RAS 3077

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USNRC

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD '01 MAY 22 P3:11

OFFICE OF SECRETARY
RULEMAKINGS AND
ADJUDICATIONS STAFF

In the Matter of:	)	Docket No. 72-22-ISFSI
PRIVATE FUEL STORAGE, LLC (Independent Spent Fuel Storage Installation)	)	ASLBP No. 97-732-02-ISFSI
	)	May 16, 2001

STATE OF UTAH'S REQUEST FOR ADMISSION OF LATE-FILED CONTENTION UTAH QQ (Seismic Stability)

Pursuant to 10 CFR § 2.714 and the Board's April 26, 2001 Order, the State seeks admission of late-filed Contention Utah QQ, which, in general, challenges the application of Private Fuel Storage's ("PFS's") newly revised design basis ground motions to the Canister Transfer Building ("CTB"), the storage pads, and their foundations; PFS's intended use and redesign of soil cement around the CTB and under and around the storage pads; and the foundation design of the CTB, storage pads, and their underlying soils, and the stability of the storage casks, to safely withstand the newly revised design basis ground motions.

Utah QQ is supported by the Declarations of Drs. Farhang Ostadan, Steven Bartlett, and James Mitchell, attached hereto as Exhibits 1, 2, and 3, respectively. The State meets the late-filed factors and, for the reasons stated below, requests the Board to admit Utah QQ.

BACKGROUND

The State's geotechnical contention, Utah L[1], admitted into this proceeding consists

[1] Utah L asserts: "The Applicant has not demonstrated the suitability of the proposed ISFSI site because the License Application and SAR do not adequately address site and subsurface investigations necessary to determine geologic conditions, potential seismicity, ground motion, soil stability and foundation loading." LBP-98-7, 47 NRC 142, 191, 253 (1998); State of Utah's Contentions (November 23, 1997) at 80.

Template = SECY-041

SECY-02

of four bases: (1) Surface Faulting; (2) Ground Motion; (3) Characterization of Subsurface Soils: (a) Subsurface Investigation, (b) Sampling and Analysis, and (c) Physical property testing for engineering analysis; and (4) Soil stability and foundation loading. PFS filed for Summary Disposition of Utah L on December 30, 2000 and the motion is pending before the Board. There are similarities and differences between Utah QQ and Utah L--similar in that PFS has still failed to incorporate critical assumptions into its dynamic analyses that the State identified in Utah L, and different in that now ground motions have increased and PFS intends to use soil cement as a structural design element to overcome strong ground motions. The impact of Utah QQ on Utah L is addressed at the end of this document.

On April 2, 1999, PFS requested an exemption from 10 CFR § 72.102 (f)(1) to allow it to conduct a probabilistic seismic hazard analysis ("PSHA") instead of the required deterministic analysis. In the final Safety Evaluation Report, the Staff issued a favorable ruling on PFS's request to use a PSHA with a 2,000 year return period. In response, the State filed its third request to amend Utah L. The Board ruled on the admissibility of the request and certified whether a challenge to the PFS exemption request should be litigated in this proceeding. Briefing is complete and the matter is pending before the Commission.

CONTENTION QQ Seismic Stability

PFS's site specific investigations, laboratory analyses, characterization of seismic loading, and design calculations, including redesign of soil cement,[2] fail to demonstrate that a) the newly revised probabilistic seismic hazard design basis ground motions have been

[2] PFS uses the term "soil cement" but the more correct term is "cement-treated soil." *See* Mitchell Dec. ¶ 12. The use of the term "soil cement" in this filing does not imply the State accepts that PFS will, in fact, use soil cement.

correctly and consistently applied to the Canister Transfer Building ("CTB"), storage pads, and their foundations; b) PFS's general design approach, including the redesign of soil cement, for the CTB, storage pads, or storage casks can safety withstand the effects of earthquakes; and c) the foundation design of the CTB, storage pads, and the underlying soils, or the stability of the storage casks, are adequate to safely withstand the newly revised probabilistic seismic hazard design basis ground motions. 10 CFR §§ 72.102(c), (d); 72.122(b).

BASIS:

PFS has conducted various seismic investigations and analyses at its proposed Independent Spent Fuel Storage Installation ("ISFSI") site on the Skull Valley Band of Goshute Indian Reservation. First, there was the initial work conducted to support the filing of PFS's license application in June 1997. Then, in 1999, PFS's contractor, Geomatrix Consultants, Inc., conducted a more extensive seismic investigation of the site. Finally, Geomatrix conducted a further seismic site investigation or seismic re-evaluation in Spring 2001. From these various investigations, PFS, using different criteria or methodologies, has developed a series of proposed design basis ground motions for the site.

When PFS submitted its original application to the NRC in June 1997, it showed the peak ground accelerations at the site to be 0.67g in the horizontal direction and 0.69g in the vertical direction. SAR, Rev. 0, Table 3.6-1 (sheet 2 of 5). Since then, PFS has estimated peak ground accelerations ranging from a low of 0.40g (horizontal) and 0.39g (vertical)[3] to a

[3] Presented by PFS in 1999 using a PSHA with a 1,000 year return period. *See* Final Safety Evaluation Report at 2-37. A change in the return period from 1,000 years to 2,000 years, produced estimated ground motions of 0.53g (horizontal) and 0.53g (vertical). Id.

high of 1.15g (horizontal) and 1.17g (vertical).[4] PFS is currently relying upon a PSHA with a 2,000 year return period which estimates peak ground accelerations to be 0.711g (horizontal) and 0.695g (vertical). SAR Rev. 21 at 2.6-107. The newly revised PHSA design basis ground motions have markedly increased[5]; concomitantly there has been a significant reduction in the factor of safety against seismic stability of the CTB, storage pads and storage casks.

In an effort to demonstrate adequate stability for the new and higher levels of design basis ground motions, PFS is now proposing extensive use of soil cement around the CTB foundation and to significantly modify the design of the soil cement in the pad emplacement area.[6] It is only through the addition of soil cement around the CTB and the redesign of the soil cement in the pad emplacement area that PFS intends to demonstrate adequate resistance to dynamic sliding for the revised design basis ground motions.[7]

The proposed redesign of the soil cement treatment in the pad emplacement area consists of three significant changes. First, reduction of the depth of soil cement treatment in the pad emplacement area. *See* Exhibit 4 at 8. Second, reduction of the percentage of cement that is to be added to the soil immediately underneath the pads. Previously, PFS had proposed to use a 1-foot minimum thick layer of soil cement with a design strength of 250 psi. *See* SWEC Calc. No. 05996.02-G(B)-4, Rev. 6, *Stability Analysis of Storage Pads*. Currently,

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[4] *See* Geomatrix, *Update of Deterministic Ground Motion Assessment*, Rev. 1, April 2001 at 3.

[5] In fact, there has been an approximate 35 percent increase in the new design peak ground accelerations above the design basis accelerations proposed in 1999 for the 2,000 year return period PSHA.

[6] *See* April 15, 2001 letter from Mr. Donnell, PFS to NRC, Summary of Changes for PFSF License Application Amendment # 22, at 8, attached hereto as Exhibit 4.

[7] *See* Stone and Webster ("SWEC") Calc. 05996.02-G(B)-13, Rev. 4, *Stability Analysis of Canister Transfer Building* ("G(B)-13, Rev. 4"); SWEC Calc. 05996.02-G(B)-4, Rev. 7, *Stability Analysis of Storage Pads* ("G(B)-14, Rev. 7").

PFS is proposing to reduce the soil cement in this same layer to achieve a minimum design strength of 100 psi. SAR, Rev. 21, at 2.6-109. Third, increase in the amount of cement treatment from 6.0 to about 8.5 percent in the soil adjacent to the storage pads. SWEC Calc. G(B)-4, Rev. 7. The first and third changes impact the dynamic stability calculations and the second change affects both the dynamic stability and the cask tipover analysis for the newly revised design basis ground motions. Bartlett Dec. ¶¶ 12, 13, Ostadan Dec. ¶¶ 11, 14.

Regarding the second design modification, PFS is now proposing to use two different soil cement mixes for the pad emplacement area. The first mix is to be placed around the pads and has a revised design strength of about 340 psi. SAR, Rev. 21, at 2.6-60. This layer is intended to provide horizontal resistance against sliding. The second mix will be placed under the pads and has a design strength of 100 psi. SAR, Rev. 21, at 2.6-109. This underlying layer is supposed to perform two functions: comply with the modulus of elasticity requirement of the hypothetical cask tipover analysis and yet still provide sufficient resistance to sliding. Exh. 4 at 8; SAR Rev. 21 at 2.6-109. Significantly, PFS has failed to recognize the difference between the static and dynamic modulus of elasticity for the cement-treated soil and the effect of confining pressure under both static and seismic loading on the modulus, thus violating the basic requirements for protection against cask drop/tipover conditions. Ostadan Dec. ¶ 11(f).

Regarding the first and third design modifications, PFS, in its revised seismic investigation and analyses, has inadequately assessed the impacts, effects and performance of cement-treated soil (Bartlett Dec. ¶¶ 9-21) and has failed to correct many of the deficiencies that the State raised in Utah L. Ostadan Dec. ¶ 8. For example, PFS still has not considered

the effect of inclined waves on the stability of the CTB, and storage pad foundations and the casks, nor has it considered the effect of tensile and bending stresses caused by such waves on the cement-treated soil. Id. ¶ 12; Bartlett Dec. ¶ 15. Also, PFS has ignored the effect of the pad and mat foundation flexibility and has not considered a range of applicable out-of-phase motions and their effects to the pads and the casks. Ostadan Dec. ¶ 13. Further PFS has not considered multiple time histories; and it has not used the real interface conditions with non-uniform friction ratios in analyzing dynamic response of the casks. Id. ¶ 11. These deficiencies are exacerbated and safety further compromised now that there has been a significant increase in the design basis ground motions. Id. ¶ 8-10.

The most serious safety-related concern is PFS's wide and pervasive attempt to use "soil-cement" as a structural element in the foundation design for the CTB and storage pads without providing sufficient evaluations, testing, calculations and design to demonstrate that the cement-treated soil will perform its intended functions, both under seismic loading and long-term operational conditions. Bartlett Dec. ¶¶ 9-21, Mitchell Dec. ¶¶ 9-14. While soil cement has been used for highway road base improvement, slope protection, and linings, PFS's intent to employ cement-treated soil as a buttress to resist seismic sliding forces in structures, systems and components important to safety ("SSCs")[8] is unprecedented and unproven. Further, PFS proposes to delay necessary strength testing until the construction phase, without first demonstrating that the soil cement concept will perform its intended

[8] In its classification of SSCs, PFS has classified the canister as Category A (Critical to Safe Operation) and the following SSCs as Category B (Major Impact on Safety): storage cask, transfer cask, associated lifting devices, Canister Transfer Building, canister transfer overhead bridge crane, canister transfer semi-gantry crane, and seismic support struts. SAR, Rev. 17 at 3.4-3, 3.4-4 and Table 3.4-1.

function of providing seismic stability. SAR Rev. 21 at 2.6-108-09. Prior to obtaining an ISFSI license, PFS must demonstrate compliance with Part 72, including 10 CFR § 72.122(b)(2), the ability of SSCs to withstand the effects of earthquakes. Postponing the design of elements that are key to safety until the construction phase (*i.e.*, after license issuance) does not satisfy 10 CFR § 72.122(b)(2).

PFS has not shown by case history precedent or by site-specific testing and dynamic analyses that the cement-treated soil will be able to resist earthquake loadings for the CTB and storage pad foundations. Bartlett Dec. ¶ 12, Mitchell Dec. ¶ 11. Furthermore, PFS has not evaluated the long term behavior of cement-treated soil under operational loading from the canister transport vehicle and from environmental factors such as curing, shrinkage, moisture content changes, frost, dessication, and chemical attack by salts and sulfides. Bartlett Dec. ¶¶ 14-21, Mitchell Dec. ¶ 13. The interaction of these factors may crack or significantly degrade the strength and performance of the cement-treated soil over the proposed 40 year design life.

As described herein and in the supporting declarations, PFS's revised approach and redesign to deal with the increased strong ground motions does not satisfy 10 CFR § 72.102(c) or (d)[9] or § 72.122(b)(2)[10]. The following highlights the most significant errors,

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[9] The requirements of 10 CFR § 72.102 are, in part:

> (c) Sites other than bedrock sites must be evaluated for ... other soil instability due to vibratory ground motion.
>
> (d) Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading.

[10] 10 CFR § 72.122(b)(2) requires:

> 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

omissions or potentially unconservative assumptions and estimates in PFS's evaluation of the dynamic forces that affect the CTB and the storage pads, and the ability of the soil, soil-cement, foundation system, and storage casks to resist those seismic loads.[11] These concerns show that the revised design is unsupportable and creates significant safety concerns.

1. Application of the New Design Basis Ground Motions to the CTB and its Foundation System.

The newly revised ground motions have significantly increased. However, in applying these revised ground motions, PFS has made several potentially unconservative assumptions regarding the redesign of the CTB and its foundation system. For example, the calculation *Development of Soil Impedance Functions for canister Transfer Building* (SWEC 05996.02-SC-4, Rev. 2), assumes that the large mat of the CTB is rigid. *See* Mitchell Dec. ¶¶ 9-10. This assumption leads to overestimation of the foundation damping and underestimation of the seismic loads. Ostadan Dec. ¶ 12. In this calculation, and the follow-up calculation *Seismic Stability of the Canister Transfer Building*, SWEC calc. 05996.02-SC-5, Rev. 2, the effect of the large cement-treated soil mass around the building has been ignored. The presence of a much stiffer cement-treated soil cap around the building to the extent of one building dimension on each side impacts the soil impedance parameters and kinematic motion of the

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> must reflect [the most severe reported natural phenomena]....
> The ISFSI or MRS should also be designed to prevent massive collapse of building structures or the dropping of heavy objects as a result of building structural failure on the spent fuel or high-level radioactive waste or on to structures, systems and components important to safety.

[11] As discussed in more detail under the Late Filed Factors, *infra*, PFS has submitted about a six inch stack of technical documents that the State's experts needed to review in thirty days (*see e.g.*, Ostadan Dec. ¶ 7) but PFS's submission is still incomplete. Therefore, given the volume of material to review and the incomplete nature of PFS's submission, at this stage, the State can only point to the most serious deficiencies in PFS's seismic re-evaluation. These deficiencies, however, are significant and threaten the safety of the facility.

foundation. Ostadan Dec. ¶ 12; Bartlett Dec. ¶ 16. The overestimation of damping, underestimation of seismic loads, and failure to analyze the effect of cement-treated soil on the foundation systems, are serious oversights which create unacceptable uncertainties in the estimation of the true seismic loadings and their potential impacts to the foundations.

In the CTB stability analysis, SWEC Calc. G(B)-13, Rev. 4, PFS relies on passive earth pressure from soil cement to resist seismic loads and estimates that a 0.39 inch moment of the building is acceptable and sufficient to develop the needed passive pressure. Ostadan Dec. ¶ 13. This assumption is insufficient and is only valid under ideal conditions. The out-of-phase motion of the building and the cement-treated soil pad is expected to develop cracking and separation around the foundation. Id. ¶ 13, Bartlett Dec. ¶13, Mitchell Dec. ¶ 9. Moreover, the cement-treated soil will experience tensile and bending stresses under seismic excitation. Ostadan Dec. ¶ 13, Bartlett Dec. ¶ 15, Mitchell Dec. ¶ 11. Also, PFS has estimated a total settlement of three inches for the CTB. SAR Rev. 21 at 2.6-74. The differential settlement between the foundation and the surrounding cement-treated soil would cause cracking of the cement-treated soil propagating away from the foundation. Consequently, the ability of the cement-treated soil to resist dynamic forces without cracking and to provide the passive resistance required to maintain stability is of serious concern and negates PFS's conclusion about stability of the CTB and the storage pads.

2. Application of the New Design Basis Ground Motions to the Storage Casks and the Storage Pads.

PFS has also made several potentially unconservative assumptions in applying the revised ground motions to the stability of the storage pads. The stability analyses of the storage casks are deficient in a number of areas. First, Holtec International's analyses in

Multi Cask Response at PFS ISFSI from 2000-Yr Seismic Event, Rev. 2, is nonlinear and has not considered the range of applicable phasing of the foundation pad motion and the casks motion, the actual interface conditions between the casks and the pad on cement-treated soil, and the applicable wide range of phasing relationship in input time histories and types of waves striking the pads. Ostadan Dec. ¶¶ 10 and 11. The most critical component in stability evaluation of the casks and the dynamic loads acting on the pads, is 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. The stability of the free standing casks under such high intensity of ground motions is of serious concern and does not support PFS's conclusion that the casks will not tip over. Id.

Second, in the stability analysis of the pads, G(B)-04, Rev.7, PFS has failed to consider the natural frequency of the cask-pad-cement-treated soil system, thus signifi-cantly underestimating the seismic loads. Ostadan Dec. ¶ 14. Third, the actual load path under seismic loading has not been considered. While it has been shown that the effect of soil-structure interaction is important in the seismic response of the cask-pad-cement-treated soil system, PFS has ignored the effect of pad-to-pad interaction for pads spaced only five feet apart in the longitudinal direction. In the stability analysis, the passive resistance for one pad will act as a pushing force on the next pad. This interaction has been totally ignored in the evaluation, thus seriously violating the conclusion on the stability of the pads. Id. ¶ 14. Fourth, the actual behavior of the cement-treated soil under tensile and bending stresses, and the cracked conditions, and separation of cement-treated soil from the pads have been ignored. Id., Bartlett Dec. ¶¶ 13, 15, 17. Furthermore the estimated static settlement for the

pads is shown to be 3.3 inches. SAR Rev. 17 at 2.6-50. The differential settlement between the pad and the surrounding cement-treated soil causes bending and cracking of the cement-treated soil propagating away from the pad. Ostadan ¶ 14. These deficiencies are major concerns on the adequacy of the pad-cement-treated soil design to resist seismic loads.

3. Survivability and Durability of Cement-Treated Soil for the Redesigned CTB and Storage Pad Foundation Systems.

In the redesign of the foundation systems for the CTB and storage pads for the revised strong ground motions, PFS has made extensive use of cement-treated soil in an attempt to demonstrate adequate resistance to dynamic sliding of the CTB and storage pads. *See* SWEC Calc. G(B)-13, Rev. 4; SWEC Calc. G(B)-4, Rev. 7. This idea is still conceptual and PFS has neglected or not considered many necessary design elements. Bartlett Dec. ¶¶ 9-10, Mitchell Dec. ¶ 11. The use of cement-treated soil for this specific application has not been supported by precedent, site-specific evaluations and testing, and engineering analyses and design. Bartlett Dec. ¶¶ 9-12, Mitchell Dec. ¶13. If PFS is to meet the Part 72 siting requirements, it is imperative that PFS provide further supporting documentation because PFS's lack of support for its cement-treated soil concept makes it impossible to complete an objective and independent review of the use of cement-treated soil as an integral part of the foundation system to resist strong ground motions produced by the design basis earthquake.

As described below, PFS has not shown that its proposal to use cement-treated soil will perform as intended. The cement-treated soil may not perform its intended function -- provide dynamic stability to the foundation system[12] -- because PFS has not addressed

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[12] *See* SWEC Calc. G(B)-13, Rev. 4; SWEC Calc. G(B)-4, Rev. 7.

several possible failure mechanisms during conceptual design. Significant concerns with soil treated cement's ability to withstand dynamic bending, torsional, and beam shear stresses; long-term durability without cracking or without significant shear strength degradation; and interaction with soil chemistry remain unaddressed. Such a deficient analysis raises serious safety concerns on the seismic performance of the use of the proposed cement treatment.

a Overstressing and Cracking due to Dynamic Bending, Torsional, and Beam Shear Stresses.

The cement-treated soil will be subjected to tensile stresses from such factors as static loading, shrinkage and dynamic loading. Mitchell Dec. ¶ 11; Bartlett Dec. ¶ 15. Of particular concern is the effect on the cement-treated soil from dynamic loading. In particular, the pads and the cement-treated soil could experience high bending stresses under seismic loads, especially given the large weight of the cask, its relatively small diameter, and the relative length of the pad. Mitchell Dec. ¶ 11. Unlike the use of soil-cement bases in heavy duty pavement where there is a large structural layer of asphalt concrete or Portland Cement Concrete to resist bending stresses, this structural layer is absent in PFS's proposed design and those bending stresses can only be resisted by the relatively weak cement-treated soil. Id. Consequently, PFS cannot demonstrate the seismic performance of the proposed cement treatment unless and until it calculates the magnitude of these bending stresses and their effect on the proposed cement-treated soil. Id.

b. Delamination or Debonding along a Cement-Treated Soil Lift Interface.

Dynamically induced bending stresses will also introduce beam shear stress, *i.e.*, shear stress between layers of a laminated material. Because cement-treated soil is constructed in lifts, preferential planes of weakness may form along these planes and the layers may become

debonded during a seismic event unless care is taken to properly prepare the interface before the placement of the next lift. Bartlett Dec. ¶ 17. PFS has not evaluated the magnitude of these stresses, or their impact to the cement-treated soil.

c. Shrinkage Cracking Due to Drying and Curing.

Cases of deleterious shrinkage cracking from curing of cement-treated soils are well documented in the literature. Bartlett Dec. ¶ 18. If significant shrinkage cracking of the cement-treated soils occurs at the PFS site, it will reduce the passive earth pressure, shear and tensile strengths available to resist seismic forces. Id. PFS has not addressed these concerns in its redesign of the CTB and pad emplacement areas. Moreover, PFS proposes to cover the pad emplacement area with gravel so that it will be essentially impossible to observe and assess the degree and nature of shrinkage cracking with time. Id.

d. Cracking due to Vehicle Loads

The canister transport vehicle will be active in the pad emplacement area. PFS has made no assessment of the structural capability of the cement-treated soil layer to resist wheel loading without fatigue damage. Bartlett Dec. ¶ 20. Fatigue damage at the interface of the cement-treated soil with the pads could seriously compromise the cement-treated soil's ability to resist the new design basis ground motions.

e. Long-term Performance of Cement-treated Soil Over a 40 Year Period.

In License Amendment No. 22 and related calculations, PFS proposes to use "soil cement" around the CTB foundation and in the redesign of the pad emplacement area to compensate for new and higher levels of design basis ground motions at the site. However, a major deficiency in PFS's proposal is that it has conducted no site specific testing to

determine the strength, survivability and durability properties of the cement-treated soil. Bartlett Dec. ¶ 9, Mitchell Dec. ¶13. This is a significant omission in the conceptual and final design of this novel approach that PFS intends to employ at the site.

From the standpoint of strength and durability, it is important to distinguish between "soil-cement" and "cement-treated soil." Mitchell Dec. ¶ 12. Soil cement has a cement content that is sufficient to attain minimum durability standards as measured by wet-dry and freeze-thaw tests. Id. From the modest amount of cement that PFS proposes to use,[13] it is apparent that PFS will use cement-treated soil and this will significantly and adversely affect the durability of the cement-treated soil to wet-dry and freeze-thaw cycles. Id. PFS has not demonstrated durability of the proposed cement-treated soil from wet-dry and freeze-thaw cycles.

In addition, PFS has not come forward with any information on the chemistry of the surficial soils. Mitchell Dec. ¶ 13. Salts and sulfates, if present, could interfere with the cement hydration, and thus affect the strength and durability of the cement-treated soils. Id.

4. Overestimation of the Sliding Resistance Provided by the Clayey-silt and Silty-clay Underlying the CTB and Storage Pads.

PFS has potentially overestimated the sliding resistance provided by the clayey-silt and silty-clay underlying the CTB and storage pads.[14] PFS assumes that approximately 2 kips per square foot of sliding resistance will be available in the silty-clay, clayey-silt layer to resist

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[13] *See* SWEC Calc. G(B)-13, Rev. 4, and SWEC Calc. G(B)-4, Rev. 7.

[14] *See* SWEC Calc. No. 05996.02-G(B), Rev. 3, *Documentation Basis for Geotechnical Parameters Provided in Geotechnical Design Criteria* ("SWEC Calc. G(B), Rev. 3"); SWEC Calc. G(B)-13, Rev. 4; SWEC Calc. G(B)-4, Rev. 7.

earthquake forces.[15] In this evaluation, PFS has not considered the effects of ad-hesion and potential water content changes during cement-treated soil placement and other long-term moisture content changes (Bartlett Dec. ¶¶ 22, 25, Mitchell Dec. ¶ 14); seismically generated pore pressures on the soil's shear strength during earthquake loading (Bartlett Dec. ¶24); and partial mobilization of the undrained shear strength by the free-field ground motion (id. ¶ 23). PFS has not demonstrated that the applied design shear strength value is representative of actual conditions and sufficiently conservative for design of the CTB and storage pads.

LATE FILED FACTORS:

The State meets the 10 CFR § 2.714(a) late filed factors for Contention Utah QQ.

Good Cause: The State has good cause for late filing Utah QQ. PFS's revised geotechnical investigation showed an increase in the design basis ground motions of about 35 percent above the design basis PFS was relying upon prior to its revised seismic investigation. This resulted in a significant amendment to PFS's license application ("Amendment 22"), including PFS's new proposal to use soil-cement around the CTB and a revision in the use of soil cement under and around the storage pads. As a consequence of the higher design basis ground motions, all dynamic analyses previously conducted had to be revised and re-evaluated. The State received calculations relating to Amendment 22 on various dates between April 6 and 16, 2001. In response to a motion by the State, the Board established May 16, 2001 for "any April 2001 application amendment late-filed submission regarding CTB design changes, including use of soil cement, or revisions to storage pad analyses, soils analyses, soil-cement design calculations/analyses, and Holtec site-specific

[15] SWEC Calc. G(B), Rev. 3; SWEC Calc. No. G(B)-13, Rev. 4.

cask analyses...." April 26, 2001 Board Order at 3.

Utah QQ addresses the issues enumerated in the Board's April 26 Order. Utah QQ addresses the deficiencies in PFS's new plan to use cement-treated soil as a structural design element around the CTB, an issue the State could not have raised in the past. As to the use of soil-treated cement in the pad emplacement area, Utah QQ addresses both the effect of the reduction in the depth of soil cement treatment and the percentage of cement added to the soils immediately underneath the pads as well as an increase in the amount of cement treatment in the soil adjacent to the storage pads. Previously, PFS intended to use cement-treated soil as a construction cost savings measure.[16] Now cement-treated soil is a structural design element of the storage pads. Again, this design change occurred in Amendment 22.

The newly revised ground motions significantly increase the seismic demand on the design of the storage pads, the CTB, and the unanchored casks and has exacerbated the deficiencies in Holtec's site-specific cask analyses. In its revised calculation, Holtec uses many of the same incorrect assumptions that it did in its original analyses (*e.g.*, assume the casks will slide in a controlled manner during an earthquake), as does Stone & Webster in its dynamic analyses of the CTB and storage pads. Utah QQ challenges those incorrect assumptions and also challenges PFS's novel concept in the use of soil cement. PFS has done no testing or analyses for determining the strength and durability properties of the cement-treated soils. Even if the State could have raised some of these issues at an earlier

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[16] By removing and re-using the surficial layer of eolian silt and mixing it *in situ* with cement to use under the foundation of the pad emplacement area, PFS saves the costs of hauling the silt off site and of bringing in new fill material. *See e.g.*, SAR, Rev. 13 at 2.6-108.

date, which the State has done in Utah L,[17] the issues enumerated in Utah QQ relate to the increase in design basis ground motions, Amendment 22 and related calculations.

Development of a Sound Record: The State will assist in the development of sound record regarding the issues it has raised in this proceeding. The State brings to this contention three respected experts. Dr. Farhang Ostadan, an expert in soil-structure interaction from the effects of earthquakes, has more than 20 years experience in the dynamic analysis and seismic safety evaluation of above and under ground structures and subsurface materials. Ostadan Dec. ¶ 2. He co-authored and implemented the computer program "SASSI" used by PFS to model soil-structure interaction at the Skull Valley site. Id. This program is used world wide for such purposes. Id. As shown in his curriculum vitae, Dr. Ostadan has considerable expertise in the dynamic analysis of nuclear facilities. Id. ¶ 3.

Dr. Bartlett, an expert in soil behavior under seismic loading, has performed seismic analyses for the Department of Energy High Level Waste Facilities at Savannah River. As a project engineer for Woodward-Clyde and a research project manager at the Utah Department of Transportation, he researched and analyzed the construction and performance of lime-cement-treated soil and the consolidation, shear strength, and seismic response of the Lake Bonneville deposits, the same deposits that are found at the PFS site. Bartlett Dec. ¶¶ 2-4. Drs. Ostadan and Bartlett, through their depositions and support of the State's Response to Summary Disposition of Utah L, have already demonstrated their expertise and familiarity of the seismic issues involved with the PFS site. They were responsible for

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[17] *See* Ostadan and Bartlett Declarations in support of the State's Response the Applicant's Motion for Summary Disposition of Utah L (January 30, 2001).

alerting PFS to its failure to consistently integrate data across various disciplines involved in the PFS seismic investigation, in particular the conflict in the soil velocity data.

Dr. Mitchell, a renowned expert in seismic stability and soil properties, including the use of soil cement, has over 40 years experience in geotechnical engineering and was a member of the Civil Engineering faculty at the University of California at Berkeley for more than 35 years. His recent and current projects include seismic stability studies of dams, embankments, airports and highways, as well as service on numerous national technical committees, including NRC study committees. Mitchell Dec. ¶¶ 1-5.

The State's three experts will be of invaluable assistance to the Board in describing the complex issues and the shortcomings in PFS's attempt to design SSCs to withstand the strong design basis ground motions at the PFS site. They are all prepared to offer testimony as described in and consistent with their supporting declarations. Ostadan ¶ 6; Bartlett ¶ 7; and Mitchell ¶ 6. Of course, new information from PFS could change their analyses.

Availability of Other Means for Protecting the State's Interests: The State has no means, other than this proceeding, of protecting its interests. Even if the Staff issues a supplement to the SER, there is no opportunity, other than through this proceeding, where the State may present its critical safety concerns, as identified by its experts, or affect the Staff's review of the way in which PFS will address the strong ground motions at the site.

Representation by Another Party: The State is the only party to this proceeding who has challenged PFS's seismic analysis of the Skull Valley site, and thus, the State's interests in this matter are not and will not be represented by any other party.

Broadening of Issues or Delay of the Proceeding: Litigation of this issue may somewhat broaden the proceeding because PFS has introduced a new component into its seismic analysis: a significant increase in design ground motions at the site and the conceptual use of soil cement to overcome those increased ground motions. Whether this issue can be accommodated in the current litigation schedule is, in part, a function of whether PFS can produce sufficient documentation to satisfy the safety concerns raised by the significant increase in ground motions.[18] Even if the current litigation schedule is delayed, it is through no fault of the State and, moreover, the importance of the safety concerns warrants a complete record and hearing on the seismic stability of the CTB, storage pads and storage casks as well as the underlying soils.

Impact of the Admission of Utah QQ on Utah L: The Board requested the parties to discuss the impact, if any, on the admission of a late filed contention "on the matters currently pending before the Board in connection with the PFS dispositive motion on contention Utah L, Geotechnical." Board Order at 3. Utah QQ does not relate to any of the issues in Bases 1 or 2 of Utah L. There is some overlap between Utah L, Basis 3 and Utah QQ, in that PFS has used the same invalid assumption in re-analyzing the dynamic stability of the CTB, storage pads, cement-treated soil in the pad emplacement area, underlying foundation soils, and cask stability for the newly revised ground motions. The newly revised ground motions, however, create greater seismic loads on the CTB, pads and stability of the casks. PFS's plans to use cement-treated soil around the CTB and change the

[18] *See* May 7, 2001 letter from E. William Brach, NRC to John D. Parkyn, PFS with enclosed Data Needed for the Completion of the PFS LA Amendment, attached hereto as Exhibit 5.

soil cement treatment under and around the pads in an effort to solve many of the stability problems associated with foundation loading raised in Utah L. Most of the problems raised by the State in Utah L remain unaddressed in PFS's latest seismic evaluation and have been amplified due to the increase in design motion. PFS seems to have recognized the need to improve seismic stability at the site and yet it has not demonstrated that the new soil cement design element is able to solve the compelling foundation stability problems. Admission of Utah QQ does not support PFS's motion for summary disposition of Utah L. Instead, Utah QQ strengthens the record for denial of PFS's motion.

The Staff has not yet set its schedule for its review of PFS's revised seismic analysis and has warned PFS that failure to produce all necessary information required by the Staff to conduct its review may delay the hearing schedule. Exhibit 5 at 1. If the schedule is delayed, Utah QQ and Utah L could both be set for hearing at the same time.

CONCLUSION

For the reasons stated herein, the State's requests the Board to admit Utah QQ.

DATED this 16th day of May, 2001.

Respectfully submitted,

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CERTIFICATE OF SERVICE

I hereby certify that a copy of STATE OF UTAH'S REQUEST FOR ADMISSION OF LATE-FILED CONTENTION UTAH QQ (Seismic Stability) was served on the persons listed below by electronic mail (unless otherwise noted) with conforming copies by United States mail first class, this 16th day of May, 2001:

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NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of:	)	Docket No. 72-22-ISFSI
PRIVATE FUEL STORAGE, LLC	)	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel Storage Installation)	)	May 16, 2001

DECLARATION OF DR. FARHANG OSTADAN

I, Dr. Farhang Ostadan, hereby declare under penalty of perjury and pursuant to 28 U.S.C. § 1746, that:

1. I hold a Ph.D. in civil engineering from the University of California at Berkeley. I am a consultant in the field of soil dynamics and geotechnical earthquake engineering. I am also a visiting lecturer at the University of California at Berkeley and teach a graduate course on soil dynamics and soil-structure interaction. My curriculum vitae listing my qualifications, experience, training, and publications has already been filed in this proceeding. *See*, Exhibit No. 2 of the "State's Motion to Compel Applicant to Respond to State's Fifth Set of Discovery Requests" (December 20, 1999).

2. I have more than 20 years experience in dynamic analysis and seismic safety evaluation of above and underground structures and subsurface materials. I co-developed and implemented SASSI, a computer program for seismic soil-structure interaction analysis currently in use by the industry worldwide. I am also the technical sponsor of this program in collaboration with the University of California at Berkeley.

3. I have participated in seismic studies and review of numerous nuclear structures, among them Diablo Canyon Nuclear Station; the NRC/EPRI large scale seismic experiment in Lotung, Taiwan; the large underground circular tunnel for Super Magnetic Energy Storage; General Electric ABWR and SBWR standard nuclear plants; Westinghouse AP600 standard nuclear plant; Tennessee Valley Authority nuclear structures (Browns Ferry, Sequoyah, Watts Bar); and the ITP, RTF, and K-facilities in the Savannah River Site for the Department of Energy. I have published numerous papers in the area of soil structure interaction and seismic design for nuclear and other structures.

4. I have reviewed the Applicant's SAR sections, and updates thereof, relating to its geotechnical investigation of the proposed site, and relevant calculations, reports, and other documents prepared by the Applicant or its contractors and submitted to the NRC or produced to the State in discovery. I have participated in answering the Applicant's discovery to the State as well as assisted in the preparation of discovery for the State directed to the Applicant. I am familiar with and have applied NRC regulations and guidance documents as they relate to geotechnical review.

5. I was designated one of the State's testifying experts for this proceeding on January 31, 2000, and deposed on a panel with Dr. Steven F. Bartlett by Private Fuel Storage ("PFS") on November 16 and 17, 2000. I was present at the State's deposition of PFS's geotechnical witnesses, Drs. Paul J. Trudeau and Thomas Y. Chang, held on November 14, 2000. In addition, I have filed a declaration in support of the State's Response to the Applicant's Motion of Summary Disposition of Utah Contention L.

6. I provide this declaration in support of the State of Utah's Request for Admission of Contention Utah QQ and, if admitted, I am prepared to offer testimony consistent with this declaration.

7. Specific to Contention Utah QQ, I have reviewed the relevant sections the of PFS's License Amendment No. 22, including errata to correct SAR Ch. 2, and calculations relating to License Amendment No. 22, in particular:

    a. *PFSF Site-Specific HI-STORM Drop/Tipover Analyses*, Rev. 0, Holtec Report No. HI-2012653 (4/3/01);
    b. *Multi Cask Response at PFS ISFSI from 2000-Yr Seismic Event*, Rev. 2, Holtec Report No. HI-2012640 (3/29/01);
    c. *Storage Pad Analysis and Design*, Calc. No. 0599602-G(PO17)-2, Rev. 3, 4/5/01 (ICEC);
    d. Calc. No. 05996.02-SC-4, Rev. 2, *Development of Soil Impedance Functions for Canister Transfer Building*, 3/21/01 (Stone & Webster ("SWEC"));
    e. Calc. No. 05996.02-SC-5, Rev. 2, *Seismic Analysis of Canister Transfer Building*, 4/4/01 (SWEC);
    f. Calc. No. 05996.02-G(B)-04, Rev. 7, *Stability Analysis of Storage Pads*, 3/30/01 (SWEC);
    g. Calc. No. 05996.02–G(B)-13, Rev. 4, *Stability Analysis of the Canister Transfer Building*, 3/30/01 (SWEC);
    h. Calc. No. 05996.02(GPO18)-2, Rev. 1, *Soil and foundation parameters*

for dynamic soil-structure interaction analysis, 2000-year return period design ground motions, 3/21/01 (Geomatrix);

i. Calc. No. 05996.02F(PO18)-3, Rev. 1, *Development of Time Histories for 2000-year return period design spectra*, 3/21/01(Geomatrix);

j. *Development of Design Basis Ground Motions for the Private Fuel Storage Facility*, Rev. 1, March 2001 (Geomatrix); and

k. *Update of Deterministic Ground Motion Assessments*, Rev. 1, April 2001 (Geomatrix).

8. I emphasized in my deposition, as did Dr. Bartlett, that there was a discrepancy between PFS's shear wave velocity data that PFS initially obtained from seismic refraction data and its later seismic cone penetration test data. Since that time, PFS has conducted further investigations and re-evaluated its design basis ground motion, which has now increased significantly. In spite of PFS's revised seismic investigation, many of the State's previous concerns still remain but now new concerns have also emerged because of a significant increase in the design motion and PFS's use of soil cement in an attempt to overcome foundation stability problems.

9. At the time PFS requested an exemption from the seismic regulations, PFS estimated ground motion at the site had a peak ground acceleration of 0.72g in the horizontal direction and 0.80g in the vertical direction using a deterministic seismic hazard analysis ("DSHA"). PFS's revised DSHA shows ground motion with peak ground acceleration of 1.15g in the horizontal direction and 1.17 g in the vertical direction. The newly revised ground motion, based on a probabilistic seismic hazard analysis methodology using a 2,000-year return period, has a peak ground acceleration of 0.711 g in the horizontal direction and 0.695 g in the vertical direction. The increase in the ground motion increases the demand on the design of the storage pads and Canister Transfer Building as well as on the unanchored casks placed on the pads.

10. Based on the increase in ground motions, Holtec International, the cask vendor, had to revise its earlier analysis of the cask-pad and to include soil cement system. I have significant concerns with Holtec's revised analysis and the invalid assumptions upon which Holtec relied. This is important because evaluating the adequacy of the foundation design is a function of the dynamic forces that will be imparted to them. In order to evaluate the response of the foundation or the soil cement to resist seismic loads and to evaluate the stability of the casks, it is critical to understand the seismic loads and the assumptions made in calculating the seismic loads.

11. Holtec Report No. HI-2012640 titled *Multi Cask Response at PFS ISFSI from 2000-Yr Seismic Event*, Rev. 2, evaluated the stability of the casks on the pads and provided the seismic loads for design of the pad-soil cement system. This report has the following major deficiencies:

a. Since the analysis of the cask-pad-soil cement is a nonlinear analysis, it is very important to consider all potential variation in the motion of the pad and the casks. If the pads and the casks move out of phase, significant instability conditions arise. The soil spring and damping used in the Holtec analysis do not properly consider the frequency dependency of these parameters. Holtec has provided no check to compare the parameters used by other available rigorous solutions to ensure the foundation parameters are reasonably accurate. In the calculation of soil spring and damping for the Canister Transfer Building ("CTB") (Calc. No. 05996.02-SC-4, Rev. 2, *Development of Soil Impedance Functions for Canister Transfer Building*, 3/21/01, SWEC) it has been shown that the soil spring and damping are highly dependent on the frequency due to soil layering at the site. The contrast in the dynamic properties of the underlying stratum has significantly increased by inclusion of soil cement in the foundation thus increasing the concern on frequency dependency of soil spring and damping.

b. Holtec has assumed that the pads are rigid. Based on the results reported in *Storage Pad Analysis and Design*, Calc. No. 0599602-GPO17-2, Rev. 3, 4/5/01, ICEC), this assumption is not valid and results in erroneous calculation of the foundation damping and also violates the assumption that the coefficient of friction between the casks and the pad is necessarily constant. For further discussion, *see* Bartlett/Ostadan Tr. at 362-365, 371, 374, Exh. 6 to State of Utah's Response to Applicant's Motion for Summary Disposition of Utah Contention L (January 30, 2001).

c. Since the PFS site is located close to a set of major faults dipping under the site (*see Development of Design Basis Ground Motions for the Private Fuel Storage Facility*, Rev. 1, March 2001, Geomatrix), seismic waves arriving at foundation structures are not necessarily vertically propagating waves. This is contrary to Holtec's assumption. The waves striking at angles will cause additional rocking and torsional motion of the foundation above and beyond the motion caused by vertically propagating waves.

d. Holtec's nonlinear analysis is sensitive to phasing of the input motion and thus multiple time histories should be used. Only one set of time histories developed by Geomatrix (Calc. No. 05996.02F(PO18)-3, Rev. 1, *Development of Time Histories for 2000-year return period design*

spectra, 3/21/01) has been used by Holtec.

e. Holtec has assumed that the casks will slide on the pad in a controlled manner during a large earthquake. There is no other redundancy built into Holtec's expected design. But such a bold assumption is negated by the potential that cold bonding between the cask and the pad may occur over time. When two bodies (cask and pad) with such a large load (the cask) are in contact, some local deformation and redistribution of stresses may occur at the points of contact which would create a bond, and this would not allow the cask to slide on the pad or move smoothly during an earthquake. *See* Bartlett/Ostadan Tr. at 346, 365-368, 372-373.

f. In the drop/tipover analysis of the casks (*PFSF Site-Specific HI-STORM Drop/Tipover Analyses*, Rev. 0, Holtec Report No. HI-2012653, 4/3/01), Holtec has assumed a lower stiffness of the soil cement under the pad to meet the drop/tipover condition. In doing so, it has failed to recognize the difference between the static and dynamic modulus of the soil cement and the effect of significant temporal and spatial change in bearing pressure acting on the soil cement. The expected large difference between the static and dynamic modulus invalidates the assumption made for the design.

12. There are deficiencies in the calculation of the seismic analysis of the Canister Transfer Building (Calculation No. 05996.02-SC-5, Rev. 2, *Seismic Analysis of Canister Transfer Building*, SWEC), and the supporting calculation (Calculation No. 05996.02-SC-4, Rev. 2, *Development of Soil Impedance Functions for Canister Transfer Building*, SWEC) that result in greater seismic loads than those calculated by the Applicant for the design of the CTB and its foundation. These deficiencies include:

a. Stone & Webster has erroneously assumed that the large mat of the CTB is rigid. This assumption leads to overestimation of foundation damping and underestimation of seismic loads for design of the building. *See* Bartlett/Ostadan Tr. at 394-96.

b. The effect of a large volume of soil cement around the building on the impedance functions as well as on the kinematic motion of the foundation has not been considered.

c. The effect of inclined waves on the building has not been considered.

As a consequence of these deficiencies, the stability of the canister transfer building under seismic loads cannot be confirmed.

13. In the calculation *Stability Analysis of the Canister Transfer Building Supported on a Mat Foundation*, Calculation No. 05996.02-G(B)-13, Rev. 4 (SWEC), the

Applicant relies on passive pressure from soil cement to resist seismic loads and estimates 0.39 inch moment of the building is acceptable and sufficient to develop the needed passive pressure. This assumption is insufficient and is only valid under ideal conditions. The out-of-phase motion of the building and the soil cement pad is expected to result in cracking and separation around the foundation. The soil cement will experience tensile and bending stresses under seismic excitation. Furthermore, on page 2.6-74 of the SAR Rev. 21, the Applicant has estimated a total settlement of 3 inches for the CTB. The differential settlement between the foundation and the surrounding soil cement would cause cracking of the soil cement propagating away from the foundation. The ability of the soil cement to provide the passive resistance required to maintain stability is of serious concern and does not support the Applicant's conclusion about the stability of the building.

14. In the stability analysis of the pads, the calculation *Stability Analysis of Storage Pads*, Calculation No. 05996.02, G(B)-04, Rev. 7 (SWEC), the Applicant has failed to consider the natural frequency of the cask-pad-soil cement system, thus underestimating the seismic loads significantly. In addition, the actual load path under seismic loading has not been considered. While it has been shown that the effect of soil-structure interaction is important in seismic response of the cask-pad-soil cement system, the effect of pad-to-pad interaction only five feet apart in the longitudinal direction has been ignored. In the stability analysis, the passive resistance for one pad will act as a pushing force on the next pad. This interaction has been totally ignored in the evaluation, thus seriously invalidating the conclusion of the stability of the pads. In the continuation of the stability analysis, a row of ten pads has been considered. The Applicant has ignored the fact that soil cement has limited capacity under tensile and bending stresses and cannot behave as a reinforced concrete mat. The cracking caused by out-of-phase motion of the pads and the soil cement, and the other impacts of striking seismic waves prevent the soil cement pad for ten rows of the pads to act as an integrated unit. Furthermore on Page 2.6-50 of the SAR Rev. 17, the estimated static settlement for the pads is shown to be 3.3 inches. The differential settlement between the pad and the surrounding soil cement cause bending and cracking of the soil cement propagating away from the pad. This condition invalidates assumption of an integrated foundation for ten rows of pads and also negates the validity of the passive pressure used in the stability analysis of the individual pads.

Executed this 16^{th} day of May 2001,

By ______________________________

Farhang Ostadan, Ph.D., P.E.

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Executed this 16th day of May 2001,

By Farhang Ostadan

Farhang Ostadan, Ph.D., P.E.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of:	)	Docket No. 72-22-ISFSI
PRIVATE FUEL STORAGE, LLC (Independent Spent Fuel Storage Installation)	)	ASLBP No. 97-732-02-ISFSI May 16, 2001

DECLARATION OF DR. STEVEN F. BARTLETT

I, Dr. Steven F. Bartlett, hereby declare under penalty of perjury and pursuant to 28 U.S.C. § 1746, that:

1. I am an Assistant Professor in the Civil and Environmental Engineering Department of the University of Utah, where I teach undergraduate and graduate courses in geotechnical engineering and conduct research. I hold a B.S. degree in Geology from Brigham Young University and a Ph.D. in Civil Engineering from Brigham Young University. I am a licensed profession engineer in the State of Utah.

2. Prior to this University of Utah faculty position, I worked for the Utah Department of Transportation ("UDOT") as a research project manager and have held a number of other positions with UDOT and other employers where I have applied my expertise in geotechnical engineering, earthquake engineering, geoenvironmental engineering, applied statistics, and project management. My curriculum vitae was submitted in this proceeding with the State's Objections and Response to Applicant's Second Set of Discovery Requests with respect to Groups II and III Contentions (June 28, 1999). My updated curriculum vitae is attached hereto.

3. I have also worked as a consulting engineer for 1996-1996 for Woodward-Clyde Consultants in Salt Lake City, mainly as a geotechnical designer for the I-15 Reconstruction Project.

4. Prior to my position at Woodward-Clyde Consultants, I worked from 1991-1995 for Department of Energy's ("DOE") contractor, Westinghouse, at the DOE Savannah River Site ("SRS"), near Aiken, South Carolina. I was Westinghouse's principal geotechnical investigator on a multi-disciplinary team overseeing the seismic qualification of the ITP/H-Area high-level radioactive waste storage tank farm for the SRS; the principal geotechnical investigator reviewing the Safety Analysis Report ("SAR") for the seismic qualification of Defense Waste Processing Facility ("DWPF"), which is a high-level radioactive waste vitrification and storage facility at

the SRS, and the project manager for the design of a hazardous waste landfill closure at the SRS. I used NRC regulatory guidance documents for my review of these projects.

5. I have reviewed the Applicant's SAR sections, and updates thereof, relating to its geotechnical investigation of the proposed site, and relevant calculations, reports, and other documents prepared by the Applicant or its contractors and submitted to the NRC or produced to the State in discovery. I have participated in answering the Applicant's discovery to the State as well as assisted in the preparation of discovery for the State directed to the Applicant. I am familiar with and have applied NRC regulations and guidance documents as they relate to geotechnical review.

6. I was designated as one of the State's testifying expert for this proceeding on June 28, 1999, and deposed individually and as a panel member with Dr. Farhang Ostadan by Private Fuel Storage ("PFS") on November 16 and 17, 2000. I was present at the State's deposition of PFS's geotechnical witnesses, Drs. Paul J. Trudeau and Thomas Y. Chang, held on November 14, 2000.

7. I provide this declaration in support of the State of Utah's Request for Admission of Contention Utah QQ and, if admitted, I am prepared to offer testimony consistent with this declaration.

8. Specific to Contention Utah QQ, I have reviewed the relevant sections the of PFS's License Amendment No. 22 and documents relating to License Amendment No. 22, including:

   a. Calc. No. 05996.02-SC-5, Rev. 2, *Seismic Analysis of Canister Transfer Building*, 4/4/01 (SWEC);
   b. Calc. No. 05996.02-G(B)-04, Rev. 7, *Stability Analysis of Storage Pads*, 3/30/01 (SWEC);
   c. Calc. No. 05996.02–G(B)-13, Rev. 4, *Stability Analysis of the Canister Transfer Building Supported on a Mat Foundation*, 3/30/01 (SWEC); and
   d. Errata to Correct SAR Chapter 2 of the PFSF License Application # 22;
   e. Calc. No. 05996.02F(PO18)-3, Rev. 1, *Development of Time Histories for 2000-year return period design spectra*, 3/21/01(Geomatrix);
   f. *Fault Evaluation Study and Seismic Hazard Assessment*, Rev. 1, March 2001 (Geomatrix);
   g. *Development of Design Basis Ground Motions for the Private Fuel Storage Facility*, Rev. 1, March 2001 (Geomatrix); and
   h. *Update of Deterministic Ground Motion Assessments*, Rev. 1, April 2001 (Geomatrix).

9. The Applicant has not presented the results of site-specific testing of the soil cement

mixture to determine its properties, reaction with native soils, constructability, and long-term performance. Rather the Applicant relies on studies for other areas and for differing applications.

10. The cited uses of soil cement at other nuclear sites, as presented by the Applicant in the SAR, are not relevant to the PFS site. *See* SAR at 2.6-109 to 2.6-110. The cited uses include slope protection for a cooling tower reservoir and liquefaction stabilization for a foundation. These are not the intended uses of the soil cement at the PFS site by the Applicant. The Applicant has not demonstrated precedence for use of soil cement to resist dynamic sliding of a safety-related nuclear facility.

11. The Applicant presumes that the soil cement strategy will provide the requisite resistance to sliding (SAR at. 2.6-108 to 2.6-110) and delays the submission of necessary design tests, documentation, and analysis to "the final design stage" (SAR at 2.6-108). This is unacceptable for two reasons: (1) Before one completes final design, one must demonstrate that the design concept will perform its intended function. This is known as <u>proof of concept</u>. (2) Once proof of concept, or conceptual design has been completed, preliminary design should be carried out with enough information to allow for independent review, verifying, and checking. Neither of these steps has been completed by the Applicant regarding the application of soil cement at the PFS site.

12. The concept of using soil cement as a "dynamic buttress" around the perimeter of the Canister Transfer Building ("CTB") and storage pads is now an integral part of the foundation design to resist the newly revised design basis earthquake ground motion (SWEC Calculation No. 05996.02-G(B)-13, Rev. 4 *Stability Analysis of Canister Transfer Building*, SWEC Calculation No. 05996.02-G(B)-4, Rev. 7 *Stability Analysis of Storage Pads*). In order to develop the required resistance to sliding, these calculations assume a minimum unconfined compressive strength of 250 psi around the CTB and 340 psi around the storage pads will be available within the soil cement to develop passive earth pressure resistance against horizontal sliding. Without this resistance, the Applicant's simplified calculations suggest that the CTB and storage pads will not have sufficient resistance to dynamic sliding under the newly revised strong ground motions. There are several issues regarding these simplified sliding analyses. The most important are concerned with: (1) the over-simplistic nature of the calculations, which does not consider key design elements and failure mechanisms, and (2) the presumption that the soil cement will behave as an integral unit with the CTB mat and storage pad foundations, without suffering any cracking or damage from seismic and environmental factors.

13. For the dynamic stability calculations (SWEC Calculation No. 05996.02-G(B)-13, Rev. 4, *Stability Analysis of Canister Transfer Building*, SWEC Calculation 05996.02-G(B)-4, Rev. 7, *Stability Analysis of Storage Pads*), the Applicant has assumed the

concrete foundations and soil-cement buttress will behave as rigid-bodies and that the soil-cement block will only be placed in compression by strictly horizontal sliding that is in-phase with the mat foundation. However, rigid body behavior with in-phase, translational motion is not realistic. A relatively thin (3 to 5-foot deep) veneer of soil cement having extremely large areal dimensions (*i.e.*, significantly exceeding the footprint dimensions of the CTB and pad emplacement area) will have to resist a variety of earthquake wave forms, each with varying inclination, wavelength and phasing. The stresses resulting from these earthquakes may simply crack the soil-cement buttress. If cracking occurs as a result of a variety of seismic and non-seismic mechanisms (*see* discussion below), these cracks may become preferential slip planes for formation of a passive failure wedge during the seismic event. Thus, the assumed passive resistance used in the redesign of the CTB and storage pad foundation systems (SWEC Calculation No. 05996.02-G(B)-13, Rev. 4, *Stability Analysis of Canister Transfer Building*; SWEC Calculation 05996.02-G(B)-4, Rev. 7, *Stability Analysis of Storage Pads*) may not be available to resist earthquake forces.

14. The potential mechanisms for cracking of the soil cement include, but are not restricted to: (1) overstressing and cracking due to dynamic bending, torsional, beam shear and compressional stresses, (2) overstressing and cracking at the soil-cement / foundation interface due to kinematic interaction, (3) delamination or debonding along a soil cement lift interface, (4) shrinkage cracking due to drying and curing, (5) frost penetration and expansion cracking, (6) cracking or overstressing due to vehicle loads (*e.g.*, canister transport vehicle), and (7) long-term desiccation cracking and other environmental effects acting on soil cement during a 40 year service period.

15. <u>Overstressing and cracking due to dynamic bending, torsional and beam stresses</u> There will be a variety of waveforms that will cause complex stresses to form in the foundation system (*i.e.*, concrete foundation, soil-cement, and soil). These include: primary or compressional waves, shear waves, and surface waves (*e.g.*, Love and Rayleigh waves). The interaction and phasing of these waves will cause the soil-cement buttress and CTB and pad foundation systems to experience bending, torsion, rocking, and out-of-phase battering. This complex interaction of the soil cement with the foundation system and the underlying untreated soils with the seismic waves will introduce significant bending, torsional and beam-shear and compressional stresses within the soil cement. The magnitude of these stresses and their effect on the foundations, soil-cement, and soil have not been considered by the Applicant.

16. <u>Overstressing and cracking at the soil-cement / foundation interface due to kinematic interaction</u>: The interface between the soil cement and the concrete foundations will have marked differences in stiffness in the horizontal direction. These stiffness differences will cause kinematic interaction (high stress and strain concentrations) during the vibratory ground motion at the soil cement / foundation

interface. The Applicant has not considered the magnitude and consequences of these stresses to the soil cement and its ability to bond with the concrete foundations.

17. Delamination or debonding along a soil cement lift interface: Because soil cement is constructed in lifts, preferential planes of weakness form along these planes and the layers become debonded unless care is taken to prepare properly the interface before the placement of the next lift. This problem has been noted by the Bureau of Reclamation in its inspections of soil cement facings used on dams.

> Inspections of these facings after a period of service, however, have revealed that the bond between lifts seems to the weak point of the facing. In addition, the results of coring on these facings indicate that the bond is weaker than the remainder of the soil cement. A closeup view of the facing at Merritt Dam in figure 2 [Figure 1 this text] shows that the layers are not bonded, at least at the outside edge. Since these layers are not bonded to any great extent, they may be considered to be a series of nearly horizontal slabs on the slope of the dam.

DeGroot, G., *Bonding Study on Layered Soil Cement*, REC-ERC-76-16, US Bureau of Reclamation, 1976.

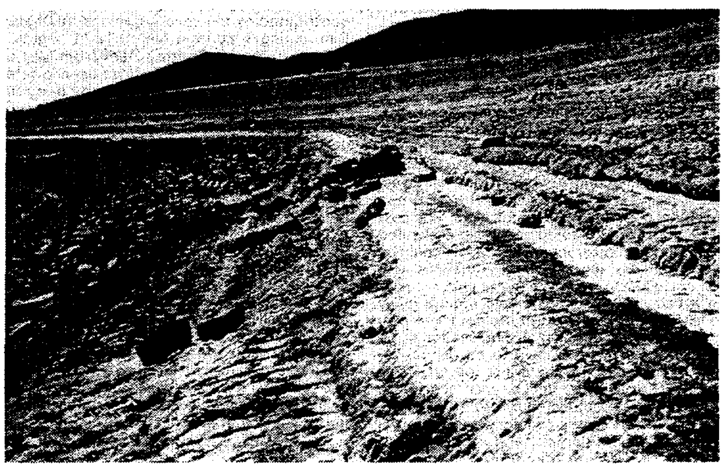

Figure 1. View of Merritt Dam soil-cement facing, Sept. 1968, 6 years after placement. Photo P637-D-66530 (after De Groot, 1976.)

Potential debonding of the soil cement, either prior to the earthquake or from beam shear stress resulting from bending during the earthquake, will significantly reduce the capacity of the soil cement buttress to resist dynamic sliding.

18. Shrinkage cracking due to drying and curing: Shrinkage cracking of soil-cement is well documented in the geotechnical and transportation literature (Figure 2). The amount of shrinkage increases with the amount of cement added to the soil.

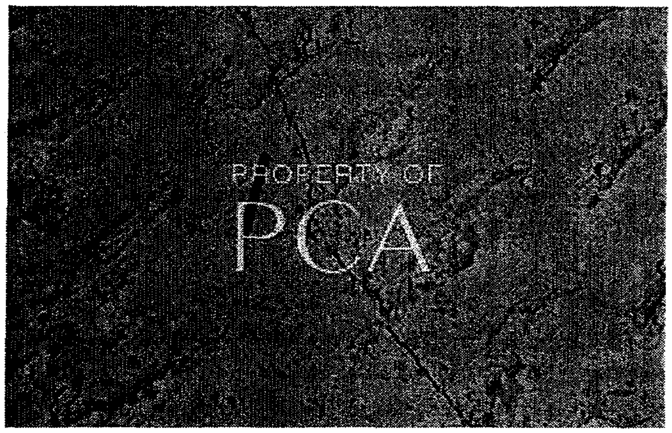

Figure 2. Example of shrinkage crack (Portland Cement Association).

The Texas Department of Transportation ("DOT") has considerable experience and performance data regarding cement stabilized bases and subbases used in highway construction. Because cement stabilized bases are prone to cracking, the use of cement has largely been replaced with lime and lime/fly ash blends to help reduce shrinkage cracking that occurred in cement stabilized bases.

> The Atlanta District has many years of experience in base stabilization. This area of Texas has relatively poor base material, most of which is iron-ore gravel. Stabilization is used in some new construction, but it is used extensively in pavement rehabilitation where a need exists to improve an existing base. In the 1960's the District started using cement as the stabilizing agent. However, thermal and shrinkage cracking resulted, and the sections became maintenance headaches. Since then the use of cement has largely been replaced with lime and lime/fly ash blends.

Little, D. N., Scullion, T., Kota, B.V.S.P., and Bhuiyan, J. (1994); *Identification of the Structural Benefits of Base and Subgrade Stabilization*, Texas Transportation Institute Research Report 1287-2, in cooperation with the Federal Highways Administration and the Texas Department of Transportation; pp. 3.25.

In its redesign for the newly revised probabilistic seismic hazard design basis ground motions, PFS is now proposing a soil cement mixture of about 8.5 percent for the pad emplacement area adjacent to the pad foundation and 6.5 percent for the

Canister Transfer Building (SWEC Calculation No. 05996.02-G(B)-13, Rev. 4, *Stability Analysis of Canister Transfer Building*; SWEC Calculation 05996.02-G(B)-4, Rev. 7, *Stability Analysis of Storage Pads*). These percentages of cement are generally higher than those used in Texas and similar shrinkage cracking may result at the PFS site. The uncertainty regarding the shrinkage behavior of the proposed cement treated soils offers good evidence for why it is necessary to obtain data and performance from site specific tests.

If shrinkage cracks do develop, they can be detrimental to the load carrying capacity of the treated soil. Kota et al. (1995) report investigations regarding the performance of heavily stabilized bases in Houston, Texas. Seven sections of stabilized bases were evaluated, all which had stabilizer contents between 5 to 6 percent and thickness of one foot. Six of the seven sections had transverse shrinkage cracks after only 3 to 7 years of service. Their observations and evaluations led to the following conclusions (Kota, B.V.S.P., Scullion, T., and Little D. (1995); *Investigation of Performance of Heavily Stabilized Bases in Houston, Texas, District*, Transportation Research Board Record No. 1486, Transportation Research Board, National Academy Press, Washington, D. C., p. 75):

> Transverse shrinkage cracks with widths greater than 2.5 mm significantly affect pavement performance. Minimum and typical load transfer efficiencies of 35 and 55 percent were noted for these wide cracks.
>
> It is important to understand the limitation of the current mechanistic design procedures for stabilized bases because they may not always result in best performing designs. Performance of the stabilized pavements can be improved by additional considerations that lead to the reduction of the formation of wide shrinkage cracks.

If significant shrinkage cracking in the cement treated soils occurs at the PFS site, it will reduce the passive earth pressure and tensile strength available to resist seismic forces. The Applicant has not addressed these concerns in its redesign of the soil cement. Furthermore, the Applicant proposes to cover the pad emplacement area with gravel so that it will be essentially impossible to observe and assess the degree and nature of cracking with time.

19. Frost penetration and expansion cracking: The soil cement will be within the zone of typical frost penetration, which is about 2 to 3 feet for typical soils in the intermountain west. Frost penetration and expansion will cause additional widening and deepening of shrinkage cracks. The Applicant has made no provision to protect the soil cement buttress from potential frost damage.

20. Cracking due to vehicle loads: The canister transport vehicle will be active in the pad emplacement area. No assessment has been made of the soil cement layer structural capability to resist wheel loading without fatigue damage.

21. Long-term Performance of Soil Cement: The soil cement must remain in service for a 40 year design life without appreciable degradation of properties and cracking. The Texas Department of Transportation has raised significant concerns regarding the longevity of stabilized soils.

> Currently, treated or stabilized subgrades are not considered in the pavement design process. No increased strength is assigned to these layers, and no reduction in overall thickness is recommended. This is because the long term benefits of stabilization have not been sufficiently documented. A concern has been that the chemical subgrade stabilization may not be permanent, and any support or confinement benefits will disappear after a few years.

Little et al., 1994. This statement highlights concerns regarding the long-term performance of cement, lime, and lime/fly ash stabilized subgrades. As a result of these concerns, Texas DOT initiated research to further investigate the characteristics and performance of various stabilizing mixes (Little et al., 1994). Similar studies and research have not been performed by the Applicant for the PFS soils. As a result, great uncertainty exists regarding the long-term performance of this soil cement treatment proposed at the PFS site and its ability to resist earthquake forces for a 40-year service period.

22. The Applicant has proposed to construct a 1-foot deep shear key around the perimeter of the CTB to permit use of the full cohesive strength of the in situ silty clay / clayey silt in resisting sliding due to loads from the design basis ground motion. However, for a foundation that is 240 feet wide by 279.5 feet long, such a shallow shear key will not be effective in forcing the shear failure into the clay layer. There is still potential for sliding along the bottom of the mat at the top of the soil interface. For this case, the adhesion of soil placed on concrete should be used. Adhesion values of concrete placed on an overconsolidated clay may be significantly less than the cohesive strength of the soil. The Applicant should justify why it has used a less conservative value. The Applicant should also explain what steps will be taken in construction to guarantee that there is not significant remolding or moisture content changes in the silty clay / clayey silt. Such changes could significantly impact adhesion values and reduce the sliding stability of the foundation.

23. The newly revised design basis ground motions are significantly higher (about 35 percent). In the Applicant's design, the Bonneville silty clay / clay silt layer is now encapsulated between two much stiffer layers. It is underlain by a denser, stiffer

granular layer and overlain by cement treated soil. This soft layer encapsulated within stiffer layers will cause significant dynamic stress and strain concentration within the softer layer. These strains will mobilize a significant portion of the soil's shear strength capacity as the soil must resist the free-field dynamic forces. This partial mobilization of shear strength from the free-field motions will reduce the shear strength capacity that remains to resist the inertial forces from foundation and mass of the CTB. The Applicant has not accounted for any partial mobilization of shear strength in the dynamic sliding calculations and has not demonstrated that the design shear strength is sufficiently conservative to account for this potential effect.

24. The Applicant has assumed that undrained loading will occur during the earthquake but has not account for a partial reduction in undrained shear strength due to pore pressure generation during earthquake cycling. Typically, a 10 to 20 percent reduction is applied to account for any elevated pore pressures that may exist.

25. PFS has not considered potential moisture content changes in the foundation soils with time and how these changes may affect the undrained shear strength used in design of the foundation systems. Unsaturated, fine-grained soils can derive a significant portion of their cohesion (*i.e.*, undrained shear strength) from matrix suction (*i.e.*, negative pore pressure) that forms in the soil due to partial saturation. Thus, the undrained shear strength of an unsaturated, fine-grained soil can be sensitive to changes in moisture content.

The placement of soil-cement in the pad emplacement area and around the CTB will not preclude changes in moisture content with time in the untreated native soils immediately below the CTB and pad foundations or cement treated soil. The geotechnical literature discusses cases of change in moisture content in a foundation soil, even after it has been capped by a relatively impermeable barrier. For example, Holtz and Kovacs (1981) discuss this effect:

> A common occurrence is that a pavement or building is constructed when the top soil layer is relatively dry. The structure covering the soil prevents further evaporation from occurring and the soils increase in water content due to capillarity; then the soil may swell.

Holtz, Robert D. and Kovacs, William D., *An Introduction to Geotechnical Engineering*, 1981, Prentice-Hall Inc., Englewood Cliffs, New Jersey.

While I do not believe that these soils will swell significantly, however, an increase in soil moisture content resulting from capillarity is still possible. Significant increases in the moisture content of the surficial clayey soils at the PFS site may reduce the soil's undrained shear strength and hence the soil's capacity to resist dynamic sliding at the foundations and/or soil cement interface. PFS has not assessed how potential

moisture content changes and the subsequent shear strength changes may impact the seismic design of the foundations.

Executed this 16th day of May 2001.

Steven Bartlett

By: ______________________________
Steven F. Bartlett, Ph.D., P.E.

Areas of Emphasis

Geotechnical Engineering
Earthquake Engineering
Transportation Engineering
Geoenvironmental Engineering
Applied Statistics
Project Management

Education

Ph.D., Civil Engineering (geotechnical emphasis), Brigham Young University, 1992.

B.S., Geology, Brigham Young University, 1983.

Professional History

Assistant Professor, Civil and Environmental Engineering Department, University of Utah, 2000-current.

Utah Dept. of Transportation, Research Project Manager, Research Division, 1998 - 2000.

Woodward-Clyde Consultants, Project Engineer, 1996-1998.

Westinghouse Savannah River Company, Senior Engineer, 1991-1995.

Brigham Young University, Research Assistant, 1988-1991.

Utah Department of Transportation, Preconstruction Materials Engineer, 1987-1988.

Utah Department of Transportation, Construction Technician, 1984-1987.

Geokinetics In-Situ Oil Shale Development, Retort Engineer and Technical Writer, 1984.

Awards and Recognitions

BYU Presidential Scholar (University Scholarship).

Alvin Barrett Scholar (Geology Department).

Professional Experience

- **Assistant Professor, Civil and Environmental Engineering** - Teaching of graduate and undergraduate courses in geotechnical engineering and performing research.
- **Research Project Manager, I-15 Reconstruction Testbed Projects** - UDOT Project manager for I-15 research involving construction and instrumentation of innovative embankment systems, foundation treatments and ground modification; long-term settlement monitoring and performance of embankments, mechanically stabilized earth walls, geofoam fills, etc.; response of pile and geopier foundation systems to lateral and uplift loads; carbon fiber retrofitting and non-destructive testing of bridges.
- **I-15 Design-Build Team** - Lead geotechnical design engineer for Woodward-Clyde Consultants responsible for geotechnical design from 800 South to 2100 South of I-15 in Salt Lake City, Utah. Design included foundation treatments, ground modification, slope stability, settlement considerations, geofoam fills, liquefaction assessments, and seismic modeling of embankment and MSE wall systems.
- **Value Engineering and Design Team** - Geotechnical member of the Value Engineering and Concept Design Team for the University Parkway Interchange (1300 South) at I-15, Orem, Utah.
- **Private Fuels Storage Facility** - Member and geotechnical expert witness for the State of Utah panel opposing the proposed interim high-level radioactive waste storage facility. Lead reviewer of the safety analysis report (SAR) and supporting calculations for geotechnical investigations, Skull Valley, Utah.
- **Kennecott Utah Copper Tailing Impoundment Modernization Project** - Performed steady state and transient seepage analyses for dewatering system for the upgrade and expansion of Kennecott's tailings impoundment, Magna, Utah.
- **Wasatch County Water Efficient Project** - Performed geologic and geotechnical assessments of canal stability and pump station

Civil Engineering Department Scholar.

BYU Scientific Research Society (Sigma-Chi) Recipient, Outstanding Ph.D. Dissertation, 1992.

Total Quality Achievement Award, Environmental Restoration Department, Westinghouse Savannah River Company, 1992, 1993.

Finalist for outstanding paper, ASCE Journal Geotechnical Engineering, 1995.

Vice President's Award, Westinghouse Engineering and Construction Services Division, 1995.

Excellence in Research Award, Utah Dept. of Transportation, 1999.

Registrations

Professional Engineer: Utah.

Affiliations

American Society of Civil Engineers.

High Level Waste Opposition Panel for the Private Fuels Storage Facility, Tooele County, Utah.

Boy Scouts of America.

Committees

Member of Transportation Research Board, Committee on Soils and Rock Instrumentation, 2000-current.

Utah Seismic Safety Commission Lifelines Subcommittee, 1998-current.

FEMA Project Impact and Salt Lake City Seismic Hazard Ordinance Committee, 2000-current.

Organizing Committee, 34th Annual Symposium on Engineering Geology and Geotechnical Engineering, Logan, Utah, April, 1999.

Organizing Committee, Environmental Geotechnology, ASCE, Salt Lake City, Utah, March, 1997.

Municipal Landfill Site Selection Committee, Columbia County, Georgia, 1993.

locations, Heber Valley, Utah.

- **Bear River Pipeline** - Performed geologic and geotechnical assessments of pipeline route alternatives for the Salt Lake Water Conservancy District, Weber, Davis and Salt Lake Counties, Utah.
- **Cainville Dam Investigation** - Project Engineer responsible for preliminary geologic and geotechnical assessments of foundation conditions at this proposed dam site. Performed drilling of abutment areas, pump testing, and seepage assessments, Wayne County, Utah.
- **DMAD and Gunnison Bend Dam Investigations** - Performed geotechnical investigations and assessments to determine the piping potential and seismic stability of these embankment dams for the State of Utah, Dam Safety Program, Delta, Utah.
- **Seismic Retrofit of Salt Lake City Waste Water Treatment Plant** - Lead geotechnical design engineer and field oversight engineer of jet grouting operations to stabilize potentially liquefiable soils under an effluent pump station, North Salt Lake City, Utah.
- **Hurricane Bridge Foundation Investigation** - Performed geologic and bridge foundation investigations and analyses for UDOT, Hurricane Bridge Crossing, Hurricane, Utah.
- **ITP/H-Area Tank Farm Geotechnical Investigation and Seismic Qualification, Department of Energy, Savannah River Site** - Westinghouse's principal geotechnical investigator on a multi-disciplinary team overseeing the seismic qualification of the ITP/H-Area high-level radioactive waste storage tank farm. This project included extensive subsurface investigations, strong ground motion modeling, probabilistic liquefaction hazard evaluations, dynamic settlement and slope stability calculations, and risk assessment.
- **Review Team for the Seismic Design of the Defense Waste Processing Facility, Department of Energy Savannah River Site** - Westinghouse's principal geotechnical investigator reviewing the Safety Analysis Report

Training and Certifications

OSHA 1910.120 Health and Safety Training for Hazardous Waste Operations and Emergency Response.

Department of Energy, Radiation Worker Training, Westinghouse Savannah River Company.

U.S. Department of Labor Mine Safety and Health Administration (MSHA) Underground Mining Training.

Peer Reviewed Publications and Reports

Youd, T.L., Hansen, C.M., Bartlett S.F., 2001, "Revised MLR Equations for Prediction of Lateral Spread Displacement," Journal of Geotechnical (in press).

Bartlett, S. F., Negussey, D., Kimball, M., 1999, "Design and Use of Geofoam on the I-15 Reconstruction Project," Transportation Research Board Annual Meeting, Jan. 2000, Washington, D.C.

Bartlett, S. F. and Youd, T. L., April 1995, "Empirical Prediction of Liquefaction-Induced Lateral Spread," Journal of Geotechnical Engineering, ASCE.

Bartlett, S. F., 1992, "Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads," Ph.D. dissertation and report published by National Earthquake Engineering Research Center, NCEER Report #92-0021.

Bartlett, S. F. and Youd, T. L., 1992, "Case Histories of Lateral Spreads from the 1964 Alaska Earthquake," NCEER Report #92-0002.

Other Publications and Reports

Saye, S. R., Esrig, M. I., Williams, J. L., Pilz J., Bartlett S.F., "Lime Cement Columns for the Reconstruction of Interstate 15 in Salt Lake City, Utah." ASCE Geo-Odessey, Blacksburg, VA., June 10 - 13th, 2001.

Bartlett, S. F., 1999, "Research Initiatives for Monitoring Long Term Performance of I-15 Embankments, Salt Lake City, Utah," 34th Annual Symposium on Engineering Geology and Geotechnical Engineering, Logan, Utah, April, 1999.

Youd, T. L., Hansen C. M., Bartlett, S. F., 1999,

(SAR) for the seismic qualification and start-up of this high-level radioactive waste vitrification and storage facility, Savannah River Site, Aiken, South Carolina.

- **Department of Energy Savannah River Site Hazardous Waste Landfill Closure** -Project manager and lead design engineer for the RCRA Facility Investigation and closure of a 51-acre hazardous waste landfill. Also, oversaw the preparation of CERCLA feasibility study for the same closure, Savannah River Site, Aiken South Carolina.
- **RCRA/CERCLA Investigations** - Project Manager for hazardous waste investigations at the Bingham Pump Outage Pits, Burma Road Rubble Pits, and H-Area Retention Ponds, Savannah River Site, Aiken, South Carolina.
- **UDOT Region 2 Preconstruction Materials Engineer** - Performed material testing and pavement design for highway alignment and urban interchanges in West Valley City and the I-215 interchange at California Avenue. Evaluated compaction and quality of subgrade for east-side I-215 between 2700 South and 4500 South. Conducted geologic investigations on new and existing highway alignments in Salt Lake and Wasatch Counties, located fill and gravel sources for construction. Instrumented and monitored I-215 fill slopes for settlement and slope stability.
- **Construction/Survey Technician** - Survey of highway projects and construction inspection. Development of construction project accounting system for UDOT.
- **Retort Engineer** - Monitored process control of underground retorting of oil shale for Geokinetics under Syn-Fuels research contracts for the Department of Energy, Vernal, Utah.

Research Experience

- **I-15 Long Term Monitoring of Embankments and Innovative Foundation Treatments** - Principal Investigator, Utah Department of Transportation (1998 - 2008).
- **Deformation and Modeling of MSE Wall**

"Improved MLR Model for Predicting Lateral Spread Displacement," 7th US-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, Seattle, Washington, August, 1999.

Simon, D. B., Shlemon, R .J., and Bartlett, S.F., 1999, "Holocene Ground Failure in Downtown Salt Lake City, Utah," Geological Society of America, Cordilleran Section, Vol. 31, Number 6, Berkeley, California, May 1999.

WSRC, 1995, "In-Tank Precipitation Facility (ITP) and H-Tank Farm (HTF) Geotechnical Report," Report No. WSRC-TR-95-0057, Rev. 0, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., 1995, "Probabilistic Liquefaction Settlement Evaluation for the In-Tank Precipitation Facility (ITP)," Report No. C-CLC-H-00815, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., 1995, "Geotechnical Seismic Assessment Report for the Defense Waste Processing Facility (DWPF)," Report No. SRC-TR-95-0072, Westinghouse Savannah River Company, Aiken, S.C.

Rouhani, S., Lin, Y. P., and Bartlett, S. F., 1995, "H-Area/ITP Geostatistical Assessment of In-Situ and Engineering Properties," Final Technical Report, ERDA Project No. 93044, Site Geotechnical Services, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., 1994, "Determination of Soft Zones and Consolidation Properties for the Santee Formation," Report No. K-CLC-H-00058, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F. and Youd, T. L., 1993, "Prediction of Liquefaction-Induced Ground Displacement Near Bridges," Proceedings from the U.S. National Earthquake Conference, Memphis, Tenn., May, 1993.

Bartlett, S. F., 1993, "RCRA Facility Investigation / CERCLA Remedial Investigation for the Burma Road Rubble Pit," Environmental Restoration Department, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., McMullin, S. R., and Serrato, M., 1993, "State of the Art Design: A Closure System for the Largest Hazardous Waste Landfill at the Savannah River Site," Proceedings of Waste Management '93 Symposium.

Behavior - Co-Principal Investigator, Utah Department of Transportation and Utah State University (1999-2000).

- **Evaluation of Properties and Long-Term Performance of Geofoam Fills** - Co-Principal Investigator, Utah Department of Transportation and Syracuse University (1998-2000).
- **Geostatistical Assessment of In-Situ and Engineering Properties at H-Tank Farm** - Co-Principal Investigator, Westinghouse Savannah River Company and Georgia Institute of Technology (1994 - 1995).
- **Evaluation of Geopiers and Pile Foundation to Lateral and Uplift Loads** - Project Manager, Utah Department of Transportation, University of Utah, and Brigham Young University (1999).
- **Design, Application, and Use of Carbon-Fiber Composites in Bridge Repair and Seismic Retrofitting** - Project Manager, Utah Department of Transportation and University of Utah (1998-2000).
- **Use of Forced Vibration Testing to Assess Bridge Damage** - Project Manager, Utah Department of Transportation and Utah State University (1998).
- **Identification and Ranking of UDOT Lifelines** - Project Manager, Utah Department of Transportation (1998 - 2000).
- **Wick Drain Performance** - Project Manager, Utah Department Transportation (1998 - 1999).
- **Assessment of Dynamic Soil Properties for the Savannah River Site** - Geotechnical Reviewer, Westinghouse Savannah River Company and University of Texas at Austin.

- **Research Assistant, Brigham Young University**, "Empirical Prediction of Liquefaction-Induced Lateral Spread," U.S. Army Corps of Engineers and National Center for Earthquake Engineering Research (1988-1991).
- **Thesis Committee Member**, Kiehl, S.J. "Distribution of Ground Displacements and Strains Induced by Lateral Spread During the

Bartlett, S. F. and Youd, T. L., 1992, "Empirical Prediction of Lateral Spread Displacement," Proceedings of 4th Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, May, 1992.

Bartlett, S. F. and Youd, T. L., 1990, "Evaluation of Ground Failure Displacement Associated with Soil Liquefaction: Compilation of Case Histories," Miscellaneous Paper S-73-1, U.S. Army Corps of Engineers.

Invited Lectures

"Instrumentation and Research of Geofoam Embankments for the I-15 Reconstruction," Huntsman Chemical Geofoam Seminar"May 16th, 2000, Salt Lake City, Utah

"Design of Geofoam Embankment for the I-15 Reconstruction," Conference on Application and Design of Expanded Polystrene, Sponsored by Taiwan Area National Expressway Engineering Bureau and China Engineering Consultants, Inc., March 3rd, 2000, Taipei, Taiwan.

"Issues Related to the Seismic Design of I-15 Reconstruction Project - A Geotechnical Perspective," Association of Engineering Geologist 42nd Annual Meetings, Sept. 28, 1999, Salt Lake City, Utah.

"Assessment of the Hazard Potential for the East Side of I-80," Conference on the Sesimic Retrofit of Utah's Highway Bridges, sponsored by the Utah Department of Transportation, January 20-22, 1999. Salt Lake City, Utah.

"Geofoam Design, Construction and Research on the I-15 Corridor Reconstruction Project," Annual Meeting of the Society of the Plastics Industry, Inc., April 23 and 24, 1998, New Orleans, La.

1964 Niigata Earthquake, Brigham Young University (1996).

- **Thesis Committee Member,** Hansen C. M, "Improved MLR Model for Predicting Lateral Spread Displacement, Brigham Young University (1999).

Teaching Experience

- **Assistant Professor**, University of Utah, Fall 2000 to current.
- **Teaching Assistant**, Earthquake Engineering, Brigham Young University, Winter Semester, 1989.
- **Teaching Assistant**, Soil Mechanics, Brigham Young University, Fall Semester, 1989.
- **Teaching Assistant**, Field and Laboratory Testing of Soil, Brigham Young University, Spring Term, 1989.
- **Missionary** - Church of Jesus Christ of Latter-Day Saints, Catania, Italy, 1979 - 1981.

Graduate Course Taught

- CVEEN 7330 Geotechnical Earthquake Engineering (1 time)

Undergraduate Course Taught

- CVEEN 3310 Geotechnical Engineering I (1 time)
- CVEEN 3320 Geotechnical Engineering II (1 time)

NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of:	)	Docket No. 72-22-ISFSI
	)	
PRIVATE FUEL STORAGE, LLC	)	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel	)	
Storage Installation)	)	May 15, 2001

DECLARATION OF DR. JAMES K. MITCHELL

I, Dr. James K. Mitchell, hereby declare under penalty of perjury and pursuant to 28 U.S.C. § 1746, that:

1. I hold a Sc.D. in civil engineering earned in 1956 from the Massachusetts Institute of Technology. Presently I am a Professor Emeritus and individual consultant on geotechnical problems and earthwork projects of many types, particularly soil stabilization, ground improvement for seismic risk mitigation, earthwork construction, and environmental geotechnology, to numerous national and international governmental and private organizations. My curriculum vitae listing my qualifications, experience, and training is attached hereto.

2. I have more than 40 years experience in the field of geotechnical engineering. I was on the faculty of the University of California, Berkeley, Department of Civil Engineering for more than 35 years, serving as Department Chair for five years. I developed and taught graduate courses in soil behavior, soil and site improvement, and foundation engineering as part of the Geotechnical Engineering Program within the Civil Engineering Department. At the same time, I was Research Engineer in the Institute of Transportation Studies and in the Earthquake Engineering Research Center. Since 1994, I served on the faculty of Virginia Tech, Via Department of Civil and Environmental Engineering, and was appointed University Distinguished Professor in 1996 and University Distinguished Professor, Emeritus, in 1999.

3. My primary research activities focused on experimental and analytical studies of soil behavior related to geotechnical problems, admixture stabilization of soils, soil improvement and ground reinforcement, physico-chemical phenomena in soils, the stress-strain time behavior of soils, in-situ measurement of soil properties, and mitigation of ground failure risk during earthquakes. I have

authored more than 350 publications, including two editions of the graduate level text and reference, "Fundamentals of Soil Behavior," and several state-of-the-art papers and guidance documents on soil stabilization, ground improvement, and earth reinforcement.

4. Some of my recent and currently active projects include the evaluation of seismic stabilities and design of liquefaction mitigation options for Success Dam in California (U.S. Army Corps of Engineers) and Pineview and Deer Creek Dams in Utah (U.S. Bureau of Reclamation); ground improvement aspects of the Port of Oakland Wharf and Embankment Strengthening Program (Harding Lawson Associates); ground improvement and fill stabilization for the proposed San Francisco Airport Expansion (Fugro West); and design review – ground improvement for the I-95/Rt.1 Interchange section of the Woodrow Wilson Bridge replacement project (Haley & Aldrich, Virginia Geotechnical Services, URS, HNTB).

5. I am licensed as a Civil Engineer and as a Geotechnical Engineer in California, and as a Professional Engineer in Virginia. I am a Fellow and Honorary Member of the American Society of Civil Engineers, and have served as an officer of the Geotechnical Engineering Division of ASCE; the United States National Committee for the International Society for Soil Mechanics and Foundation Engineering; the ASCE Committee on Soil Properties, the Committee on Placement and Improvement of Soils; the San Francisco Section of ASCE and the California State Council of ASCE; the Transportation Research Board Committee on Physico-Chemical Phenomena in Soils; the Geotechnical Board of the U.S. National Research Council; the International Society for Soil Mechanics and Foundation Engineering; and I recently completed service as Vice Chair of a NRC study committee for development of science needs for remediation of contaminated Department of Energy weapons sites. I am presently a member of a NRC study committee to advise the Department of Energy on Remediation Science and Technology for the Hanford Site.

6. I provide this declaration in support of the State of Utah's Request for Admission of Contention Utah QQ and, if admitted, I am prepared to offer testimony consistent with this declaration.

7. I have reviewed portions of the Applicant's Safety Analysis Report, relating to its geotechnical investigation of the proposed site, and relevant calculations, reports, and other documents prepared by the Applicant or its contractors and submitted to the NRC.

8. Specific to Utah QQ, I have reviewed the relevant sections the of PFS's License Amendment No. 22 and documents relating to License Amendment No. 22, including:

   a. Calc. No. 05996.02-G(B)-04, Rev. 7, *Stability Analysis of Storage Pads*, 3/30/01 (Stone & Webster ("SWEC")); and
   b. Calc. No. 05996.02–G(B)-13, Rev. 4, *Stability Analysis of the Canister Transfer Building Supported on a Mat Foundation*, 3/30/01 (SWEC).

9. SWEC Calculation No. 05996.02-G(B)-13, Revision 4, *Stability Analysis of Canister Transfer Building* and SWEC Calculation 05996.02-G(B)-4, Revision 7, *Stability Analysis of Storage Pads* consider sliding of a rigid foundation within a cement treated soil. However, it does not appear realistic to treat the CTB and the soil-cement mat essentially as rigid blocks when they are such large structures that extend over such large plan areas. It does not appear that the CTB will slide as a uniform unit. Instead, one would expect that there would be phase differences in inertial loading between one part of the building and another. The same argument applies to the cask storage pads in the north-south (*i.e.*, longitudinal) direction. For flexible foundations such as these that will experience high levels of ground motion proposed by the revised PSHA analysis, a dynamic soil-structure interaction analysis is more appropriate. In such an analysis, the differences in the deformation characteristics of the ground and foundations would be taken into account.

10. SWEC Calculation No. 05996.02-G(B)-13, Revision 4, *Stability Analysis of Canister Transfer Building*, calculates a factor of safety for overturning for the CTB as if it were a rigid body. However, such an overturning mechanism for the CTB does not appear plausible, due to the large size and flexibility of the structure.

11. The cement-treated soil will be subjected to tensile stresses from static loading, from freeze-thaw and wet-dry, from shrinkage, and from dynamic loading. The tensile strength of cement-treated soil is typically only about a fifth to a third of the unconfined compressive strength; so even rather low tensile stresses can cause cracking. The dynamic stability of the cask storage pads may be the most critical from this aspect, especially when they are loaded eccentrically (the 2 or 4 cask case) (SWEC Calculation 05996.02-G(B)-4, Revision 7, *Stability Analysis of Storage Pads*). The cask storage pads in the north-south direction are much like pavement structures; that is, they are long and thin relative to their width, and they are constructed in layers. In heavy-duty pavements with soil-cement bases, however, there is usually a structural asphalt concrete or Portland Cement

Concrete pavement layer of 8 to 16 inches thickness above the cement-treated soil to help resist bending stresses. This structural layer is absent in the Applicant's proposed design. The bending stresses and their consequences could be more critical than in most pavement structures given the large cask weight (356 kips) and relatively small diameter (11 ft) relative to the pad length (67 ft). Therefore, the bending stresses that develop must be resisted only by a relatively weak cement-treated soil. Because the Applicant has not calculated the magnitude of these stresses, one cannot demonstrate the seismic performance and structural adequacy of the proposed cement treatment.

12. From a strength and durability standpoint, it is important to distinguish between soil-cement and cement-treated soil. Soil-cement has a cement content that is sufficient to attain minimum durability standards as measured by American Society for Testing and Materials (ASTM) wet-dry and freeze-thaw tests. More cement is needed as the fines content in the soil to be treated increases. The strength of soil cement generally decreases as soil plasticity increases. The amounts of cement that are proposed to be added by the Applicant (between 3 to 8.5 percent) (SWEC Calculation No. 05996.02-G(B)-13, Revision 4, *Stability Analysis of Canister Transfer Building* and SWEC Calculation 05996.02-G(B)-4, Revision 7, *Stability Analysis of Storage Pads*) may not be sufficient to produce a true soil-cement. If it is not a true soil-cement, then the durability of the cement-treated soil may be an issue, because the wet-dry, freeze-thaw exposure may be significant for this site.

13. It is surprising that no site specific testing has been done to date to obtain the strength and durability properties of the cement-treated soil. For a project of this importance and sensitivity, the Applicant should obtain test data using the actual site soil and cement to aid in support of the conceptual and final design. The SAR indicates that the groundwater is of reasonably good quality, but gives no information regarding the chemistry of the surficial soils. Given the desert and playa lake environment, there could be salts and evaporites that could interfere with cement hydration. Of most concern may be the possible presence of sulfates. If sulfates are present in any abundance, then ettringite, a calcium-aluminum-sulfate compound that is very expansive when exposed to water, could form resulting in a loss of any benefit from the cement. The Applicant has not completed chemical testing for salts and sulfates on the soils at the PFS site.

14. Placing the soil-cement under the storage pads will lead to an increase in the water content of the partly saturated silty clay, clayey silt soils beneath them. This could impact the settlement, the strength, and the adhesion between the soil and the soil-cement. Also, the Applicant proposes to place the cement-treated soil directly

upon an overconsolidated silty clay, clayey silt subbase. If care is not taken during construction, the use of heavy placement equipment could cause significant remolding of the subbase soils, and such remolding could markedly affect the shear strength of the subbase at the interface with the cement treated soil.

Executed this 15th day of May 2001,

By ______________________________
James K. Mitchell, Ph.D., P.E.

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Executed this 15th day of May 2001,

By Jas K Mitchell
James K. Mitchell, Ph.D., P.E.

JAMES KENNETH MITCHELL
University Distinguished Professor, Emeritus
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Consulting Geotechnical Engineer

Dr. James K. Mitchell received his Bachelor of Civil Engineering Degree from Rensselaer Polytechnic Institute in 1951, Master of Science Degree from the Massachusetts Institute of Technology in 1953, and the Doctor of Science Degree, also from M.I.T., in 1956.

He joined the faculty of the University of California, Berkeley in 1958 and held the Edward G. Cahill and John R. Cahill Chair in the Department of Civil Engineering at the time of his retirement from Berkeley in 1993. Concurrently he was Research Engineer in the Institute of Transportation Studies and in the Earthquake Engineering Research Center. He developed and taught graduate courses in soil behavior, soil and site improvement, and foundation engineering as part of the Geotechnical Engineering Program within the Civil Engineering Department. He served as Chairman of the Department of Civil Engineering from 1979 through 1984. He was appointed the first Charles E. Via, Jr. Professor in the Via Department of Civil Engineering at Virginia Tech in 1994, University Distinguished Professor in 1996, and University Distinguished Professor, Emeritus, in 1999.

His primary research activities have focused on experimental and analytical studies of soil behavior related to geotechnical problems, admixture stabilization of soils, soil improvement and ground reinforcement, physico-chemical phenomena in soils, the stress-strain time behavior of soils, in-situ measurement of soil properties, and mitigation of ground failure risk during earthquakes. He supervised the dissertation research of 72 Ph.D. students. He has authored more than 350 publications, including two editions of the graduate level text and reference, "Fundamentals of Soil Behavior," and several state-of-the-art papers and guidance documents on soil stabilization, ground improvement, and earth reinforcement. During the 1960's and early 1970's he served as the NASA Principal Investigator for the Soil Mechanics Experiment, which was a part of Apollo Missions 14-17 to the Moon.

sites. He now is a member of a NRC study committee to advise the Department of Energy on Remediation Science and Technology for the Hanford Site. He was Vice President of the International Society for Soil Mechanics and Foundation Engineering from 1989-1994.

Dr. Mitchell was awarded the Norman Medal in 1972 and 1995, the Thomas A. Middlebrooks Award (three times), the Walter L. Huber Research Prize and the Karl Terzaghi Award, all from the American Society of Civil Engineers; the Distinguished Teaching Award and the Berkeley Citation from the University of California; the Western Electric Fund Award of the American Society for Engineering Education; the Medal for Exceptional Scientific Achievement from the National Aeronautics and Space Administration, and has been selected as the recipient of the 2001 Kevin Nash Gold Medal of the International Society for Soil Mechanics and Geotechnical Engineering. He was elected to the United States National Academy of Engineering in 1976 and to the U. S. National Academy of Sciences in 1998.

Lists of projects and publications are available on request.

April 2001

Private Fuel Storage, L.L.C.

ATTORNEY GENERAL

APR 17 2001

ENVIRONMENT

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April 16, 2001

SUMMARY OF CHANGES FOR PFSF LICENSE APPLICATION AMENDMENT #22
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: PFS letter, Parkyn to U.S. NRC, License Application Amendment #22, dated March 30, 2001

The purpose of this letter is to summarize the various changes made in Amendment #22 to the Private Fuel Storage Facility (PFSF) License Application submitted with the above referenced letter. The changes can basically be grouped into four major categories:

- Changes due to the revised design basis ground motion
- Changes due to the revised storage cask/pad spacing
- Changes to the Canister Transfer Building design
- Other miscellaneous changes

A description of each category of changes along with an explanation for the change is given below. We have also provided a list of supporting analyses and reports that were revised as a result of the changes.

REVISED DESIGN BASIS GROUND MOTION

Re-evaluation of previously collected test data for the PFSF site indicated that some of the data that had not been completely incorporated into the PFSF Fault Evaluation Study and Seismic Hazard Assessment, prepared for PFS by Geomatrix Consultants, Inc., needed to be incorporated. Specifically:

1. The seismic shear wave velocity profiles obtained during the 1999 cone penetration testing program at the site for the top 30 feet of soil were evaluated

by Geomatrix and incorporated into the calculation "Soil and Foundation Parameters for Dynamic Soil-Structure Interaction Analysis, 2000-Year Return Period Design Ground Motions." However, Geomatrix concluded at the time that these velocity profiles were consistent with the average velocity profile used in the "Fault Evaluation Study and Seismic Hazard Assessment" and that revisions to that Assessment were not required.

2. The unit weight for the soil for both the Skull Valley and generic California deep soil profiles used in the original Fault Evaluation Study and Seismic Hazard Assessment was 131 lb/ft^3. The appropriate unit weight for the soil at the PFSF varies from 80 lb/ft^3 near the surface to 115 lb/ft^3 at a depth of 26 ft. It was initially concluded that this difference in unit weight was not a significant contributor to the outcome of the Fault Evaluation Study and Seismic Hazard Assessment.

A re-evaluation of the above two items determined that the Fault Evaluation Study and Seismic Hazard Assessment needed to be revised to include these differences. When the Fault Evaluation Study and Seismic Hazard Assessment was revised to account for these differences, it predicted new Peak Ground Accelerations (PGA) of 0.711g horizontal and 0.695g vertical. This change in the design basis ground motion, in turn, necessitated that the following reports and analyses to be revised:

Geomatrix Consultants, Inc.

- Calculation No. 05996.02–G(PO18)–2, Revision 1, entitled "Soil and foundation parameters for dynamic soil-structure interaction analysis, 2000-year return period design ground motions"
- Calculation No. 05996.02–G(PO18)–3, Revision 1, entitled "Development of Time Histories for 2000-year return period design spectra"
- Fault Evaluation Study And Seismic Hazard Assessment, Revision 1, March 2001
- Development of Design Basis Ground Motions for the Private Fuel Storage Facility, Revision 1, March 2001.

Holtec International

- Multi Cask Response at the PFS ISFSI from 2000-Yr Seismic Event (Rev. 2), Holtec Report No. HI-2012640, dated March 29, 2001.

International Civil Engineering Consultants, Inc

- Storage Pad Analysis and Design, calculation number 0599602–G(PO17)–2, Revision 3, dated April 5, 2001.

Stone and Webster

- Calculation No. 05996.02–SC–4, Revision 2, entitled "Development of Soil Impedance Functions for Canister Transfer Building"

- Calculation No. 05996.02–SC-5, Revision 2, entitled "Seismic Analysis of Canister Transfer Building"
- Calculation No. 05996.02–SC-10, Revision 1, entitled "Seismic Restraints for Spent Fuel Handling Casks"
- Calculation No. 05996.02–G(B)-04, Revision 7, entitled "Stability Analysis of Cask Storage Pads"
- Calculation No. 05996.02–G(B)-13, Revision 4, entitled "Stability Analysis of the Canister Transfer Building Supported on a Mat Foundation"

The following is a list of the sections of the License Application that were updated to incorporate the results of these revised reports and analyses:

- Section 2.6.2 of the SAR was revised to incorporate results of the revised site response analyses. Section 2.6.1.12 of the SAR was updated to incorporate the results of revisions to the dynamic stability analyses of the storage pads and the Canister Transfer Building resulting from changes to the PFSF design basis ground motion. SAR Section 2.6.4.9 was updated to identify the new design basis ground motions. Changes to maintain consistency were also made to other subsections of Section 2.6 of the SAR.
- Section 2.6.1.12 of the SAR was revised to update the discussions of the results of dynamic stability analyses of the storage pads and the Canister Transfer Building resulting from changes to the PFSF site design basis ground motion.
- SAR Section 2.6.4.9 was updated to identify the new design basis ground motions.
- Changes to maintain consistency were also made to other subsections of Section 2.6 of the SAR.
- An explanation was provided in Appendix 2G that the conclusions of the Appendix had not changed, even though the design basis ground motion values were revised in this recent revision.
- Section 3.2.10.1.1 of the SAR was revised to reflect the site-specific horizontal and vertical response spectra associated with the new design basis ground motion.
- Changes to maintain consistency were also made to other subsections of Section 3.2.10 of the SAR.
- Section 4.2.1.5.1(H) of the SAR, which evaluates the structural design of the storage cask under seismic conditions, was updated to reflect the results of the HI-STORM storage cask stability analyses based on the new seismic response spectra.
- SAR Section 4.2.3.5.1 was revised to reflect the dynamic analyses of the storage pads for the new design basis ground motion.
- SAR Sections 4.7.1 and 4.7.2 were updated to incorporate changes resulting from the new seismic loads.

- SAR Section 8.2.1 was revised to reflect the new design basis ground motion and the results of the HI-STORM storage cask stability analyses based on the new seismic response spectra.
- The discussion of the stability of a loaded cask transporter under seismic conditions (Section 8.2.6.2) was updated for the new design basis ground motion.
- Section 2.6 of the PFSF Environmental Report, which includes a summary of the geotechnical and seismic information in Chapter 2 of the SAR, was updated to be consistent with the information presented in the SAR.
- Section 2.6.5 of the ER was revised to incorporate the changes made to the velocity profiles and resulting changes to the site response analyses and idealized soil profiles that were used in the soil-structure interaction analyses.
- ER Section 2.6.8 was updated to identify the new design basis ground motion.
- Changes to maintain consistency were also made to other subsections of Section 2.6 of the ER.

STORAGE CASK/PAD SPACING

In the process of preparing the specification for the PFSF storage cask transporter, it was determined that current transporter designs have become larger than those evaluated in the PFSF design (the PFSF design for the Canister Transfer Building and cask storage area had been based on a transporter used at Point Beach). The dimensions of the new generation transporters that have been designed to date for use with HI-STORM storage casks have been substantially larger than those provided by the designer/fabricator of the Point Beach transporter.

Based on extensive discussions with two transporter vendors, we concluded that the PFSF design should accommodate transporter dimensions of up to 17'-4" wide and approximately 25-ft long. With the previous 15-ft center-to-center spacing between storage casks, the clearance between the outside edge of the transporter and an adjacent cask could be as little as 3 inches, assuming worst case cask placement tolerances. Such limited clearances would make cask placement difficult and time consuming, and could create a risk of the transporter bumping an adjacent cask. Increasing the cask spacing to 16-ft would increase the most limiting clearance to 1'-3", improving operational ease in placing the casks. Therefore, it was decided to increase the length of each pad from 64-ft to 67-ft, which provides for the 16-ft center-to-center cask spacing in the pad length direction (north-south). The cask spacing in the pad width direction (east-west) remains at 15-ft. Since there are 20 pads in a column in the cask storage area, from north to south, the total additional length required to accommodate the new pad size is 20 x 3-ft = 60-ft. This 60-ft distance was accommodated by reducing the 150-ft space between the north and south pad quadrants to 90-ft, therefore not impacting the overall outer dimensions of the cask storage area or the location of the Restricted Area (RA) fence.

In discussions with the transporter vendors it was determined that the anticipated diagonal length of this larger transporter is approximately 30-ft. Since the aisles between the pads were previously 30-ft wide, the increased diagonal length could involve contact with one or both pads on either side of the aisle. Both vendors recommended a minimum aisle width of 35-ft to make the turn without interference and potential damage to a pad edge. Therefore, it was decided to increase the aisle spacing between columns of pads from 30-ft to 35-ft. Reducing the 150-ft space that previously existed between the east and west pad quadrants to 35-ft, allowed the aisle width to be increased to 35-ft between each of the 25 columns of pads with no change in the overall outer dimensions of the cask storage area or the location of the Restricted Area fence.

As a result of discussions with the cask transporter vendors, it was decided not to construct the storage pads 3.5 inches above grade to accommodate potential settling. Rather, the pads will be constructed so that their tops are level with grade. Any settling of the storage pads is now expected to be minimal due to the presence of an underlying soil cement layer, and would be addressed by scraping crushed aggregate from between pads so that the aggregate layer was flush with the top of the pads if the need arose.

The following analyses and changes to the License Application documents were required as a result of the changes in the cask/pad spacing described above:

- Technical Specification Design Feature 4.2.3 was revised to specify the new storage cask spacing requirements.
- A number of figures were revised to show the new cask/pad spacing, such as the PFSF General Arrangement drawing that appears in the SAR, ER, and EP.
- Holtec reanalyzed dose rates at the RA fence, the owner controlled area boundary, and at the nearest residence (approximately 2 miles from the PFSF) using an assumed array of 4,000 HI-STORM storage casks based on the new cask/pad spacing. Maximum doses (at the north RA fence and OCA boundary), and doses at the nearest residence, increased marginally (less than 5%) from the previous dose analysis. The results of this dose assessment are discussed primarily in SAR Section 7.3.3.5. The dose rates calculated in Holtec's dose assessment were used to reevaluate doses to construction workers in ER Section 4.1.9, and doses to wildlife postulated to spend time at the RA fence, in ER Section 4.2.9. As a result of the changes in cask/pad spacing, the following reports and analyses were revised to reflect the changes in dose rates:

    1. Holtec International "Radiation Shielding Analysis for the Private Fuel Storage Facility (Rev 2), Holtec Report HI-971645, March 16, 2001.
    2. S&W Calculation No. 05996.02–UR–5, Revision 2, entitled "Dose Rate Estimates from Storage Cask Inlet Duct Clearing Operations"

3. S&W Calculation No. 05996.02–UR(D)–8, Revision 1, entitled "Dose Rate Calculations at PFSF Locations Potentially Accessible to Wildlife and Estimates of Annual Doses to Individual Animals"
4. S&W Calculation No. 05996.02–UR(D)–11, Revision 1, entitled "Personnel Dose Rate Estimates During Construction of the Storage Pads at the Private Fuel Storage Facility"
5. S&W Calculation No. 05996.02–UR(D)–12, Revision 1, entitled "Dose Rates From the 4000 Storage Cask PFSF Array Representative of PFSF Typical Spent Fuel, Assumed to be PWR Fuel Having 35 GWd/MTU Burnup and 20 Year Cooling Time"

- Holtec reevaluated the site-specific HI-STORM storage cask thermal performance based on the revised cask/pad spacing. (Holtec International, "Additional Thermal Evaluation of the HI-STORM 100 System for Deployment at Skull Valley" Revision 1, Report HI-2002413, dated March, 2001). The results of this thermal assessment are discussed in SAR Section 4.2.1.5.2.
- Changing the length of the storage pads increased the volume of concrete associated with the pads, which impacted the quantity of imported solid construction materials (ER Table 4.1-6) as well as the water volumes drawn from the on-site well(s) during PFSF construction (ER Section 4.5.4). This change in concrete volume due to the storage pad changes had a minor impact on the traffic during PFSF construction which relates to construction noise levels evaluated in ER Section 4.1.7 and air quality, discussed in ER Section 4.1.3.
- Chapter 4 of Appendix B to the License Application, "Decommissioning Cost Estimate", was revised to address the change in storage pad dimensions, which increases the pad surface area (by less than 5%) that could potentially require decontamination.
- The volume of earthwork, discussed in ER Section 4.1.5.2, was revised to account for the new pad spacing/layout. This affected the fugitive dust emissions (Tables 4.1-4 and 4.1-5) and quantity of water required to be trucked in for soil compaction and dust control (ER Sections 4.1.7 and 4.5.4).

CANISTER TRANSFER BUILDING DESIGN

Several changes were made to the Canister Transfer Building to accommodate increased seismic ground motions and transporter changes discussed above, to increase operational efficiency, and to reduce construction effort. These changes include:

- Increasing the area of the base mat. This was done to maintain the desired factor of safety against sliding and overturning for the increased seismic loads.
- As a result of discussions with the transporter vendors it was decided that improved access to the transfer cells should be provided to avoid the 90-degree turns required with the original design to enter the transfer cells. Three additional

doors were incorporated into the West wall of the transporter aisle to make access into the transfer cells easier. It was decided that the tornado missile boundary should be moved from column line A.8 (transporter aisle west wall) to column line C (transporter aisle east wall). Since it is no longer needed as a missile barrier, the concrete wall on column line A.8 was replaced with a steel frame and metal sided wall, as was the wall on column line F (office area).

- The doors entering each transfer cell from the transporter aisle were widened from 20-ft to 22-ft to accommodate a larger cask transporter.
- The building north wall was moved 5-ft in the North direction to accommodate crane hook approach requirements
- As the result of a constructability review, the roof beams were changed from reinforced concrete to structural steel. The roof slabs were reevaluated and the thickness reduced from 1 foot in thickness to 8 inches, while still satisfying tornado missile protection criteria. These changes will reduce construction time and cost.
- The transporter aisle was increased in width by 7-ft to accommodate larger transporters.

SAR Figures 4.1-1, 4.3-1 and 4.7-1 were revised to incorporate these changes.

<u>OTHER MISCELLANEOUS CHANGES</u>

<u>RAI Incorporation</u>
PFS's responses to the NRC's Third Round EIS Request for Additional Information (RAI), NRC Letter, M. Delligatti to J. Parkyn, dated October 24, 2000 were incorporated in the licensing documents, as applicable.

- ER Chapter 7 was updated to include the results of the cost benefit analyses performed in response to the RAI. These analyses account for changes to the PFS membership and the date when it is anticipated that the PFSF will become operational (the latter part of 2003). Several revisions were also made to ER Chapter 1 as a result of these analyses.
- Information on the proposed project schedule was updated in several sections of the licensing documents (SAR Section 1.1, ER Sections 1.3 and 3.2.1, and LA Section 1.8)
- ER Section 1.2 was updated regarding the remaining fuel assembly storage capacity in the PFS member fuel pools (accounting for changes in the PFS membership), and the projected dates for loss of full-core offload capability.
- ER Figure 2.5-2 was updated to reflect the latest information in the Utah Division of Water Rights database concerning water wells within 5 miles of the PFSF site.

Soil Cement
In December 2000, 16 test pits were dug at the PFSF site in the pad emplacement area to obtain soil samples for use in the laboratory analyses necessary to design the soil cement mix. It was observed from these test pits that the depth of the eolian silt was shallower (approximately 2-ft on average rather than 3-ft) than previously believed. The earlier borings performed in this area obtained soil samples at approximate depths from grade to 2-ft and from 5-ft to 7-ft; therefore, the interface layer between the eolian silt and the silty clay/clayey silt fell between the samples collected. Our previous interpretation of the tip resistance curves (Q_t) from the near-surface cone penetration tests conservatively assumed that this boundary was where the initial spike in tip resistance bottomed out, in order to obtain an upper-bound estimate of the amount of soil cement required. This increase in tip resistance was previously interpreted as a layer of slightly cemented eolian silt. As observed in the soil cement test pits, the interface layer between the eolian silt and the silty clay/clayey silt is actually at a depth corresponding to the initial increase in tip resistance. This reduced amount of eolian silt results in the need for less soil cement under the cask storage pads (See SAR Figure 4.2-7).

Two different soil cement mixes will be required in the pad emplacement area. The soil cement to be placed above the base of the pads will have higher strength, to provide sufficient horizontal resistance to obtain a factor of safety against sliding that exceeds the criterion (FS=1.1) for dynamic loadings and to withstand environmental loads due to freeze/thaw and wet/dry cycles. The strength of the soil cement beneath the pads must be limited to satisfy the modulus of elasticity requirements of the hypothetical cask tipover analysis, but it must still provide an adequate factor of safety with respect to sliding of the pads embedded within the soil cement. Analyses indicate that designing the soil cement that will be placed under the pads to have an unconfined compressive strength that ranges from 40 psi to 100 psi will provide an adequate factor of safety against sliding and will limit the modulus of the soil cement under the pads to an acceptable level for the hypothetical cask tipover considerations.

The large extent of soil-cement in the storage pad emplacement area allows the soil-cement layer to be considered as part of the free field soil profile for the site response analyses. The properties of the soil cement, higher shear wave velocity and higher density than the existing soils in the area, help to minimize the response at the surface of the site caused by the design basis ground motion. Soil cement was added around the Canister Transfer Building foundation mat to make the free field soil profile for the building consistent with that for the storage pad emplacement area and to help resist sliding forces due to the new higher design ground motions. The soil cement extends out from the foundation mat a distance equal to one mat dimension in each direction from the foundation mat. The depth of the soil cement is 4'-4" with an 8 inch layer of crushed aggregate on top. This is discussed in SAR Section 2.6.4.11.

These changes in the use of soil cement resulted in a net increase of approximately 10% in the amount of soil cement needed for the facility.

Storage Cask Tipover and Vertical End Drop Analyses

The tipover and vertical end drop analyses documented in the HI-STORM FSAR assume a concrete thickness of 36 inches. a concrete compressive strength of 4,200 psi (at 28 days), reinforcement at the top and bottom (both directions) of the pad consisting of 60 ksi yield strength ASTM material. and a soil effective modulus of elasticity of 28,000 psi. The PFSF pads are 36 inches thick, the pad concrete compressive strength shall not exceed 4,200 psi (at 28 days), and the pad reinforcing bar is 60 ksi yield strength ASTM material. The soil foundation beginning not more than 2 foot below the ISFSI pad concrete has an effective soil Young's Modulus not exceeding 28,000 psi. However, the soil-cement mixture extending a maximum of 2 feet directly below the ISFSI pad has an effective Young's Modulus not to exceed 75,000 psi. To ensure that the HI-STORM storage cask 45g limit at the top of the fuel is met, PFSF site-specific tipover and vertical drop events were analyzed by Holtec International (Holtec Report No. 2012653, PFSF Site-Specific HI-STORM Drop/Tipover Analyses) using the same methodology and computer codes used in the analyses discussed in the HI-STORM FSAR.

The results of these analyses are discussed in Sections 4.2.1.5.1E, and 8.2.6 of the PFSF SAR. The results from the site-specific hypothetical tipover analysis demonstrate that the maximum deceleration at the top of the active fuel region is below the HI-STORM design basis value of 45g. The results from the site-specific vertical end drop analysis determine that the maximum cask deceleration remains below 45g for a 9 inch drop height. This required a change to PFSF Technical Specification 4.2.5, "Cask Transporter", to require that the cask transporter be designed to mechanically limit the lifting height of a storage cask to a maximum of 9 inches (the previous maximum permissible lift height was 10 inches). This change in the analyzed drop height maximum permissible lift height required revisions to several other sections in the SAR, and in the Emergency Plan.

Truck Trips

Changes were made to the number of truck trips required to support PFSF construction (imported material truck trips and water truck trips) in ER Section 4.1.7 and Table 4.1-3. The imported material quantities that were substantially modified were the common fill material, materials needed to produce concrete (sand, large aggregate, cement), and soil cement (cement and water). It was previously assumed that the common fill material would be imported. The current design utilizes a site earthwork balance where no common fill material is imported. The overall concrete volume for the facility construction has increased as a result of the increase in the length of the storage pads and the increase in size of the Canister Transfer Building basemat. The site soil cement quantities have increased as discussed above.

The water requirements for PFSF construction are dependent upon the imported material quantities. Imported water is required for making soil cement, and for compacting soils and controlling dust. Since the earthwork and soil cement quantities have changed, the water needs for making soil cement and conducting earthwork activities (compacting soils and controlling dust) changed accordingly.

The following changes to the License Application documents and analyses were required as a result of the changes in the imported material and water truck trips described above:

- Sections 4.1.7.1 through 4.1.7.3 of the ER discuss the effects of noise and traffic for the three construction phases of the PFSF. Truck trip quantities along with noise levels generated from the trips were modified in these sections. These values are also reflected in Tables 4.1.3 and 4.1.6 of the ER.
- Information regarding the volume of concrete production, quantities of earthwork affected, quantities of aggregate, and construction traffic levels associated with phase 1 facility construction were used to revise information on air pollution and air quality impacts in ER Section 4.1.3, and ER Tables 4.1-4 and 4.1-5.

The supporting calculations were also revised as follows:

- The revised water requirements for PFSF construction are calculated in Stone & Webster Calculation 05996.01-P-002 Rev. 5, "Miscellaneous Design Data Required for PFSF Licensing Documents."
- The revised truck trips (imported material and water) are calculated in Stone & Webster Calculation 05996.01-SY-7 Rev. 5, "Truck Traffic Estimates on Skull Valley Road."
- The revised traffic sound levels are calculated in Stone & Webster Calculation 05996.01-E(B)-03 Rev. 3, "Traffic/Sound Levels – Skull Valley Road Construction Thru Operation."

Technical Specifications

PFSF Technical Specification Design Feature 4.2.5, "Cask Transporter", prescribed that the cask transporter was to be designed such as to ensure that it does not begin to tip during the PFSF design basis ground motion. However, this was not consistent with the Technical Specification for the design basis tornado-driven missile for the cask transporter which utilized the drop height limitation. Therefore, for consistency this specification was revised to require that the cask transporter be designed to ensure that the transporter not tip over in the event of the PFSF design basis ground motion, and any tipping must be limited to ensure that the storage cask does not temporarily rise above its analyzed drop height of 9 inches. This now applies the same criteria to the design basis

ground motion for the cask transporter that are specified for the design basis tornado-driven missile.

PFSF Technical Specification Design Feature 4.2.6, "Storage Pads", prescribed requirements for the storage pads to assure that the pads and underlying soil are not harder than the reference storage pad upon which the design basis tipover and vertical end drop accidents are based in the HI-STORM FSAR. This specification was originally extracted from Appendix B, Section 3.4.6 of the HI-STORM 100 Cask System Certificate of Compliance (C of C) No. 72-1014. Holtec International revised the corresponding specification in their HI-STAR storage system C of C, and submitted a proposed amendment to this section of the HI-STORM C of C, which would permit site specific analyses to determine that the 45g deceleration HI-STORM design criteria is not exceeded for hypothetical storage cask tipover and postulated vertical end drop events. PFS revised Design Feature 4.2.6 accordingly, requiring that "The storage pads and underlying foundation shall be verified by analysis to limit cask deceleration during design basis drop and hypothetical tipover events to $\leq$45 g's at the top of the CANISTER fuel basket. Analyses shall be performed using methodologies consistent with those described in the HI-STORM 100 FSAR." This change is reflected in SAR Section 3.2.11.3. Technical Specification 5.5.4, "Onsite Cask Transport Evaluation Program", was revised to be compatible with the revised Design Feature 4.2.6.

Technical Specification Design Feature 4.2.3 was revised to specify the new storage cask spacing requirements, as stated previously.

<u>License Application Chapter 1</u>

Certain information in Chapter 1 of the License Application was updated. For example, the list of the PFS Board of Managers was updated (Section 1.10) to be current. Similarly, the financial information was updated (Section 1.6) to correspond to the information presented by PFS in the licensing proceeding.

<u>Permitting</u>

PFS has updated Chapter 9 of the Environmental Report (Environmental Approvals and Consultation) to take into account the results of the wetland and stream survey conducted by PFS to determine if any jurisdictional waters of the United States are present along the proposed railroad alignment (PFS had committed to such an update in our letter Donnell to U.S. NRC, "Responses to Third Round EIS Request for Information", dated November 7, 2000). This survey concluded that there are no jurisdictional waters of the United States, wetlands or other kinds of water, along the proposed railroad alignment. PFS believes this survey along the rail corridor reflects the characteristics of the entire area around the facility, which has minimal drainage features as compared to the railroad alignment itself. Because of this determination, concurred in by the U.S. Army Corps of

Engineers, various Federal and State permits required under Clean Water Act previously identified in Chapter 9 are not required. Chapter 9 was updated to reflect this detemination and was generally updated as well to reflect PFS's current identification of required permits and status towards obtaining those permits.

If you have any questions or need additional information, please contact me at 303-741-7009.

Sincerely,

John L. Donnell

John L. Donnell
Project Director
Private Fuel Storage L.L.C.

copy to:

Mark Delligatti
Scott Flanders
Asadul Chowdhury
John Parkyn
Jay Silberg
Sherwin Turk
Greg Zimmerman
Scott Northard
Richard E. Condit
John Paul Kennedy
Joro Walker
Denise Chancellor

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

ATTORNEY GENERAL

MAY 10 2001

ENVIRONMENT

May 7, 2001

Mr. John D. Parkyn
Chairman of the Board
Private Fuel Storage, LLC
3200 East Avenue South
La Crosse, WI 54602-0817

SUBJECT: MARCH 30, 2001, LICENSE APPLICATION AMENDMENT

Dear Mr. Parkyn:

The purpose of this letter is twofold: to discuss the Nuclear Regulatory Commission (NRC) staff's concerns regarding the Private Fuel Storage, Limited Liability Company (PFS) license application (LA) amendment, as submitted on March 30, 2001 (and supplemented with calculation packages and letters during the succeeding two weeks), and to request that you attend a management meeting on May 30, 2001, on this subject. The NRC staff believes that the LA amendment does not contain sufficient information to permit a complete and adequate technical review. The staff will not be able to complete its review or formulate a schedule for the development of a supplement to the Safety Evaluation Report (SER) for the proposed PFS facility until all missing information has been submitted. The failure to submit a complete and accurate license application amendment necessarily impacts the schedule for completion of the licensing process, and is likely to impact the schedule for the adjudicatory proceedings before the Atomic Safety and Licensing Board.

As part of our acceptance review of the LA amendment, the staff held a noticed public meeting with PFS on April 18, 2001, in San Antonio, TX. The meeting was held at the offices of the staff's technical assistance contractor, the Center for Nuclear Waste Regulatory Analyses (CNWRA). Enclosed is a list of information not included in the LA amendment which the staff, with the assistance of the CNWRA, identified during the San Antonio meeting. As discussed at the meeting, the staff expects that PFS, expanding upon its April 16, 2001, letter will develop a document discussing all of the changes being proposed in the LA amendment and summarizing the effect of the changes, with supporting bases, on the adequacy of the design. This discussion should demonstrate that PFS considered the need to integrate all new and changed information into all appropriate parts of the license application. This document should be submitted in addition to the information and data described in the enclosure. PFS has provided some of the information identified in the April 18, 2001, meeting in the May 1, 2001, letter from John Donnell of your staff. However, we believe that a significant amount of analytical and supporting information must still be submitted, to address all of the concerns identified and communicated by the staff at the San Antonio meeting, in the meeting summary dated April 25, 2001, and in this letter.

To ensure that we bring timely closure to the safety review, I believe that it would be useful for us to hold the management meeting on May 30, 2001, here at NRC headquarters in Rockville, Maryland. This will be a noticed public meeting. The meeting will provide us with an opportunity to review the progress made to date on the revisions to the LA amendment, as well

as any other outstanding questions related to the completion of the SER supplement and publication of the Final Environmental Impact Statement (and its relationship to the information presented in the LA amendment).

Please be prepared to tell us at the management meeting, your schedule for providing the information necessary to complete the LA amendment submittal. This will allow for appropriate schedule and resource planning by the staff and its contractors. Upon receipt of all of the identified information, we will establish a new review schedule

If you have any questions regarding this letter, or the upcoming management meeting, please contact Mr. Mark Delligatti of my staff at (301) 415-8518.

Sincerely,

E. William Brach

E. William Brach, Director
Spent Fuel Project Office
Office of Nuclear Material Safety
and Safeguards

Docket No.: 72-22

Enclosure: Information missing from License Application Amendment

cc: Service Lists
Dr. A. Chowdhury, CNWRA
Mr. G. Zimmerman, ORNL

DATA NEEDED FOR THE COMPLETION OF THE PFS LA AMENDMENT

Seismic Hazard Analysis:

The following data is needed to complete a review of the new seismic hazard attenuation results submitted in the PFS LA Amendment:

1. Deaggregated hazard curves (mean and fractiles) for horizontal and vertical ground motion for each attenuation and site response model at all 16 frequencies.

2. Site velocity measurements, the 30 random property models (all parameters - shear wave velocity, damping, modulus reduction ratio as a function of shear strain), results of simulations, and input spectra (earthquake magnitude and distance matrix of inputs).

3. Results of the soil structure interaction calculations - spectral ratio or free field vs. building structural foundation (top) motion.

4. Confirmation from Bay Geophysical's experts that the new shear wave velocities will not alter their conclusions regarding the shallow seismic reflection profiles.

5. Complete description of the site soil characterization update including:

   a. site data,
   b. discussion of the site investigation timeline,
   c. complete description of the evolution of the site model, noting parameters that have remained constant as well as those that have changes,
   d. suite of sensitivity results that show the ramifications of changing from a "soil" model to a "rock" model,
   e. sensitivity results to demonstrate the sensitivity (or insensitivity) of the weighting factor (empirical vs. model).

6. Complete revised hazard analysis report (or at least a complete section 6).

7. Well data for soil below 30 ft.

8. More site specific data (i.e., beyond the one existing deep well) for the soil between 30 ft and the Tertiary strata or provide an analysis that shows that the applicant has captured the uncertainty of the soil properties sufficiently such that any new information will not again significantly change the ground motions (i.e., sensitivity study of the site response model that would incorporate the variability of the soil parameters expected for this site).

Soil Engineering:

1. A site plan showing location of any new borings and test pits used to support PFS analyses.

2. Logs for any new borings or test pits used to support PFS analyses.

3. Revised analyses of the stability of the storage pads to include a clear identification of the potential failure modes and failure surfaces, and the material strengths required to

satisfy the regulatory requirement, considering the critical failure modes and failure surfaces.

Design of Facility:

Storage Pads

1. Assessment of the edge effects on the stability of the Storage Pads under new seismic loads.

Cask Transfer Building

1. General description of the major structural elements of the CTB. This should include the reinforced concrete walls, columns, roof, and slab and the structural steel elements including the roof support beams.

2. New calculation package (SC) for Design of Tornado Doors on cells in canister transfer building (CTB).

3. New SC for Design of roof steel members.

4. Updated letter from Ederer, Incorporated on impact of new seismic levels.

5. Updated G(B)-11 Dynamic Settlements of the soils underlying the site.

6. Updated SC-4 Impedance Functions for CTB.

7. Assessment of the design changes to the slab in terms of load transfer from the walls to the slab and resulting loads on soils. Emphasis should be on the pad areas extending beyond the building walls.

8. Assessment of fire impact on the new design of the CTB.

9. Assessment of the drop of a cask onto the slab of the CTB.

cc: Private Fuel Storage

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