chi Epsilon: Member – 1969
American Society of Civil Engineers: Member 1971
Boston Society of Civil Engineers Section/ASCE: Member 1971
International Society of Soil Mechanics and Foundation Engineering: Member 1974

BSCES Director
BSCES Awards Committee – Chairman
BSCES Student Chapter Committee – Chairman
BSCES Membership Committee – Member
BSCES Task Force for Younger Members – Member
ASCE National Convention Attendance Committee – Co-Chairman
BSCES Geotechnical Engineering Practice Lecture Series Committee – Member

Publications

Trudeau, P.J., Whitman, R.V., and Christian, J.T., "Shear Wave Velocity and Modulus of a Marine Clay," Journal of the Boston Society of Civil Engineers, January 1974.

Pierce, D.S., and Trudeau, P.J., "Digital and Analog Methods for the Development of Stereoscopic Contour Maps for Geological and Geophysical Analysis," Geological Society of America Abstracts with Programs, Vol. 10, No. 7, 1978.

Experience History

STONE & WEBSTER, BOSTON, MASSACHUSETTS – 1973 TO PRESENT

Mixed Oxide (MOx) Fuel Fabrication Facility (Oct 2001 to Present)
U. S. Department of Energy

As Lead Geotechnical Engineer, responsible for geotechnical engineering efforts associated with the licensing and design of the $500M MOx Fuel Fabrication Facility (MFFF). Responsible for performing geotechnical investigations, preparing geotechnical analyses, developing geotechnical design criteria for other disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical, and for preparing geotechnical reports. This work was performed in accordance with quality assurance programs that satisfied the quality assurance requirements of Appendix B of 10CFR Part 50 and NQA-1.

The MFFF work is being done under a 3-year, $125M base contract for the U.S. Department of Energy. Options 1 and 2 of the contract will include construction management and operation of the MFFF, respectively. The facility will be based on the proven technology of the COGEMA Melox Plant in southern France. The facility will be licensed by the NRC.

Independent Spent Fuel Storage Installation – (March 2001 to Present)
Duane Arnold Energy Center, Palo, IA

As Lead Geotechnical Engineer, responsible for development of field programs and associated engineering services scopes of work for performing subsurface investigations required to document existing conditions for licensing and design of the foundations of the proposed Independent Spent Fuel Storage Installation (ISFSI), and to comply with US Nuclear Regulatory requirements. This effort included reviewing existing geotechnical data, development of the subsurface exploration plan, including boring programs, verticality survey, geophysical survey, and laboratory testing, comparison of bids, selection of the drilling, laboratory testing, and geophysical contractors, and drilling and sampling soil and rock at the site, review and reporting of the results of the laboratory testing of soils from the site, as well as the results of the cross-hole and down-hole seismic surveys that were performed at the site. Also developed the groundwater monitoring procedure and incorporated data collected in the geotechnical report. Responsible for development of the seismic design basis of the facility and for preparation of geotechnical calculations, design criteria, and geotechnical report.

Private Fuel Storage Facility – Skull Valley, UT (Dec 1997 to Present)
Private Fuel Storage, Limited Liability Corporation

As Lead Geotechnical Engineer for the Private Fuel Storage Facility – Skull Valley, responsible for preparation of responses to questions received from the Nuclear Regulatory Commission (NRC) and intervenors regarding geotechnical sections of the Safety Analysis Report and the Environmental Report. Participated in litigation, including developing responses to discovery requests and interrogatories from intervenors and providing depositions, as well as attending meetings and public hearings with the NRC and intervenors, responding to questions regarding geotechnical issues on this project. Recommended and developed proposal to use soil cement to stabilize the near-surface eolian silts to support the cask storage pads at the elevations required for flood protection and to provide enhanced stability against sliding due to the loads associated with the design basis ground motion. Developed additional field programs, including borings, cone penetration testing, and geophysical surveys, and associated engineering services scopes of work (ESSOWs). The cone penetration testing included standard tip and sleeve resistance measurements, resistivity measurements, as well as down-hole seismic shear and compression wave velocity tests and dilatometer tests. Responsible for review of the verification and

validation of the software used to process the cone penetration test data. Participated in resolution of survey problems with respect to the locations of the borings performed at the site. Developed laboratory testing programs and associated engineering services scopes of work. Prepared comparisons of bids and participated in negotiations with bidders prior to award of contracts. Supervised execution of laboratory testing and prepared engineering calculations, incorporating results of these studies in developing responses to questions from the NRC and intervenors. Also updated the respective sections of calculations, the Environmental Report, Geotechnical Report, and the Safety Analysis Report to incorporate the change of the design basis ground motion from the original deterministic earthquake to the probabilistic seismic hazard analysis 2,000-yr return period earthquake. These ESSOWs, laboratory testing, and analyses were prepared in accordance with a quality assurance program that satisfied the quality assurance requirements of Appendix B of 10CFR Part 50.

Maine Yankee Decommissioning Project (Oct 1999 to Mar 2000)
Maine Yankee Atomic Power Company – Wiscasset, ME

As Lead Geotechnical Engineer, responsible for development of the subsurface investigation performed using geoprobes to ascertain the soil types of the near-surface soils and the depth to ground water and rock in areas proposed for temporary storage of rubblized concrete from the demolition of existing structures at the plant. This effort was required as part of the development of the solid waste storage permit required by the State of Maine Department of Environmental Protection. Prepared geotechnical sections of the solid waste storage permit application.

Independent Spent Fuel Storage Installation (Sept 1998 to June 2001)
Maine Yankee Atomic Power Company – Wiscasset, ME

As Lead Geotechnical Engineer for the Independent Spent Fuel Storage Installation for Maine Yankee Atomic Power Company in Wiscasset, ME, responsible for all geotechnical activities associated with permitting, licensing, design, and construction of the Independent Spent Fuel Storage Installation (ISFSI). This effort included reviewing existing geotechnical data, development of the subsurface exploration plan and boring and test pit ESSOWs, comparison of bids, selection of the drilling contractor, and drilling and sampling soil and rock at the site. Mr. Trudeau also prepared a project-specific procedure for performing a cross-hole shear wave velocity survey, and he performed that survey to obtain soil properties for dynamic analyses. Also responsible for preparation of the procedure for monitoring observation wells and reducing the data generated by that program and reporting it to the State of Maine Department of Environmental Protection as part of the Site Location of Development Permit process. Responsible for responding to nonconformity and disposition reports, preparation of specifications for construction, including earthwork and subdrain installations. Responsible for preparation of geotechnical calculations, including SHAKE analyses, slope stability analyses, and analyses of bearing capacity, settlement, and reduction of cross-hole velocity data. Also responsible for preparation of geotechnical design criteria and the geotechnical report.

Independent Spent Fuel Storage Installation (Sept 1999 to Feb 2000)
Indian Point Nuclear Power Station, Consolidated Edison, NY

Assisted the Lead Geotechnical Engineer in preparation of the boring location plan and ESSOW for the preliminary subsurface investigation for the proposed ISFSI.

Mystic, Edgar, and Medway Combined Cycle Power Plants (Mar 1998 to Dec 1998)
Sithe Energies, Inc

Geotechnical Engineer

Terminal A Area 8 (Mar 1998 to Oct 1998)
MASSPORT

Geotechnical Engineer, responsible for development of specifications and geotechnical support during construction.

Combined-Cycle Power Plant (Feb 1998 to Feb 2000)
EMI, Rumford, ME and Tiverton, RI

Lead Geotechnical Engineer, responsible for site investigations, development of design criteria and specifications, and providing geotechnical support during construction.

Santeetlah Dam (Dec 1997 & July/Aug 1998)
Tapoco Developments

Geotechnical Engineer and Computer Consultant

Cheoah Dam (Aug 1997 to Sept 1997)
Tapoco Developments

Geotechnical Engineer and Computer Consultant

Big Brown Steam Electric Station, Fairfield, TX (July 1997 to Nov 1998)
TU Electric Company

Geotechnical Engineer, responsible for site investigations, development of design criteria and specifications, and providing geotechnical support during construction.

Building 99 Fuel Oil Storage Facility (June 1997 to Aug 1997)
GE River Works Plant – Lynn, MA

Lead Geotechnical Engineer, responsible for site investigations, development of design criteria and specifications, and providing geotechnical support during construction.

VX Full Scale Plant (April 1997 to January 2000)
U.S. Army Program Manager for Chemical Demilitarization, Newport, IN

Lead Geotechnical Engineer, responsible for site investigations and geotechnical analyses.

Private Fuel Storage Facility – Skull Valley, UT (Jan 1997 to Oct 1997)
Private Fuel Storage, Limited Liability Corporation

Geotechnical Engineer

Building 66 G & L G60TX Foundation (Dec 1996 to Jan 1997)
GE River Works Plant – Lynn, MA

Lead Geotechnical Engineer, responsible for site investigations, development of design criteria and specifications, and providing geotechnical support during construction.

Calderwood Dam (Nov 1996 to Feb 1997)
Tapoco Developments

Geotechnical Engineer and Computer Consultant

9th St Substation (Oct 1996 to Jan 1998)
Potomac Electric Power Co, Washington, D. C.
Geotechnical Engineer

Boston Ramps (Feb 1996 to Dec 1996)
Massachusetts Turnpike Authority
Geotechnical Engineer

Goodhue County Independent Spent Fuel Storage Installation (Dec 1995 to Sept 1996)
Northern States Power Company
Geotechnical Engineer

Central Artery/Third Harbor Tunnel Project (Feb 1994 to January 1997)
Mass. Department of Public Works
Manager of Computer Services for Area 5 Geotechnical Consultant

Granite State Gas Transmission Company (Nov 1993)
Computer Consultant & Database Manager

Bellefonte Nuclear Plant (Oct 1993 to Mar 1994)
Tennessee Valley Authority
Lead Geotechnical Engineer

Chubb & Son, Incorporated (Sept 1993 to Jan l994)
Geotechnical Consultant

Pease Air Force Base (Aug 1993)
United States Air Force
Geotechnical Engineer

Petersburg Generating Station (July 1993 to Sept 1993)
Indianapolis Power and Light Company

Green Mountain Power Corporation (July 1993)
Geotechnical Engineer, responsible for site investigations.

E. W. Stout Generating Station (July 1993)
Indianapolis Power and Light Company
Geotechnical Engineer and Computer Consultant

Hanscom Air Force Base (Apr 1993 to July 1993)
United States Air Force
Lead Geotechnical Engineer, responsible for site investigations and development of design criteria and the geotechnical report.

Portland Natural Gas Transmission System (Nov 1992 to Apr 1993)
Computer Consultant & Database Manager

Maine Low-Level Radioactive Waste Authority (Oct 1992 to May 1993)
Geotechnical Engineer

Afobaka Dam (Oct 1992 to Jan 1993)
Suriname Aluminum Company
Geotechnical Engineer and Computer Consultant

Widows Creek (Sept 1992 to Feb 1993)
Tennessee Valley Authority
Lead Geotechnical Engineer

General Support Services Contract, Richland Field Office (Sept 1992 to Oct 1992)
U. S. Department of Energy
Geotechnical Engineer

Patriot Generating Station (June 1992 to Aug 1992)
Indianapolis Power and Light Company
Geotechnical Engineer and Computer Consultant

Bellefonte Nuclear Plant (Feb 1992 to July 1992)
Tennessee Valley Authority
Lead Geotechnical Engineer

Petersburg Generating Station (Sept 1991 to May 1992)
Indianapolis Power and Light Company
Geotechnical Engineer and Computer Consultant

North Anna Nuclear Power Station (Sept 1991)
Virginia Power Company
Geotechnical Engineer

EG & G Rocky Flats (Sept 1991)
US Department of Energy
Geotechnical Engineer (SHAKE Analyses)

New Production Reactor (Feb 1991 to Oct 1991)
US Department of Energy
Geotechnical Engineer and Computer Consultant

Widows Creek Steam Plant – Unit 8 (Feb 1991 to June 1991)
Tennessee Valley Authority
Lead Geotechnical Engineer

Hanscom Air Force Base (Jan 1991 to Feb 1991)
United States Air Force
Lead Geotechnical Engineer

Central Artery/Third Harbor Tunnel Project (Mar 1990 to Feb 1992)
Mass. Department of Public Works
Manager of Computer Services for Area 5 Geotechnical Consultant

Hanscom Air Force Base (Jan 1990)
United States Air Force
Lead Geotechnical Engineer

Sludge Management Project (Sept 1989 to July 1990)
New York City Department of Environmental Protection
Geotechnical Engineer / Geotechnical Field Inspector / Lead Scientist/Field Supervisor

Plattsburgh 12 In. Diameter Gas Pipeline (Feb 1989 to Apr 1990)
Falcon Seaboard Pipeline Company
Lead Geotechnical Engineer

Great Northern Paper Company (Feb 1989 to May 1989)
Geotechnical Engineer

Salt Cave Hydroelectric Project (Apr 1986 to May 1986)
City of Klamath Falls, Oregon
Geotechnical Engineer

Bradley Lake Project (Feb 1986 to Oct 1986)
Alaska Power Authority
Geotechnical Engineer

Beaver Valley Power Station – Unit 2 (Oct 1984 to Aug 1985)
Duquesne Light Company
Geotechnical Engineer

Shoreham Nuclear Power Station – Unit No. 1 (Jan 1983 to Mar 1992)
Long Island Lighting Company
Lead Geotechnical Engineer

Malakoff Site (Apr 1982 to Dec 1982)
Houston Lighting & Power Company
Geotechnical Engineer

Office of Nuclear Waste Isolation (ONWI) of Battelle Memorial Institute (Jan 1982 to Oct 1987)
U.S. Department of Energy
Geotechnical Computer Consultant

Western Fuels Association. Inc. (Dec 1980)
Geotechnical Computer Consultant

Patriot Station (Nov 1980 to July 1981)
Indiana Power and Light Company
Geotechnical Computer Consultant

Site X (Oct 1980 to Dec 1981)
Houston Lighting & Power Company
Geotechnical Engineer

Pumped Storage Project (Apr 1980 to July 1980)
Public Service Company of New Mexico
Geotechnical Computer Consultant

Beaver Valley Power Station – Unit No. 2 (Feb 1980 to Mar 1980)
Duquesne Light Company
Geotechnical Computer Consultant

Millstone Unit No. 3 (Feb 1980)
Northeast Utilities Service Company
Geotechnical Engineer

Martin Cooling Dike (Jan 1980)
Florida Power and Light Company
Geotechnical Engineer

Beaver Valley Power Station – Unit No. 1 (Mar 1979 to May 1979)
Duquesne Light Company
Geotechnical Computer Consultant

Haven Nuclear Power Station (Dec 1978 to Jan 1979)
Wisconsin Electric Power Company
Geotechnical Engineer

Office of Nuclear Waste Isolation (ONWI) of Battelle Memorial Institute (Sept 1978 to Nov 1979)
U.S. Department of Energy
Geotechnical Computer Consultant

Stuyvesant & New Haven Sites (Apr 1978 to Sept 1978)
New York State Electric and Gas Corp.
Geotechnical Computer Consultant

Sundesert 500 kV Transmission and Substation Project (Aug 1977 to Dec 1977)
San Diego Gas and Electric Company
Geotechnical Computer Consultant

Shoreham Nuclear Power Station (Oct 1973 to June 1976)
Long Island Lighting Company
Geotechnical Engineer

Jamesport Nuclear Power Station (Aug 1973 to Apr 1977)
Long Island Lighting Company
Geotechnical Engineer

Northfield Mountain Pumped Storage Project (Aug 1973 to Oct 1973)
Northeast Utilities Service Company
Geotechnical Engineer and Computer Consultant

Geotechnical Division Computer Coordinator (Mar 1973 to Jan 1999)

North Anna Power Station (Feb 1973)
Virginia Electric and Power Company
Geotechnical Engineer

Massachusetts Institute of Technology – Cambridge, Massachusetts – 1971 to 1973
Graduate Research Assistant

April 1, 2002

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

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

APPLICANT'S PREFACE TO THE TESTIMONY OF BRUCE EBBESON ON SECTION D OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESS

Bruce E. Ebbeson

Bruce E. Ebbeson is a Senior Lead Structural Engineer with Stone & Webster, Inc., a Shaw Group Company ("S&W"), in Cherry Hill, New Jersey. He has approximately thirty years of experience as a Civil/Structural Engineer, specializing in the structural design and analysis, including seismic analysis, of nuclear facilities. Mr. Ebbeson is currently the supervisor of the structural division for S&W's Cherry Hill office and serves as structural engineering consultant on various projects performed by S&W in its Cherry Hill, Boston, Denver and Taiwan offices. Mr. Ebbeson has been the Principal Structural Engineer on many nuclear facility projects. Among other activities, Mr. Ebbeson has performed and supervised the performance of original designs and design modifications for those nuclear facility projects, as well as safety evaluations to meet licensing requirements. Additionally, Mr. Ebbeson has also performed independent design reviews of nuclear facilities at various stages of their licensing and operation.

II. TESTIMONY

A. SCOPE

Mr. Ebbeson will describe the structural design of the Canister Transfer Building ("CTB") at the PFSF and the ability of the building to withstand seismic loadings. Mr. Ebbeson will also will address the allegations raised by the State in Section D.2 of Unified Contention Utah L/QQ concerning PFS's seismic design of the CTB and its foundation. In this testimony, Mr. Ebbeson will respond to the allegations raised by the State in Sections D.2.a(i) (with respect to the potential lack of rigidity of the basemat of the building and its effect on the underestimation of the dynamic loading on the foundation); D.2.a(ii) (with respect to the potential lack of rigidity of the basemat of the building and its effect on the overestimation of foundation damping); D.2.b(i) (with respect to the potential effect of the soil cement on the soil impedance pa-

rameters); D.2.b(ii) (with respect to the potential interaction between the foundation mat and the surrounding soil cement); and D.2.d (with respect to the potential effect of non-vertically propagating waves on the rocking and torsional motion of the building and its foundations).

B. SEISMIC DESIGN OF THE CTB

Mr. Ebbeson will describe in detail the conservatisms that have been built into the structural design of the CTB and the ability of the CTB and the important-to-safety structures, systems and components ("SSCs") it contains to survive not only the ground motions from the 2,000 year return period earthquake but the motions produced by far more severe earthquakes.

C. RIGIDITY OF THE CTB BASE MAT

Mr. Ebbeson will discuss why it is correct and in accordance with industry codes and standards to treat the base mat of the CTB as a rigid body, and will explain that the assumption of rigidity will have no impact on the dynamic loadings on the building or on foundation damping.

D. EFFECT OF SOIL CEMENT ON IMPEDANCE PARAMETERS

Mr. Ebbeson will show that is consistent with the guidance in industry standards to disregard the effect of soil around foundations on the soil impedance function, and that in any event the soil cement around the CTB will have little or no effect on the soil impedance.

E. KINEMATIC INTERACTION BETWEEN THE CTB AND THE SOIL CEMENT AROUND THE BUILDING

Mr. Ebbeson will explain that the presence of soil cement was included in the input to the CTB's seismic analyses.

F. NON-VERTICALLY PROPAGATING WAVES

Mr. Ebbeson will demonstrate that the guidance in industry standards allows assuming that incoming seismic waves are vertically-propagating as long as a mass eccentricity factor of 5% is incorporated into the actual design, which PFS is doing.

April 1, 2002

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

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

TESTIMONY OF BRUCE E. EBBESON ON SECTION D OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESS BACKGROUND

Q1. Please state your full name.

A1. Bruce E. Ebbeson.

Q2. By whom are you employed and what is your position?

A2. I am a Senior Lead Structural Engineer with Stone & Webster, Inc., a Shaw Group Company ("S&W"), in Cherry Hill, New Jersey.

Q3. Please summarize your educational and professional qualifications.

A3. My professional and educational experience is described in the *curriculum vitae* attached to this testimony. Briefly summarized, I have approximately thirty years of experience as a Civil/Structural Engineer, specializing in the structural design and analysis, including seismic analysis, of nuclear facilities. I am currently 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. My experience has included assignments as Principal Structural Engineer on many nuclear facility projects. I

have, among other activities, performed and supervised the performance of original designs and design modifications for those projects, as well as safety evaluations to meet licensing requirements. I have also performed independent design reviews of nuclear facilities at various stages of their licensing and operation.

Q4. What is the basis of your familiarity with the Private Fuel Storage Facility?

A4. S&W is the Architect/Engineer for the Private Fuel Storage Facility ("PFSF") under contract with Private Fuel Storage, L.L.C. ("PFS" or "Applicant"). As such, it coordinates the facility design activities, including the studies needed to characterize the PFSF site and establish its suitability. I have been involved in the design of the PFSF since June 1998. My duties include planning and supervising the preparation of calculations and drawings for the facility and responding to questions posed by the U. S. Nuclear Regulatory Commission ("NRC"). In particular, I am responsible for the seismic analysis and structural design of the Canister Transfer Building ("CTB") for the PFSF.

Q5. What is the purpose of your testimony?

A5. One of the purposes of my testimony is to describe the structural design of the CTB and the ability of the building to withstand the seismic loadings imparted by the 2,000-year return period earthquake and other, more severe seismic events. My testimony demonstrates that there are significant margins beyond the design basis requirements in the designs of the CTB and the important-to-safety structures, systems and components ("SSCs") it contains that will enable them to survive earthquake ground motions much greater than those of the 2000-year design basis earthquake. My testimony will also respond to certain allegations raised by the State of Utah in Part D of Unified Contention Utah L/QQ with respect to the seismic analysis and design of the CTB and its foundation.

II. FUNCTIONS AND CONSERVATIVE DESIGN FEATURES OF THE CANISTER TRANSFER BUILDING AT THE PFSF

Q6. What are the design functions of the CTB?

A6. As discussed in Section 4.7.1 of the PFSF Safety Analysis Report ("SAR"), the CTB provides physical protection and shielding for the canisters containing spent fuel during their transfer from the shipping casks in which they are brought to the site to the storage casks used to store them at the PFSF. The CTB is a reinforced concrete structure with thick walls providing tornado-generated missile protection and radiation shielding.

The main function of the CTB is to facilitate the safe performance of canister transfer operations at the PFSF. Specific CTB functions include:

- Load or unload spent fuel shipping casks from railcars or heavy-haul tractor/trailers.
- Provide weather and tornado protection for performing the canister transfer operations.
- Provide the support structure for the single failure-proof cranes required for the transfer operations.
- Provide radiological shielding during the transfer operation.
- Store potential low-level radioactive waste from health physics surveys.
- Provide storage and laydown space for transfer and shipping equipment.
- Provide a staging area for storage casks.

The important-to-safety SSCs in the CTB include a 200 ton overhead bridge crane, a 150 ton semi-gantry crane, seismic support struts, the spent fuel canisters, shipping and storage casks, and transfer casks used during the canister transfer operation.

Q7. What are the main NRC regulatory and industry guidance documents used in the seismic design of the CTB?

A7. PFS follows the criteria specified by the NRC in the Standard Review Plan ("SRP") for independent spent fuel storage installations ("ISFSIs"), NUREG-1567, for the seismic design of structures such as the CTB. In addition, the criteria used for the seismic design of the CTB are those used to meet the safe shutdown earthquake loads in accordance with the NRC Standard Review Plan for nuclear power plants, NUREG-0800, to the extent those criteria are pertinent to ISFSIs such as the PFSF. Both NUREG-1567 and NUREG-0800 provide load combinations and acceptance criteria which, for the loads applicable to the PFSF, are very similar, and provide similar degrees of conservatism.

The seismic analysis and design of the CTB are performed in accordance with the standards set forth in nuclear industry standard ASCE 4-86 (relevant sections of which are included as PFS Exhibit XX), an accepted standard widely used and accepted in the seismic design of nuclear power plants and ISFSIs, which provides comparable levels of conservatism to those in the SRPs.[1]

The concrete portions of the building are designed for the appropriate load combinations, as described in Section 3.2.11.4.1 of the SAR. The strength capacity of a concrete cross-section under the seismic load combinations was determined using the guidance in the ACI 349 Code. Use of this standard is called for under SRP guidelines for nuclear facilities, including both nuclear power plants and ISFSIs.

For structural steel portions (primarily roof beams and girders), the allowable stresses are computed using the applicable load combinations for normal and shear stresses, as described in Section 3.2.11.4.1 of the SAR. The allowable steel stresses are determined following the guidance in the AISC N690 code, another standard code used in nuclear power plant design. The N-690 code is more stringent than the AISC code "Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design," which is endorsed by both NUREG-0800 and NUREG-1567.

To the extent pertinent for ISFSIs, the load combinations and acceptance criteria for the CTB under seismic loadings are those specified in NUREG-0800 for the safe shutdown earthquake loadings for nuclear power plants.

Q8. Would you please describe the main features of the design of the CTB and its foundation that provide protection against the forces resulting from an earthquake?

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1 In his deposition, State witness Dr. Farhang Ostadan acknowledged that ASCE 4-86 and its subsequent revision ASCE 4-98 are very important standards used in the seismic design of nuclear facilities. See Ostadan Tr. at 72-73.

A8. The CTB roof consists of an eight-inch thick reinforced concrete slab supported on structural steel beams spanning in the N-S direction, which are in turn supported by plate girders spanning in the E-W direction. There are studs on the beams and girders to prevent the roof slab from uplifting during a design basis tornado. The beams and girders are designed as simply supported members, with no consideration of composite behavior. The roof has been designed for a vertical acceleration of 1.84 g at the roof center.

The CTB is supported by a heavily reinforced concrete foundation mat. The foundation mat is 240 ft. in the E-W direction, 279.5 ft. in the N-S direction, and 5 ft. thick. A reinforced concrete key, 1.5 ft. deep by 6.5 ft. wide, will be constructed around the perimeter of the foundation mat. The purpose of this key is to ensure that the full shear strength of the clayey soils beneath the foundation is available to resist sliding of the structure due to the loads from the design basis ground motion.

The CTB foundation design calls for soil cement to be placed around the base mat to help resist earthquake sliding forces. Soil cement will thus surround the foundation mat and will extend outward from the mat to a distance equal to the associated mat dimension; i.e., approximately 240 ft. out from the mat in the E-W direction and approximately 280 ft out in the N-S direction. Existing soils will be excavated to a depth of approximately 5 ft. 8 in. below grade, mixed with cement, and placed and compacted around the foundation mat. The soil cement placed around the CTB foundation mat will be 5 ft. thick and have a minimum unconfined compressive strength of 250 psi. The top 8 inches will be filled with compacted aggregate, similar to that used in the pad emplacement area.

Q9. Are there conservatisms embodied in the codes and standards you referenced and in the manner you applied them in developing the structural design of the CTB?

A9. Yes.

Q10. What are some of the main conservatisms?

A10. A major conservatism that is incorporated in the applicable codes and standards is that stresses resulting from the design basis earthquake are required to be limited to levels below the specified yield point of the materials. It is well known that concrete and steel structures have substantial deformation capacity above and beyond the point of first yielding. Codes used to design conventional buildings, such as the Uniform Building Code, recognize this fact and increase allowable seismic loads for ductile structures. The CTB and the SSCs of interest in it are generally ductile and have significant deformation capacity beyond yield.

Q11. Are there additional design elements that provide further conservatisms?

A11. Yes, the criteria recommended by the NRC in the SRP for ISFSIs (NUREG-1567) for use in the seismic design of structures such as the CTB provide large additional margins against building failure in an earthquake. The CTB is designed to resist lateral force through a series of reinforced concrete shear walls. This design is highly effective in resisting earthquake forces. The conservatisms built into the design of the CTB can be illustrated by comparing the design of the CTB to a structure that would fulfill similar functions designed under conventional structure codes. The use of conventional building codes would result in a structure designed for much lower seismic forces.

Q12. How does the design of the CTB compare to a similar structure that has been built under conventional building codes?

A12. If one were to design a building of the same general design as the CTB in accordance with the Uniform Building Code (1994) ("1994 UBC") (which was the version of the UBC in effect at the time the license application for the PFSF was filed) under the most conservative assumptions possible, i.e., if it were located in the most severe earthquake area in the continental US (Seismic Zone 4, $A_v = 0.40$); the location had the worst soil conditions (Soil Profile Type S_4); and the facility had the highest Seismic Importance Factor ($I = 1.25$, hazardous facilities), the combination of these conditions would require that the building be designed for a base shear force of 0.23 times the building weight above the base. By contrast, the PFSF CTB has been designed for a base shear force of

approximately 1.17 times the weight above the base. In other words, the CTB has been designed for seismic forces 5 times those for which a conventional structure would be designed, assuming that structure was subject to the most severe seismic design requirements possible under the 1994 UBC.

Q13. How would the design of the CTB compare to a similar structure designed under conventional building codes for Utah?

A13. Since Utah is located in Seismic Zone 3 (and 2), the CTB at the PFSF has been designed for seismic forces almost 7 times those for which a conventional structure located in Utah would have been designed for under the 1994 UBC and previous codes.

Q14. Are conservatisms incorporated into the designs of other components in the CTB?

A14. Yes. The applicable seismic load combinations for the cranes within the CTB are described in Section 3.2.11.5 of the SAR. Allowable stresses on the cranes are conservatively limited to the allowable levels of ASME NOG-1.

These cranes are designed to the same design codes as a crane that would be installed at a nuclear power plant and are, therefore, the same, to the extent applicable, as those specified in the NUREG-0800, the SRP for nuclear power plants.

Q15. Is there reserve capacity in the CTB and the structures it contains that would allow them to resist seismic loadings beyond those from the design basis earthquake?

A15. Yes. Reserve capacity exists due to many factors, including, but not limited to: a redistribution of stresses from highly stressed areas of the structure to adjacent areas which occurs after yielding; the fact that the actual material yield strength (for concrete, the compressive strength) exceeds the nominal yield strength values; and the fact that the materials' ultimate strength is significantly greater than its yield strength. Additionally, the seismic loads are of short duration and reverse direction several times each second. Thus, even where some yielding occurs, the load will likely reverse direction before significant distortion can occur and the stresses will return to the elastic range.

Q16. Are there also conservatisms incorporated in the seismic design of the foundations of the CTB?

A16. Yes. A number of conservatisms are incorporated into the design of the CTB foundations. Those conservatisms are evidenced in the building's safety factors against dynamic sliding, overturning, and bearing capacity failures, as described in Calculation 05996.02-G(B)-13, Rev. 6, *Stability Analyses of Canister Transfer Building* (July 26, 2001) ("CTB Stability Calc. Rev. 6") and in the testimony of Paul J. Trudeau on Part D of Unified Contention L/QQ, filed simultaneously herewith.

Because of these conservatisms, there is no concern about potential overturning of the CTB under beyond-design basis earthquake loadings. The CTB is a very stable structure, exhibiting a factor of safety of 1.95 under design basis (2000-year return period earthquake) loadings. Even if the factor of safety against overturning were reduced to less than 1.0 in a beyond-design basis earthquake, the building would not overturn. There could be some lift-off, but the building would tend to return to contact with reversals of the ground acceleration, thus there would be no safety consequences from the lift-off.

This conclusion can be demonstrated by comparing the CTB to the casks on the storage pads. The casks have a tendency to tip (i.e. lift off) during the 2000-year earthquake, but do not overturn. Because of its more stable configuration (240 feet wide and less than 100 feet high, with much of the mass concentrated at the bottom) the CTB is inherently more stable than the casks, and exhibits no such tendency to tip during the 2000-year earthquake.

Holtec has performed analyses of the pads and casks to evaluate their response to a beyond-design basis, 10,000-year return period earthquake. The analyses indicate that the casks will not overturn in such an earthquake. Since the CTB is more stable than the casks, it can be safely predicted that the CTB will not overturn during a 10,000-year earthquake.

Q17. How would the seismic loadings on the CTB change from the 2,000-year return period design basis earthquake to a 10,000-year return period earthquake?

A17. My understanding, based on accelerations corresponding to a 10,000-year return period earthquake provided by PFS's consultant, Geomatrix Consultants, Inc., is that the free-field ground motion due to the 10,000-year return period earthquake has a peak acceleration estimated to be 70-90% greater than that due to the 2,000-year return period earthquake (depending on the direction of motion).

Q18. Would such an increase in peak earthquake acceleration result in a corresponding increase in building accelerations for the CTB?

A18. No. The building accelerations will not necessarily increase in the same proportion as the free-field ground motion. This is due to several factors. First, the soil strains will be higher under higher accelerations. This will result in a reduction in soil shear modulus and increased soil damping. The seismic analysis of the CTB (Calculation 05996.02-SC-5) clearly shows that the building accelerations decrease considerably as the soil stiffness decreases (based on examination of results from the best estimate, lower bound and upper bound soil cases). Both reduced soil stiffness and increased damping will reduce building accelerations.

Furthermore, at the high ground acceleration levels produced by a 10,000-year return period earthquake, the CTB will exhibit non-linear behavior, with the building sliding on and separating from the soil for brief periods of time. Since the vertical acceleration will at times exceed 1.0 g, it is obvious that there will be times that the building will not be in contact with the soil. These non-linear effects will significantly reduce the building accelerations, similar to the manner in which a base-isolated structure behaves under seismic loadings, resulting in no adverse safety consequences.

Q19. What effect do these conservatisms in the design of the CTB have on its ability to withstand a potential building collapse in the event of a beyond-design-basis earthquake?

A19. While the CTB is designed – in accordance with NRC guidance – to withstand the forces resulting from a 2,000-year return period earthquake, due to the structural

factors and the mechanics of a beyond-design-basis earthquake, the CTB has a large additional reserve capacity. Given this reserve capacity, the CTB would be expected to survive a much more severe earthquake than the 2,000-year return period earthquake.

A primary concern with respect to building collapse is the potential for collapse of the CTB roof during canister transfer operations. However, the CTB roof has the capacity to withstand accelerations well in excess of those produced by the design basis, 2,000-year return period earthquake for the following reasons:

- The bending moment capacity due to downward loads of a typical girder is 9598 ft-kips, based on N-690 code allowable stresses. The maximum calculated moment is only 6861 ft-kips (71% of capacity).
- The roof bending moment capacity of 9598 ft-kips is based on the N-690 code allowable stresses. The ultimate moment capacity based on the plastic section modulus and minimum material tensile strength is approximately 14,800 ft-kips (54% higher).
- While the studs on the beams and girders have not been designed to provide full composite action, the existing design provides some increase in strength. Fully composite behavior would allow for a vertical acceleration of up to 4 g. I estimate that the existing design can resist a vertical acceleration of at least 3 g.
- The girders are assumed to be simply supported at their ends, where they attach to the N-S walls. Since the girders are connected to the roof slab, and the roof slab is integral with the walls, rotation of the girder will be restrained at the walls, reducing the bending moment at midspan. I estimate that this arrangement increases the load carrying capacity by about 30%.

Due to these factors, the CTB roof should be capable of withstanding accelerations significantly greater than those produced by a 2,000-year return period earthquake without failing.

Q20. Would other SSCs in the CTB also be capable of withstanding a beyond-design-basis earthquake?

A20. Yes. The structural members of the cranes that handle the spent fuel casks and canisters have the same type of reserve capacity as the CTB's structural steel

elements. I have learned from a consultant to Stone & Webster with more than twenty years of experience in the design of cranes, including those for nuclear power plants, that the CTB cranes' mechanical components have additional design margins to accommodate increases in seismic loading. The ultimate strengths of mechanical component materials subject to tension and compressive loads are designed such that the ultimate strength of the material is five (5) times that required to support the lifted load. Additionally, if failure of a mechanical component could cause the load to drop, the design of the component is then increased such that the ultimate strength of the material is (10) times that required to support the lifted load. For example, the hoisting cables, as addressed in ASME NOG-1, have a "maximum allowable load" under the design basis earthquake that is less than 40% of the rope's breaking strength. Thus, the cranes to be used in the PFSF CTB would be able to withstand the forces resulting for an earthquake with a return period significantly greater than the 2,000-year return period of the design basis earthquake.

Similar margins exist in the design of the seismic support struts (restraints) for the casks used during canister transfer operations. Those restraints are designed to ASME NF criteria and, therefore, meet the same standards to which comparable nuclear power plant safety-related components are designed. Thus, under code acceptance criteria, the nominal capacity of the seismic struts is 400 kips. The maximum strut load due to the 2000-year return period earthquake is 395 kips. While this would seem to push the capacity of the struts, there is additional conservatism built into the design. Based on an evaluation of the critical components of the seismic strut assembly (tie rods, tie rod welds, strut pins, strut pipe strut pipe end welds, and bracket welds), the ultimate strut load capacity is at least 571 kips. Thus, the seismic struts used in the PFSF CTB will be able to withstand the forces resulting from an earthquake with a return period significantly greater than the 2,000 years of the design basis earthquake.

Q21. What is your conclusion about the survivability of the CTB and the important-to-safety SSCs it contains in the event of a beyond-design-basis seismic event?

A21. The CTB and all important-to-safety SSCs it contains possess far greater seismic loading capacities than those for which they were nominally designed. In addition to the margins that can be explicitly calculated (as discussed above), there are also margins which are known to exist but which are not easily quantifiable. For example, as discussed above, steel structures have reserve capacity above the onset of yielding due to, among other things, the redistribution of stresses -- from highly stressed areas to adjacent areas -- which occurs after yielding. This combination of quantifiable and non-quantifiable margins provides a great degree of assurance that the structures will be able to perform well beyond their design limits. Consequently, there is no doubt that the CTB and the important-to-safety SSCs it houses can withstand acceleration levels well in excess of those associated with the design basis earthquake and have a high likelihood of surviving without loss of safety function in an earthquake with a return period significantly greater than the 2000 years of the design basis earthquake.

III. RESPONSE TO STATE CLAIMS IN SECTION D RELATING TO THE DESIGN OF THE CTB AND ITS FOUNDATION

Q22. In Paragraph D.2 of Unified Contention Utah L/QQ, the State alleges that several deficiencies exist in the seismic design of the CTB and its foundation. Are you familiar with those allegations?

A22. Yes.

Q23. What is your general response to the State's allegations?

A23. The claims raised by the State are hypothetical and are contrary to the guidance in applicable industry standards. Moreover, any potential adverse effect on the seismic performance of the CTB resulting from the factors raised by the State is within the limits of accuracy of the analysis. Any such adverse effect is also made up by other features of the analysis and the design, and is readily absorbed by the factors of safety that exist in the design codes and standards.

Q24. In Subparagraph D.2.a(i) of Unified Contention Utah L/QQ, the State asserts that PFS's calculations incorrectly assume that the CTB mat foundation will behave rigidly during

the DBE, an assumption that is alleged to lead to a significant underestimation of the dynamic loading to the mat foundation. How do you respond to these assertions?

A24. Assuming that the CTB mat is rigid is appropriate. Because of its five foot thickness and the stiffening provided by the shear walls connected to the mat, the mat can be assumed to behave rigidly in an earthquake. This is consistent with Section 3.3.1.6 of the industry code that governs the structural design of the CTB, ASCE 4-86, which states: "The effect of mat flexibility for mat foundations and the effect of wall flexibility for embedded walls need not be considered in the SSI analysis." See PFS Exh. XX at 26.

I have reviewed the CTB basemat displacement results of Stone & Webster Calculation Nos. 05996.02-SC-6, "Finite Element Analysis of Canister Transfer Building", Revision 1, which is in the final stages of completion. That calculation shows that, for the loading combination with the full vertical earthquake acting downward (40% of each of the horizontal earthquakes acting, in addition to dead and live loads), the maximum variation of displacement along the centerline of the building in the N-S direction is .164 inches over the length of 279.5 ft. (less than 0.005%) deflection. The maximum variation of displacement in the E-W direction is. 334 inches over the length of 240 ft. (about 0.01% deflection). These small differential displacements further demonstrate the appropriateness of treating the CTB base mat as rigid in the PFS seismic analyses. See PFS Exh. YY.

Q25. State witness Dr. Ostadan testified as follows with respect to the potential flexibility of the CTB mat in his deposition taken on March 8, 2002 at p. 136: "As I indicated before, I would not have raised this issue if we had a good margin under the sliding conditions. I think Holtec or Stone & Webster is on the record that the factor of safety for sliding would be less than one if we do not include soil cement. And then they rely on the soil cement, that we have a number of issues with, to provide the passive resistance. So that, to me, is a slim margin that we have, be it safe or unsafe. Now you talk about the mat being rigid or flexible enough to increase the seismic loads. If it was flexible, it becomes important. Even though the general guidance is not to worry about it. I think it should be viewed in light of the overall design and the margin." How do you respond to Dr. Ostadan's position?

A25. Dr. Ostadan has produced no evidence to suggest that the CTB base mat does not behave rigidly, and he has in fact acknowledged that he knows of no such evidence (Ostadan Dep. Tr. at 137). Industry practice, as reflected in the above referenced ASCE standard, endorses treatment of mats such as this as rigid and the results of the SC-6 calculation discussed above demonstrate that the assumption of rigidity is appropriate. Also, the allowable factor of safety against sliding to which Dr. Ostadan refers as slim is actually 1.1, which in itself represents a 10% design margin, since the onset of sliding will not occur until the factor of safety goes below 1.0. (This factor of safety is set in accordance with the guidance in NUREG-0800, the SRP for nuclear power plants.) Thus, the potential effect of mat flexibility is accommodated by the factor of safety applied in the seismic stability calculations. Ultimately, of course, whether the CTB slides is inconsequential, since the building is free-standing and there are no safety-related components connected to it which could be affected by the sliding of the building.

Q26. In Subparagraph D.2.a(ii) of Unified Contention Utah L/QQ, the State asserts that PFS's incorrect assumption that the CTB mat foundation will behave rigidly during the DBE leads to an assumption that is alleged to lead to an overestimation of foundation damping. What is your response?

A26. As I indicated, the assumption of rigid mat behavior is appropriate and consistent with industry practice. However, even if such an assumption led to some overestimation of foundation damping, there is sufficient margin in other areas of the CTB foundation design to compensate for it.

I would also note that if the frequency-dependent properties of a structure change due to a change in the structure's flexibility, both the stiffness and the damping components of the impedance change. It is not appropriate to look at one aspect (damping) of the impedance without the considering the other (stiffness). The effects of a lessening of the foundation damping would tend to be offset by the effects of the simultaneous reduction that would occur in the structure's stiffness. PFS Calculation 05996.02-SC-5 Rev. 2, "Seismic Analysis of the Canister

Transfer Building," shows that the accelerations experienced by the CTB during a seismic event tend to decrease as the soil stiffness is reduced.

Q27. Dr. Ostadan goes on to testify at p. 137 of his deposition that, if the CTB mat is flexible instead of rigid there will be less radiation damping of the structure, which will result in higher vertical loads. Do you agree?

A27. The significance of mat flexibility hinges on the relative stiffness between the mat and the surrounding soil. Analyses in the literature[2] show that the frequency dependent values of stiffness and damping of the structure are significantly different from the rigid case values only if the ratio of mat-to-soil stiffness is very low, which is definitely not the case for the CTB mat. In particular, for vertical radiation damping, which is the parameter of interest, there is little difference between the rigid case and one in which limited mat flexibility is present. Therefore, even if one assumed that the CTB mat was somewhat flexible, there would be no discernible increase in the vertical loads on the structure.

In addition, a study I performed for the storage cask pads (referenced in Dr. Tseng's testimony) demonstrates that the effects of pad flexibility on the impedance functions are not significant. See PFS Exh. MM. Because of the greater thickness (five feet) of the CTB mat and the stiffening effect of the interior and exterior shear walls, I would expect the effect of potential flexibility on impedance to be of even less significance for the CTB base mat than it is for the pads.

Q28. In Subparagraph D.2.b(i) of Unified Contention Utah L/QQ, the State claims that the PFS calculations ignore the presence of a much stiffer, cement-treated soil cap around the CTB but that the presence of this soil cap impacts the soil impedance parameters. First of all, what are the soil impedance parameters?

A28. They are the frequency-dependent spring and damping parameters that are used to characterize the soil in soil-structure interaction analyses.

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2 M. Iguchi and J. E. Luco, " Dynamic Response of Flexible Rectangular Foundations on an Elastic Half-space," Earthquake Engineering & Structural Dynamics, Vol. 9, No. 3, May - June 1981, Figs. 4 and 5.

Q29. Would the presence of a soil cement layer around the CTB affect the soil impedance parameters?

A29. It might, but any impact would be minimal and can be disregarded in accordance with standard industry practice.

Q30. To what industry practice do you refer?

A30. The soil cement around the CTB is no different than soil backfill, except for being somewhat stiffer. Section 3.3.4.2.4 of ASCE 4-86 states: "For shallow embedments (depth-to-equivalent-radius ratio less than 0.3), the effect of embedment may be neglected in obtaining the impedance functions, provided the soil profile and properties below the basemat elevation are used for the impedance calculations." See PFS Exh. XX at 29. In talking about embedments, the standard is referring to the portion of the soil that surrounds the foundations. The standard is saying that the effect of the soil layer around a foundation can be disregarded in computing the soil impedance for soil structure interaction analyses, if certain conditions are met.

We have complied with those conditions. The depth-to-equivalent-radius ratio for the CTB is less than 0.04, which is much less than 0.3; and actual soil properties below the basemat elevation were used in the impedance calculations. Therefore, the effect of the soil cement around the CTB can be disregarded.

Q31. Dr. Ostadan testified at his deposition (Tr. at 225-31) that the fact that soil cement is present around the CTB foundation makes a difference in the values of soil impedance parameters such that the code guidance does not apply. How do you respond?

A31. I do not believe the distinction Dr. Ostadan is trying to draw is a sound one. The guidance in the ASCE standard would allow us to ignore embedments even if we had 40 feet of compacted backfill around the building. Five feet of soil cement should have less impact on the impedance calculations than 40 feet of backfill. For that reason, it is appropriate to ignore the contribution of the soil cement layer around the CTB foundations in the soil impedance calculations.

Considering the issue from the physical standpoint, the main interface between the CTB and the subgrade occurs at the base of the foundation mat. Energy radiates downward and outward into the soil at this interface. The presence of a soil-cement cap around the CTB has no effect on this energy-dissipation mechanism, which is directed downward and not in the horizontal direction.

Looking in particular at the vertical and rocking components of motion, I cannot envision how the presence of soil cement would have any impact at all on those components of motion. As State witnesses have pointed out, the settlement of the CTB relative to the soil-cement will cause vertical cracks at the mat-soil cement interface. After that happens, the soil cement will not be able to influence vertical and rocking movements of the mat, i.e., the mat will be able to move up and down relative to the soil-cement.

Finally, I would note that the SSI analysis is done with three sets of impedance functions to cover possible variations in soil properties, and the most conservative (least favorable) results are used for design of the CTB. This enveloping technique accounts for any minor variations in soil impedance, caused by soil cement or other conditions.

Q32. In Subparagraph D.2.b(ii) of Unified Contention Utah L/QQ, the State claims that the PFS calculations ignore the presence of a much stiffer, cement-treated soil cap around the CTB but that the presence of this soil cap impacts the kinematic motion of the foundation of the CTB. In the State's response to Interrogatory No. 14 in the Applicant's Eighth Set of Discovery Requests, the State explains this concern as follows: "The soil-cement and the concrete mat foundation will have significantly different stiffnesses and such contrasts in stiffness (or impedance parameters) will cause kinematic interaction between the soil-cement and the CTB mat foundation. This interaction may lead to overstressing and cracking of the soil-cement placed immediately adjacent to the CTB and renders it ineffective in performing its intended function used for CTB analysis." Is this a valid concern?

A32. No. The input to the CTB seismic analysis includes the free-field motion and the strain-dependent soil properties, both of which were developed by Geomatrix Consultants, Inc. ("Geomatrix"), and I understand Geomatrix included the presence of soil cement in developing these inputs to our analysis. In our

dynamic analysis of the CTB, we use the free-field motion as an input located at the top of the profile. However, the applicable motion to use would be the motion at the base of the building foundation, which is at the top of the clay layer. Based on my review of the results of the Geomatrix analyses that developed the strain-dependent soil properties, the seismic motion at the base of the mat (the top of the underlying clay layer) is slightly lower than at the surface; hence, the input provided by Geomatrix is conservative, and so is our seismic analysis.

The issue of the potential cracking of the soil cement and its effect on stability analyses for the CTB is discussed in the testimony of Paul Trudeau on Section D of Unified Contention L/QQ, filed simultaneously with my testimony.

Q33. In subparagraph D.2.d of Unified Contention Utah L/QQ, the State claims that 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. Should the effect of non-vertically propagating waves have been taken into account in the CTB seismic calculations?

A33. No. This is essentially the same claim raised with respect to the seismic design of the cask storage pads in subparagraph D.1.a of Unified Contention Utah L/QQ. That claim is refuted in the testimony of Robert Youngs and Wen-Shou Tseng on Unified Contention L/QQ, which is being filed simultaneously herewith. The evaluation performed by Drs. Youngs and Tseng led them to conclude that the angles at which seismic waves would impinge the PFSF site are, for all practical purposes, vertical.

In addition, the Commentary to Section 3.3.1.2(a) of ASCE 4-86 Code allows the seismic analyses of structures such as the CTB to assume incoming seismic waves to be vertically propagating waves provided a mass eccentricity factor of 5% is incorporated into the actual design of the structures to address the effects of inclined and incoherent waves. See PFS Exh. XX at 66. S&W is following this recommendation in the design of the CTB, so there is no reason why it would need to account in the seismic analyses of the building for non-vertical propagation of seismic waves.

Q34. Do you know whether the State witnesses agree with your position?

A34. Yes. State witness Dr. Ostadan testified in his March 8, 2002 deposition at p. 78-79 that if an accidental mass eccentricity factor was included in the design of the CTB, there was no need to consider the potential for non-vertical propagation of seismic waves.

Q35. Does that conclude your testimony?

A35. Yes, it does.

Experience Summary

Mr. Ebbeson has 29 years of experience in the engineering industry. Currently, he is the supervisor of the structural division for Stone & Webster's Cherry Hill office. In addition to these duties, he is presently involved in a number of projects, including design of a nuclear spent fuel storage facilities in Iowa and Utah and seismic analysis of a nuclear power plant in Taiwan. He serves as a structural engineering consultant on various projects performed in Stone and Webster's Cherry Hill, Boston, Denver and Taiwan offices. Previously, his experience has included assignments on many nuclear power plant projects as a Principal Structural Engineer in a supervisory capacity. He has designed plant modifications and performed safety evaluations to meet licensing requirements. He also has coordinated the implementation of modifications with construction groups and has performed independent design reviews of nuclear power plants at various stages of licensing/operation.

Upon joining Stone & Webster Engineering Corporation in 1973, he was first assigned as a Career Development Engineer in the Structural Division where he was assigned to the Structural Mechanics Section. He was later assigned to the Engineering Mechanics Division as a support engineer in the Structural Mechanics Staff Group. He was reassigned to the Cherry Hill Office in July 1979, to assume the responsibilities as Principal Structural Mechanics Engineer on the River Bend Project. He has worked on various projects where his duties have included conceptual arrangement, analysis, and design of structural components of nuclear power plants.

Prior to joining Stone & Webster Engineering Corporation, Mr. Ebbeson was a Structural Design Engineer with the Philadelphia Water Department, Philadelphia, Pennsylvania.

Education

M.S., Civil Engineering – Tufts University - 1973
B.S., Civil Engineering - Tufts University - 1970

Training

Various courses in Engineering Management - Drexel University
Various Stone & Webster Management Training Classes

Licenses, Registrations, and Certifications

Professional Engineer - Massachusetts - 1977
Professional Engineer - Louisiana - 1981
Professional Engineer - New Jersey - 1983

Professional Affiliations

American Society of Civil Engineers - Member

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Experience History

STONE & WEBSTER ENGINEERING CORPORATION, CHERRY HILL, NEW JERSEY - 1979 TO PRESENT

Structural Division Supervisor (Apr 1999 to Present)

Presently, Mr. Ebbeson is responsible for all Civil/Structural activities in the Cherry Hill Office, including hiring, personnel evaluations, project staffing and technical direction. Additionally, he is actively involved as a consultant on a number of projects, including the Private Fuel Storage Skull Valley project, the Duane Arnold ISFSI project and Taiwan Power's Lungmen project.

Department of Energy MOX Project (July 2000 to Present)

Mr. Ebbeson serves as a member of the Technical Oversight Committee that was responsible for the review of the seismic analysis and structural design of a mixed oxide fuel production facility to be built on the Savannah River site.

AT&T Point of Presence (POP) Building, 700A Street, Wilmington, DE (Sept 1999 to Jan 2000)

Mr. Ebbeson provided civil/structural consulting support for the development of conceptual designs for the 24,000 sq. ft. network building. He was involved in the review of the Geotechnical report and in the preparation of a report performed to evaluate the risk to the facility from floods.

AT&T (Oct 1998 to Nov 1999)

Mr. Ebbeson was assigned to a team responsible for performing reliability assessments of AT&T facilities including those in Durham NC, Dublin O, Chicago, Boston, Staten Island, Miami, Florham Park and Jersey City. He was responsible for performing the civil/structural portion of the assessments, including preparation of reports.

Private Fuel Storage Facility (June 1998 to Nov 2001)

Mr. Ebbeson was responsible for the seismic analysis and structural design of the Canister Transfer Building for a proposed facility that will store spent nuclear fuel. His duties included planning and supervising the preparation of calculations and drawings for the facility, and responding to questions posed by the Nuclear Regulatory Commission.

Public Service Electric & Gas Company (Feb 1990 to Oct 1998)

As Lead Civil/Structural Task Manager, Mr. Ebbeson was responsible for coordinating the civil/structural activities on all tasks for the Hope Creek and Salem Nuclear Generating Stations. He has developed design criteria and technical standards for the design of structures and structural components. He has performed and directed structural activities for a number of major design changes, including feedwater heater replacement, control room architectural renovation, auxiliary building ventilation upgrades, containment fan coil unit upgrades, addition of tornado missile barriers and Salem Unit 3 leakage/spill containment. These activities include design of HVAC, electrical raceway and piping systems, seismic qualification of safety-related equipment, design of equipment

supports, design of new structures, evaluation of existing structures for increased loadings, and design of rigging systems. When necessary, finite element and structural dynamic analyses were performed. He also served as Task Manager, responsible for developing schedules and budgets, managing the task execution, and interfacing with the client's Project Manager, for a number of projects.

Browns Ferry Nuclear Plant (Sept 1989 to Dec 1989)
Tennessee Valley Authority

Assigned to the site as lead Structural Engineer, Mr. Ebbeson was responsible for the update and verification of the Final Safety Analysis Report (FSAR).

Industrial Projects Group (May 1989 to Sept 1989)

As Principal Structural Engineer, Mr. Ebbeson was responsible for a variety of structural tasks, including design of steel and concrete structures for a solid waste resource recovery facility (Pasco County), design of improvements to office buildings (New Jersey Bell), and rewriting of structural specifications (Niagara Mohawk Power Corporation's Nine Mile Point Nuclear Station). Also responsible for investigation of structural adequacy at IBM's East Fishkill, New York, facility.

Limerick Generating Station - Unit 2 (June 1988 to Apr 1989)
Philadelphia Electric Company

As Lead Structural Engineer, Mr. Ebbeson was responsible for the preparation of review plans, performing technical reviews and writing a final report for submittal to the NRC as part of the integrated design and construction assessment.

Brown's Ferry Nuclear Plant (Feb 1988 to Apr 1989)
Tennessee Valley Authority

As Lead Structural Engineer, Mr. Ebbeson was responsible for directing the structural portion of the calculation review program. This program consisted of a technical review of the structural design to verify the adequacy of the existing facility. Also responsible for directing the structural design and analysis tasks required to improve the design of the existing plant.

Comanche Peak Steam Electric Station (Sept 1986 to Jan 1988)
TU Electric Company

As Assistant Lead Engineer, Mr. Ebbeson was responsible for design verification of the containment building base mat and shell, the auxiliary/electric building and the safeguards building. Responsible also for the verification of structural seismic analysis results. Duties also included preparation of estimates, development of design criteria, and writing of reports.

Beaver Valley Power Station Unit 2 - (May 1986 to June 1986)
Duquesne Light Company

As Technical Reviewer, Mr. Ebbeson was responsible for the overall review of structural work. Activities included review of licensing criteria, design basis, technical review of calculations, review of drawings and specifications, and preparation of a final report.

BWR Continuing Services Project (Mar 1986 to Aug 1987)

As Lead Structural Engineer, Mr. Ebbeson was responsible for all structural work performed by SWEC on three existing BWR nuclear projects.

Oyster Creek Nuclear Generating Station (Nov 1983 to Feb 1986)
General Public Utilities Nuclear Corporation

As Lead Structural Engineer, Mr. Ebbeson was responsible for all structural work, concerned with field modifications to the existing nuclear facility.

Structural Division Staff (June 1982 to Feb 1985)

As Principal Staff Engineer, Mr. Ebbeson was responsible for planning and supervising all structural seismic and hydrodynamic analyses for nuclear projects.

Field Assignment (March 1983 to June 1983)

Temporary assignment to Washington Public Power Supply System (WPPSS) offices in Richland, Washington. Mr. Ebbeson served as a consultant to WPPSS in the civil/structural area during final design reverification of a nuclear project.

River Bend Station - Unit 1 (July 1979 to May 1982)
Gulf States Utilities Company

As Principal Engineer, Mr. Ebbeson was responsible for the planning and supervision of the analysis and design of the reactor building concrete structures and steel containment as well as the dynamic analyses of all Category I buildings. Also responsible for preparing licensing documents, writing reports, and resolving construction problems.

STONE & WEBSTER ENGINEERING CORPORATION, BOSTON, MASSACHUSETTS - 1973 TO 1979

As Structural Engineer (Dec 1978 to July 1979), Mr. Ebbeson was responsible for analysis and design of nuclear power plant containment structures and internal structural components. Projects included Montague (miscellaneous studies), NYSE&G, and the EPRI breeder conceptual study (structural design of reactor building). Also worked on a special task force to re-analyze five nuclear plant shut down in March 1979.

As Support Engineer (Aug 1973 to Dec 1978), Mr. Ebbeson was responsible for working in the area of barrier designs for protection from tornado and accident generated missiles. Also responsible for development of computer programs, planning of a physical testing program, inspection of a tornado disaster area, and analysis and design of steel and concrete missile barriers. Also worked on analysis and design of structures on various projects. Projects included Shoreham, Philadelphia Electric (equipment drop impact problems), SWEC's Reference Nuclear Power Plant (RNPP) (conceptual design of containment internal structures and seismic analysis), and Beaver Valley - Unit 2 (seismic analysis and checking of containment internal structures design).

Oswego Steam Station - Units 5 and 6
Niagara Mohawk Power Corporation (June 1973 to Aug 1973)

As Career Development Engineer, Mr. Ebbeson was responsible for assisting Structural Engineers on a fossil fuel power plant project. Duties included helping with the preparation of specifications,

comparison of bids, and coordination of design and construction activities.

PHILADELPHIA WATER DEPARTMENT, PHILADELPHIA, PENNSYLVANIA - 1970 TO 1971

As Structural Design Engineer (June 1970 to Aug 1971), Mr. Ebbeson was responsible for design of steel and concrete structural elements, preparation of drawings, and checking of designs and drawings.