2.1 Conduct of Review

REMOVE the second paragraph in Section 2.1 on page 2-1 of the SER and INSERT:

The staff evaluated site characteristics by reviewing Chapter 2 of the SAR, documents cited in the SAR, and other relevant literature. The staff also considered information and analyses, with respect to geotechnical and seismic considerations, that were submitted by the applicant (Private Fuel Storage, 2001) subsequent to the issuance of the staff's SER on September 29, 2000. The applicant requested an exemption to 10 CFR 72.102(f), which requires a deterministic seismic hazard analysis (DSHA) approach for determining the impact of earthquakes on the Facility. The applicant requested instead to apply a probabilistic seismic hazard analysis (PSHA) approach for analyzing potential seismic events. The staff reviewed this exemption request, as presented in Chapter 2 of the SAR, and conducted an independent evaluation of seismic ground motion hazard at the site based on a survey of existing literature, state of the knowledge in PSHAs and DSHAs, and consideration of existing NRC regulations and regulatory guidance documents (Stamatakos et al., 1999) regarding seismic analyses. As discussed in Section 2.1.6.2, the staff agrees that the use of the PSHA methodology with a 2,000-year return period is acceptable and there is a sufficient basis to grant an exemption to 10 CFR 72.102(f) at the time a license is issued for the Facility. The exemption will only be issued upon completion of the applicable regulatory process described in 10 CFR Part 72. As discussed in Chapters 4 and 5 of this SER, the Facility is designed to withstand a 2,000-year return period ground motion.

2.1.1.4 Land and Water Uses

REMOVE the first full paragraph on page 2-5 (in Section 2.1.1.4) of the SER and INSERT:

Domestic water wells in Skull Valley, Utah, are almost exclusively in unconsolidated alluvial fan deposits along the east side of the valley. Some stock wells in the central part of Skull Valley, Utah, operate under artesian conditions (Arabasz et al., 1987). Water quality varies from good along the east side of the valley to poor in the central part due to the high total dissolved solids content. It is anticipated that water wells will be drilled within the Facility's controlled area to accommodate water needs during construction and operation of the Facility. The applicant will locate and develop the water wells in a manner that prevents any impact (e.g., groundwater drawdown) on adjacent wells (the nearest of which is 1.5 miles from the Facility). Estimated water pumpage from all sources in Skull Valley, Utah is about 5,000 acre-feet of water per year. The applicant estimates water needs at no more than 10,000 gallons per day during construction and on average 1,800 gallons per day during operation. Assuming a conservative 365 days in a year, the water usage is estimated to be 11.2 acre-feet per year during construction activities and 2.0 acre-feet per year for operational activities. Therefore, the projected amount of water used during construction activities is about 0.2 percent (11.2 acre-feet divided by 5,000 acre-feet used per year) of current total water production estimated in Skull Valley and 0.04 percent (2.0 acre-feet divided by 5,000 acre-feet used per year) for operations. These water-use amounts attributed to the Facility are very small when compared with the total ground water budget and should have no perceptible impact on current water use.

REMOVE Sections 2.1.6, 2.2, and 2.3 on pages 2-24 to 2-65 of the SER and INSERT:

2.1.6 Geology and Seismology

Section 2.6 of the SAR, Geology and Seismology, describes the geological and seismological setting of the proposed site, geographically located within Skull Valley, Utah. This review corresponds to the following sections of the SAR: 2.6.1, Basic Geologic and Seismic Information; 2.6.2, Vibratory Ground Motion; 2.6.3, Surface Faulting; 2.6.4, Stability of Subsurface Materials; and 2.6.5, Slope Stability. The review includes the applicant's SAR (Private Fuel Storage Limited Liability Company, 2001), responses to requests for additional information (Parkyn, 1999a, 2001; Donnell, 1999d, 2001), and additional supporting documents (Geomatrix Consultants, Inc., 1999a, 2001a, 2001b, 2001c, and 2001d; Bay Geophysical Associates, Inc., 1999; Northland Geophysical, L.L.C., 2001; Stone and Webster Engineering Corporation, 2001a, 2001b, 2001c, 2001d). The staff reviewed the geology and seismology of the site with respect to the following regulatory requirements:

- 10 CFR 72.90(a) requires site characteristics that may directly affect the safety or environmental impact of the ISFSI to be investigated and assessed.

- 10 CFR 72.90(b) requires proposed sites for the ISFSI to be examined with respect to the frequency and severity of external natural and man-induced events that could affect the safe operation of the ISFSI.

- 10 CFR 72.90(c) requires design basis external events to be determined for each combination of proposed site and proposed ISFSI design.

- 10 CFR 72.90(d) requires the proposed sites with design basis external events for which adequate protection cannot be provided through ISFSI design be deemed unsuitable for the location of the ISFSI.

- 10 CFR 72.92(a) requires that natural phenomena that may exist or that can occur in the region of a proposed site must be identified and assessed according to their potential effects on the safe operation of the ISFSI. The important natural phenomena that affect the ISFSI design must be identified.

- 10 CFR 72.92(b) requires that records of the occurrence and severity of those important natural phenomena must be collected for the region and evaluated for reliability, accuracy, and completeness. The applicant shall retain these records until the license is issued.

- 10 CFR 72.92(c) requires that appropriate methods must be adopted for evaluating the design basis external natural events based on the characteristics of the region and the current state of knowledge about such events.

- 10 CFR 72.98(a) requires that the regional extent of external phenomena, man-made or natural, that are used as a basis for the design of the ISFSI be identified.

- 10 CFR 72.98(b) requires that the potential regional impact due to the construction, operation or decommissioning of the ISFSI be identified. The extent of regional impacts must be determined on the basis of potential measurable effects on the population or the environment from ISFSI activities.

- 10 CFR 72.98(c) requires that those regions identified pursuant to paragraphs 10 CFR 72.98 (a) and (b) must be investigated as appropriate with respect to: (1) The present and future character and the distribution of population, (2) Consideration of present and projected future uses of land and water within the region, and (3) Any special characteristics that may influence the potential consequences of a release of radioactive material during the operational lifetime of the ISFSI.

- 10 CFR 72.102 (b) requires that West of the Rocky Mountain Front (west of approximately 104 west longitude), and in other areas of known potential seismic activity, seismicity will be evaluated by the techniques of Appendix A of Part 100 of this chapter. Sites that lie within the range of strong near-field ground motion from historical earthquakes on large capable faults should be avoided.

- 10 CFR 72.102(c) requires that sites other than bedrock sites must be evaluated for their liquefaction potential or other soil instability due to vibratory ground motion.

- 10 CFR 72.102(d) requires that site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading.

- 10 CFR 72.102(e) requires that in an evaluation of alternative sites, those which require a minimum of engineered provisions to correct site deficiencies are preferred. Sites with unstable geologic characteristics should be avoided.

- 10 CFR 72.102(f) requires that the design earthquake for use in the design of structures must be determined as follows: (1) For sites that have been evaluated under the criteria of Appendix A of 10 CFR Part 100, the design earthquake must be equivalent to the safe shutdown earthquake for a nuclear power plant. (2)

Regardless of the results of the investigations anywhere in the continental U.S., the design earthquake must have a value for the horizontal ground motion of no less than 0.10 g with the appropriate response spectrum.

- 10 CFR 72.122(b) requires (1) Structures, systems, and components important to safety must be designed to accommodate the effects of, and to be compatible with, site characteristics and environmental conditions associated with normal operation, maintenance, and testing of the ISFSI and to withstand postulated accidents. (2) Structures, systems, and components important to safety must b designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, lightning, hurricanes, floods, tsunami, and seiches, without impairing their capability to perform safety functions. The design bases for these structures, systems, and components must reflect: (i) Appropriate consideration of the most severe of the natural phenomena reported for the site and surrounding area, with appropriate margins to take into account the limitations of the data and the period of time in which the data have accumulated, and (ii) Appropriate combinations of the effects of normal and accident conditions and the effects of natural phenomena. The ISFSI 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. (3) Capability must be provided for determining the intensity of natural phenomena that may occur for comparison with design bases of structures, systems, and components important to safety. (4) If the ISFSI is located over an aquifer which is a major water resource, measures must be taken to preclude the transport of radioactive materials to the environment through this potential pathway.

Summary of Review

The staff reviewed information presented in Section 2.6 of the SAR, Geology and Seismology. The staff also reviewed relevant literature cited in the SAR and and supporting documents. In Revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000), geologic and seismic information included (i) review of published and unpublished literature;
(ii) reconnaissance geological mapping of the valley; (iii) the test boring program performed by Earthcore, Inc. under the supervision of Stone & Webster Engineering Corporation (Appendix 2A); (iv) P and S wave seismic reflections surveys performed by Geosphere Midwest under the supervision of Stone & Webster Engineering Corporation (Appendix 2B); (v) the DSHA performed by Geomatrix Consultants, Inc. (Appendix 2D); (vi) a consulting report on surface geomorphology features prepared by D. Curry of the University of Utah, at the request of Stone & Webster (Appendix 2C); (vii) consultant reports on the composition and age of volcanic ash layers found in the test borings prepared by W. Nash of the University of Utah (Appendix 2E); and (viii) additional seismic evaluation (Appendix 2G).

The applicant's response to Round 1 RAIs 2-5 and 2-7 (Parkyn, 1999a; Donnell, 1999d; Geomatrix Consultants, Inc., 1999a; Bay Geophysical Associates, Inc., 1999), provided an extensive 8-month geological and geophysical investigation of the site. The additional analyses are summarized by Geomatrix Consultants, Inc. (1999a) and Bay Geophysical Associates, Inc. (1999). Additional information provided in support of fault displacement and seismic hazard assessments includes (i) a 1:12,000-scale compilation map of geology and surface features; (ii) supplementary discussions with R.B. Smith, R. Bruhn, and W. Arabasz of the University of Utah, and J.M. Helm, all professional researchers with expert knowledge of local and regional geological and geophysical conditions; (iii) two regional cross sections showing possible relationships of faults to the depth of the seismogenic crust; (iv) photo-geologic interpretations of low-sun-angle photographs of geomorphic features; (v) reconnaissance field investigations of active faulting along southern segments of the Stansbury fault; (vi) existing proprietary gravity data of the valley, previously collected by EDCON in support of petroleum exploration; (vii) 3.8 miles of high-resolution S-wave seismic reflection data acquired by Bay Geophysical Associates, Inc. (1999); (viii) reprocessed industrial P-wave reflection seismic data; (ix) ground magnetic and electric conductivity data acquired to assess the feasibility of additional ground penetrating radar studies; (x) 30 new boreholes drilled across the site to provide additional control on subsurface stratigraphy and to support surface mapping and subsurface geophysical investigations; (xi) 25 test pits and 2 trenches excavated on the site to provide detailed profiles of near-subsurface faulting and stratigraphy; (xii) geochronologic age dating to determine radiometric ages of important stratigraphic horizons used to correlate paleo-lake deposits and confirm ages of inferred Bonneville Lake cycle stratigraphy; (xiii) the applicant's PSHA; and (xiv) a probabilistic fault displacement assessment.

The applicant provided documentation (Donnell, 1999g) on formulation of ground-motion and fault-displacement hazards, including the methodology used to develop the probabilistic seismic and fault displacement hazard assessments and the applicability of methods and results for this analysis developed in conjunction with the U.S. Department of Energy (DOE) seismic hazard analyses at Yucca Mountain, Nevada. Discussion included how models and data generated by DOE expert elicitation for ground-motion and fault-displacement hazards at Yucca Mountain are applicable to the applicant's site in Skull Valley, Utah.

The applicant also provided (i) gravity data used to support geological interpretations of the site geology (specifically, the industry EDCON gravity data set and gravity profiles collected by J. Baer at Brigham Young University), (ii) assessments of near-field ground motions from earthquakes that could possibly occur on faults near the site, and (iii) updated deterministic ground-motion assessment for the site based on recent revisions to the site characterization for comparison to probabilistic assessment (Donnell, 1999h; Parkyn, 1999b).

The applicant completed additional site characterization work in January, 2001 to verify shear-wave velocity information in the upper strata of the soil column (approximately 35 feet) at the PFS Facility site. In particular, shear-wave velocity data were collected by Northland Geophysical, L.L.C (2001) in two boreholes located near the planned Waste Handling Facility

CTB-5(OW) and CTB-5A. The applicant also performed additional site-response modeling to evaluate the effects of alternative interpretations of the soil shear-wave velocity, modulus reduction, and damping on the ground motion hazard.

In March 2001, Geomatrix Consultants, Inc., (2001a, and 2001d) revised the dynamic soil properties, proposing a 9-sediment-layer soil model as representative of the site for a depth of 700 feet. Six of these layers are located within the upper 35 feet. These changes in the dynamic soil properties also led the applicant to make two changes to the ground motion attenuation models.

Based on the revised site model, the applicant concluded in the SAR that the PFS site had properties closer to that of a generic western United States rock site (Geomatrix Consultants, Inc., 2001a, Appendix F) than the soil model it had used previously. In Revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000), the applicant likened the Skull Valley soils to a generic western United States deep soil site (Geomatrix Consultants, Inc., 1999a, Appendix F). This information and re-characterization of the PFS site led to a change in the set of empirical ground motion attenuation models that were the starting point for the development of the PFS ground motion models.

A second revision to the applicant's methodology involved the site adjustment factors to account for the differences in the generic western United States site and the proposed PFS site in Skull Valley. In Revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000), the applicant used a site response model to estimate site response adjustment factors (Geomatrix Consultants, Inc., 1999a, Appendix F). In the revised analysis presented in Revision 22 of the SAR (Fuel Storage Limited Liability Company, 2001; Geomatrix Consultants, Inc., 2001a, Appendix F), the applicant considered two alternative approaches to estimate the site response factors. The first approach was based on site response modeling and the new site model. The second approach derived site adjustment factors from empirical strong motion data. The final site adjustment factor was determined from a weighting of the two approaches. The applicant assigned a 1/3 weight to the empirical strong motion approach and a 2/3 weighting to the site response modeling approach.

The staff evaluated the revised information submitted by the applicant in the revised SAR and all pertinent documents and analyses (revised and new). This new and revised information includes the following references: (Private Fuel Storage Limited Liability Company, 2001); (Geomatrix Consultants, Inc., 2001a, 2001b, 2001c, and 2001d); (Northland Geophysical, L.L.C., 2001; (Stone and Webster Engineering Corporation, 2001a,b,c,d); and (Donnell, 2001; Parkyn, 2001).

The staff has reviewed the information presented in Section 2.6, Geology and Seismology in the SAR regarding the site. The documentation is acceptable because the breadth and depth of geological and geophysical investigations, especially those reported in Geomatrix Consultants, Inc. (1999a; 2001a), represent a comprehensive technical foundation of geological

knowledge from which the potential for seismic and faulting hazards at the site can be adequately deduced. The applicant has sufficiently documented these investigations in the SAR and supporting documents. The staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Facility, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR 72.92(a), 72.92(b), and 72.102(e) with respect to this issue.

2.1.6.1 Basic Geologic and Seismic Information

Basic geologic and seismic characteristics of the site and vicinity are presented in Section 2.6.1 of the SAR, Basic Geological and Seismological Information. These include discussions of physiographic background and site geomorphology, regional and site geological history, structural geologic conditions, and engineering evaluation of geologic features. Detailed static and dynamic engineering properties of soil and rock underlying the site are presented in Section 2.6.4 of the SAR, Stability of Subsurface Materials.

Physiography and Site Geomorphology

As summarized in the SAR, the proposed site is located in the northeastern margin of the Basin and Range Province, a wide zone of active extension and distributed normal faulting that extends from the Wasatch Front in central Utah to the Sierra Nevada Mountains in western Nevada and eastern California. Topography within the Basin and Range Province reflects Miocene to recent, east-west extensional faulting, in which tilted and exhumed footwall blocks form subparallel north-south striking ranges separating elongated and internally drained basins. Ranges are up to several hundred kilometers long with elevations up to 6,500 feet above the basin floors. Much of the surface faulting took place at the base of the ranges along normal faults that dip moderately (~60°) beneath the adjacent basins (herein defined as range-front faults), although complex faulting within the basins is also common [i.e., the fault-rupture patterns of the 1954 Rainbow Mountain-Stillwater or 1959 Hebgen Lake earthquakes as summarized in dePolo et al. (1991)].

The proposed site in Skull Valley lies in one of the typical basins of the province, bounded on the east by the Stansbury Mountains and the Stansbury fault and on the west and south by the Cedar Mountains and the East Cedar Mountain fault. The basin is underlain by late Quaternary lacustrine deposits laid down from repeated flooding of the valley during transgressions of intermontane lakes, most notably Lake Bonneville, which flooded Skull Valley several times during the Pleistocene and Holocene (e.g., Currey and Oviatt, 1985). These deposits form the basis for paleoseismic evaluations of the Skull Valley site. Topography of the proposed site is relatively smooth, reflecting the origin of the valley floor as the bottom of Lake Bonneville. The site gently slopes to the north with a slope of less than 0.1°. Detailed topographic maps of the region and the site were provided in the SAR. This smooth valley floor contains small washes up to 4 feet deep and soil ridges up to 4 feet high.

The geomorphology of Skull Valley in the vicinity of the site is typical of a semiarid to arid desert setting. The adjacent mountain ranges are affected by mass-wasting processes and stream erosion that deliver sediment loads to a complex of alluvial fans (aprons) situated at the bases of the ranges. Runoff is conveyed down the ranges and over the alluvial fans through a series of small channels to the valley floor. Stream and spring flows are absorbed into the fan and the valley floor near the fan-floor interface, resulting in minimal surface runoff reaching the central valley near the site. There is no evidence of flash-flooding near the site nor are there deposits indicative of geologically recent [last 2 Ma (million years)] mudflows or landslides.

The valley floor near the site comprises beach ridges and shoreline deposits interrupted by bedrock outcrops, such as Hickman Knolls rising about 400 feet above the valley bottom. The valley bottom relief comprises a series of braided, northerly flowing dry washes. The washes are disrupted and convey runoff for only short distances before merging into other washes or open space. This network of shallow washes extends offsite to the north where it confluences with the central valley drainage system and from there flows to the Great Salt Lake. The only perennial surface water is located approximately 10 miles north of the site. The central valley in the vicinity of the Facility is unaffected by fluvial processes.

In the southern and eastern parts of the proposed site, numerous north-trending linear sand ridges interrupt the otherwise smooth valley floor. The ridges, which are typically 8 feet high and 100 feet wide, were originally mapped as possible fault traces by Sack (1993). In the SAR, a brief summary report (Appendix 2C) reviewed the available surficial information and concluded that these features constitute sandy beach ridges deposited by southward longshore transport within the Stansbury shoreline coastal zone of Lake Bonneville. The applicant provided technical information (Parkyn, 1999a) about the nature and origin of the ridges to substantiate the conclusions reached in Appendix 2C of the Revision 2 of the SAR. This information, especially Figure 1-3 and associated discussion in Section 5.2.1 of Geomatrix Consultants, Inc. (1999a), was sufficient to document the conclusions. In addition, discussion of the stratigraphic relationships in test pit T-11 (Geomatrix Consultants, Inc., Revision 10, 1999a, Volume II, Figure C-1) provided additional technical information in support of the conclusion that these ridges have a depositional not tectonic origin.

In a few locations, bedrock composed of Paleozoic carbonate rocks crop out of the smooth valley floor. The largest of these is a small group of hills 1.3 miles south of the proposed site known as Hickman Knolls. Rocks of this outcrop are medium to dark gray dolomite breccia. The origin and stratigraphic correlation of the Hickman Knolls carbonate rocks within the Paleozoic section is not well known. The preferred interpretation put forth by Geomatrix Consultants, Inc. (1999a) is that they are rooted bedrock outcrops. The alternative interpretation based on independent modeling of gravity data by the staff (Stamatakos et al., 1999) is that they are landslide deposits, resting unconformably on the Tertiary sediments in the valley. Geomatrix Consultants, Inc. (1999a) correlated them with the Upper Ordovician Fish Haven Formation based on descriptions of the regional stratigraphy by Hintze (1988) and the geological bedrock maps of Teichert (1959) and Rigby (1958). The differences in these two

interpretations lead to differences in the estimated seismic hazard. In the Geomatrix preferred interpretation, rooted bedrock requires a significant and seismogenic fault just west of Hickman Knolls. In the alternative interpretation, no such fault is necessary. Therefore, the Geomatrix preferred interpretation leads to a slightly more conservative seismic hazard (see Stamatakos et al., 1999, for complete discussion).

The applicant's surface mapping and related field investigations (Geomatrix Consultants, Inc. 1999a) are sufficient to show that Hickman Knolls shows no evidence of significant karst features (e.g., collapsed solution cavities). Karstification is also not widespread in carbonate bedrock of the surrounding ranges. Because similar rocks lie beneath the valley floor, the staff concludes that karst processes have not affected the site and are not a concern to site suitability.

Regional and Site Geologic History

The SAR discusses the geological history of the site and surrounding region. The discussion includes background information about the tectonic setting of the region in the Precambrian and Paleozoic that led to the deposition of the bedrock stratigraphy presently exposed in the Stansbury and Cedar Mountains. In brief, the structural framework of bedrock across the region reflects overprinting of several major periods of North American tectonic activity. These include contractional deformation structures such as thin- and thick-skinned thrusts and folds associated with the Devonian Antler, Jurassic to Cretaceous Sevier, and Cretaceous-Tertiary Laramide orogenies (e.g., Cowan and Bruhn, 1992) and extensional normal and detachment faults associated with the Eocene to the current Basin and Range extension (e.g., Wernicke, 1992; Axen et al., 1993).

The proposed site lies near the center of a typical Basin and Range valley, situated between roughly north-south and northwest-southeast elongated ranges of exhumed bedrock. Exhumation of the ranges was accomplished by extensional faulting along range-front normal faults. Faulting tilted the ridges to the east. The adjacent basins subsided concomitant with exhumation while they accumulated sediment shed from the eroding ranges. In Skull Valley, as in much of central and western Utah, the valleys are also flooded by transgressions of the intramontane saline lakes. Tertiary and Quaternary deposits in and around the site document numerous transgressions associated with Lake Bonneville and pre-Lake Bonneville lacustrian cycles. The Great Salt Lake is the present-day remnant of Lake Bonneville.

In the SAR, the structural framework of the site within the valley is based on interpretations presented in the available literature integrated with detailed site geological studies, including site stratigraphy, geologic mapping, cross-sectional construction, and geophysical investigations (Geomatrix Consultants, Inc.,1999a; Bay Geophysical Associates, Inc., 1999). Most important to the evaluations of seismic and faulting hazards was identification and characterization of a detailed Quaternary stratigraphy, that provided critical constraints on faulting activity and local and regional active faults.

Valley fill sediments in Skull Valley consist of Tertiary age siltstones, claystones, and tuffaceous sediments overlain by Quaternary lacustrian deposits. Late Miocene to Pliocene deposits of the Salt Lake Formation were exposed in Trench T1 and in Boring C-5. Microprobe analyses of glass shards from vitric tuffs (ash fall deposits) within the sediments were used to correlate the tuffs with volcanic rocks of known age. The analyses indicate ages for the stratigraphic units between 16 and 6 Ma consistent with the known age of the Salt Lake Formation. Microprobe analyses, performed by M. Perkins at the University of Utah, are documented in Appendix D of Geomatrix Consultants, Inc. (1999a).

During the Quaternary (approximately the last 2 Ma), especially the last 700 ka (thousand years), sedimentation in Skull Valley was dominated by fluctuations associated with lacustrian cycles in the Bonneville Basin (e.g., Machette and Scott, 1988; Oviatt, 1997). The SAR provides a detailed analysis of these deposits from trenches, test pits, and borings, including two radiocarbon ages on ostracodes and charophytes. The radiocarbon ages were performed by Beta Analytic, Inc. under the direction of G. Hood and are documented in Appendix D of Geomatrix Consultants, Inc. (1999a).

The stratigraphy was also critical to interpretations of the reflection seismic profiles. Two prominent paleosols were developed during inter pluvial periods near the Tertiary-Quaternary boundary (~2 Ma) and between the Lake Bonneville and Little Valley cycles (130–28 Ka). These buried soils are characterized by relatively well-developed pedogenic carbonate, both in the soil matrix and as coatings on pebbles. As such, these paleosols form strong reflectors that are readily apparent on the seismic reflection profiles. These horizons were also correlated with cores from the borings drilled directly beneath the seismic profile lines. These detailed constraints on the Quaternary stratigraphy and the high quality seismic reflection profiles provided in Geomatrix Consultants, Inc. (1999a) are sufficient to document the Quaternary faulting record of the site and to provide a necessary stratigraphic framework for reliable paleoseismic analyses of active faults in and around Skull Valley.

Structural Geologic Conditions

Primary faults. Classical structural models for the Basin and Range envision a simple horst and graben framework in which range-front faults are planar and extend to the base of the transition between the brittle and ductile crust, 9–12.5 miles below the surface (e.g., Stewart, 1978). More recent work has shown that many normal faults are not planar but curved or listric, and they sole into detachments that may or may not coincide within the brittle-ductile transition in the crust (e.g., Wernicke and Burchfiel, 1982). In Skull Valley, the detachment model places the Stansbury fault as the master or controlling fault of a half graben. The other side of the half graben would include the antithetic East Cedar Mountain fault and a series of antithetic and synthetic faults within the basin, all of which would sole into the Stansbury fault 1–12.5 miles deep in the crust. Details of these two alternatives to fault geometry are discussed in Stamatakos et al. (1999).

In Geomatrix Consultants, Inc. (1999a), two regional cross sections were developed that depict the overall structural framework of Skull Valley and the surrounding ranges. These cross sections were constructed from a compilation and analysis of existing geological map data, reprocessed and new seismic profiles across the valley, and interpretation of proprietary gravity data. The cross sections were based on acceptable structural geology procedures for cross-sectional restoration and interpretation of subsurface geometries (e.g., Woodward et al., 1989; Suppe, 1983). The cross sections depict a series of pre-Tertiary folds and thrusts related to the Sevier and older contraction deformation that have been cut by a series of Tertiary and Quaternary normal faults related to Basin and Range extension. The normal faults are considered moderately dipping (~60°) planar features following the horst and graben model described previously.

As discussed in Stamatakos et al. (1999), this horst and graben model is conservative for predicting a maximum earthquake potential for these faults. Faults that extend all the way to the base of the seismogenic crust define a larger area for earthquake rupture and thus greater maximum magnitude earthquakes than those that terminate into a detachment above the brittle-ductile transition. The added feature of a detachment beneath the valley does not contribute to the earthquake hazard because large earthquakes on detachment faults are exceedingly rare or nonexistent (Wernicke, 1995; Ofoegbu and Ferrill, 1997). The staff notes the horst and graben model does not consider the possibility of triggered ruptures (e.g., rupture of the master basin fault triggering subsequent co-seismic ruptures on the opposing antithetic or synthetic faults in the basin). This is acceptable because the faults act independently.

The cross sections show three first-order, west-dipping normal faults and one east-dipping fault (the East Cedar Mountain fault). The west-dipping faults are the Stansbury and two previously unknown faults in the basin informally named the East and West faults. These new faults were interpreted based mainly on analyses of the gravity and seismic reflection data and by analogy to other faults in the Basin and Range. Discovery of these new faults and related structures has important implications to both the seismic and fault displacement hazard assessments (see Sections 2.1.6.2 and 2.1.6.3).

A critical aspect of the interpretation of the East and West faults centers on the origin and nature of rocks exposed at Hickman Knolls, which are composed of monolithologic carbonate breccias. Two possibilities were presented in the SAR:

(1) The breccias are part of a detached landslide block of a bedrock dislodged from one of the nearby ranges by Tertiary or Quaternary earthquake activity along the range fronts.

(2) The breccias are rooted to the Paleozoic basement beneath the basin fill. (In this latter interpretation, brecciation and related features represent *in situ* deformation associated with early post-depositional processes.)

Alternative (1) was based on an interpretation of gravity data collected and analyzed by J. Baer of Brigham Young University. Indeed, many characteristics of the Hickman Knolls breccias are similar to mapped landslide deposits throughout the Basin and Range Province (e.g.,Yarnold, 1993; Bishop, 1997). Observations of chaotic and low-angle faulting and folding of the Tertiary deposits in Trench T-1 also suggest Tertiary landslide activity (Geomatrix Consultants, Inc., 1999a).

Alternative (2) was based on the Geomatrix Consultants, Inc. (1999a) interpretation of the proprietary industry gravity data and detailed mapping of the meso-scale structures at Hickman Knolls. Deformation features, especially low-angle and high-temperature ductile shears overprinted by minor low-temperature and brittle faults and fractures, suggest a protracted history of *in situ* deformation of rooted bedrock. In this interpretation, the deformation of the Tertiary sediments in Trench T-1 are considered to represent a local landslide that originated on the flanks of Hickman Knolls itself.

The difference between these alternatives is important to structural interpretations of Skull Valley. In alternative (1), the significant structural relief of the basin would lie east of Hickman Knolls along both the East and Stansbury faults. This interpretation would reduce cumulative displacement along the East fault and thereby reduce its contribution to the overall seismic hazard. This interpretation is represented in Geomatrix Consultants, Inc. (1999a) seismogenic fault rupture Model A. In alternative (2), major relief in the basin lies west of the Knolls with significant displacement along the West fault. In this alternative, the West fault becomes a significant contributor to the overall seismic hazard as represented in Geomatrix Consultants, Inc. (1999a) seismogenic fault rupture Model B. Alternative (2) is favored in Geomatrix Consultants, Inc. (1999a), although some credence is given to alternative (1). In building the logic tree for seismogenic sources in the PSHA, alternative (1) is given a weight of 0.3 and alternative (2) is given a weight of 0.7 (see discussion in Section 2.1.6.2).

Independent analysis of EDCON gravity data provided in the SAR (Stamatakos et al., 1999) favors alternative (2). The West fault appears to be a splay of the East fault and, therefore, not capable of independently triggering earthquakes. Given that Geomatrix Consultants, Inc. (1999a) included the West fault coupled with other conservative assumptions about seismicity, Stamatakos et al. (1999) concluded that the Geomatrix Consultants, Inc. assessment has led to a conservative hazard assessment, in terms of the seismic source characterization.

Secondary faults. Within the valley fill itself, the SAR documents several additional secondary faults designated as fault zones A to F. Each fault zone has a number of secondary splays that are designated with numeral subscripts (e.g., A1 to A7, B1 and B2, and so forth). These fault zones are all considered secondary faults related to deformation of the hanging wall above the larger East and West faults. They are too small to be independent seismic sources but large enough to be considered important in the fault displacement analysis. The largest of the secondary faults is F fault, which appears to be a splay of the East fault. The characteristics of

these secondary faults and their contributions to the surface faulting hazard at the proposed site are discussed in detail in Section 2.1.6.3 of this SER.

Engineering Evaluation of Geologic Features

The static and dynamic engineering soil and rock properties of the various materials underlying the site are evaluated in Section 2.1.6.4 of this SER. The properties evaluated include grain size classification, Atterberg limits, water content, unit weight, shear strength, relative density, shear modulus, Poisson's ratio, bulk modulus, damping, consolidation characteristics, seismic wave velocities, density, porosity, strength characteristics, and strength under cyclic loading.

Staff Review

The staff reviewed the information in Section 2.6.1 of the SAR and found it acceptable because the basic geologic and seismic characteristics of the site and vicinity have been adequately described in detail to allow investigation of seismic characteristics of the Facility. The staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Facility, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR 72.92(a), 72.92(b), 72.102(e), and 72.122(b) with respect to this issue.

2.1.6.2 Ground Vibration and Exemption Request

Earthquake ground motion is discussed in Section 2.6.2 of the SAR, Vibratory Ground Motion. In the SAR, vibratory ground motion is addressed through discussions of historical seismicity and procedures to determine the design earthquake, including identification of potential seismic sources and their characteristics, correlation of earthquake activity with geologic structures, maximum earthquake potential, and seismic wave transmission characteristics.

According to 10 CFR 72.122(b)(2), structures, systems, and components important to safety must be designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions. For sites west of the Rocky Mountains, such as Skull Valley, 10 CFR Part 72 requires that seismicity be evaluated by techniques set forth in Appendix A of 10 CFR Part 100 for nuclear power plants. This appendix defines the safe shutdown earthquake as the earthquake that produces the maximum vibratory ground motion at the site, and requires that the structures, systems, and components be designed to withstand the ground motion produced by the safe shutdown earthquake. This seismic design method implies use of a DSHA approach because it considers only the most significant event, and the method is a time-independent statement (i.e., it does not take into consideration the planned operating period of the Facility or how frequent or rare the seismic events are that control the deterministic ground motion). Also, 10 CFR 72.102(f)(1) requires that analyses using the

Appendix A methodology use a design peak horizontal acceleration equivalent to that of the safe shutdown earthquake for a nuclear power reactor.

A detailed geological survey conducted by Geomatrix Consultants, Inc.(1999a) identified additional faults in the vicinity of the site. Taking into account these newly discovered faults with the DSHA methodology, in revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000), the applicant estimated the peak horizontal and vertical acceleration values from the seismic event to be 0.72 and 0.80g, respectively (Geomatrix Consultants, Inc., 1999b). In Revision 22 of the SAR (Private Fuel Storage Limited Liability Company, 2001), using the DSHA methodology, the applicant estimated the peak horizontal and vertical acceleration values from a seismic event to be 1.15g and 1.17g respectively (Geomatrix Consultants Inc., 2001d). These values exceed the SAR proposed design values.

To resolve the issue of seismic design, the applicant submitted to the NRC, a request for an exemption to the seismic design requirement of 10 CFR 72.102(f)(1) to use PSHA along with considerations of risk to establish the design earthquake ground motion levels at the Facility (Parkyn, 1999b). The exemption request also proposed to design the Facility to the ground motions produced by 1,000-year return period earthquakes. Based on information supporting Revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000), these design-ground motions were calculated to have a peak horizontal acceleration of 0.40g and a peak vertical acceleration of 0.39g, resulting from a recent site-specific PSHA conducted by the Geomatrix Consultants, Inc. (1999a). These values were subsequently updated in Revision 22 of the SAR (Private Fuel Storage Limited Liability Company, 2001), as discussed below.

As part of the evaluation of PFS's exemption request, the staff conducted an independent technical review of seismic hazard investigations at the proposed site (Stamatakos et al. 1999). The objectives of this seismic investigation were to (i) conduct an independent review of existing seismic hazard studies at Skull Valley, in particular, to identify seismic and faulting issues important to siting the Facility; (ii) evaluate the adequacy and acceptability of PFS's seismic design approach; and (iii) determine an appropriate design basis return period for the PFS-proposed seismic design approach. The staff conducted its evaluation by reviewing information provided by the applicant, surveying other state-of-the-art literature, analyzing the bases of current NRC regulations, and performing independent analyses of geophysical data and sensitivity studies of model alternatives and consideration of uncertainties. This section of the SER summarizes information presented in the Revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000), the result of the staff's independent investigation, and staff's review of new information presented in Revision 22 of the SAR (Private Fuel Storage Limited Liability Company, 2001). A summary is included at the end of this section pertaining to the staff's evaluation of the adequacy of the PFS-proposed seismic design for the Facility.

Geological and Seismotectonic Setting

Seismicity in the Basin and Range is generally concentrated along the Wasatch Front, Sierra Nevada and a medial zone called the Central Nevada Seismic Belt (dePolo et al., 1991). Within the region surrounding the proposed site are four seismotectonic provinces: (i) the Basin and Range, (ii) Wasatch Front as part of the Intermountain Seismic Belt, (iii) the Snake River Plain, and (iv) the Colorado Plateau. Of these four seismotectonic provinces, the Wasatch Front is the only one with levels of seismic activity that could affect the proposed site (see Stamatakos et al. (1999) for a more thorough discussion of the seismotectonic provinces).

The Skull Valley site is approximately 50 miles west of the Wasatch Front. The seismotectonic setting of the proposed site was discussed (Private Fuel Storage Limited Liability Company, 2000, Appendix 2D) within the larger context of the tectonic evolution and historic seismicity of the western Cordillera. This discussion included a brief discourse of regional crustal stresses and the driving forces of the Basin and Range extension. The SAR concluded that gravitationally derived buoyancy forces drive extension (Jones et al., 1996; England and Jackson, 1989), although recent global positioning system data used to assess present strain rates across the Basin and Range seem to suggest that external forces from motion of the Pacific and Sierra Nevada tectonic plates also play a role in driving deformation (Thatcher et al., 1999). As concluded in the Revision 2 of the SAR (Private Fuel Storage Limited Liability Company, 2000, Appendix 2D), the site in Skull Valley is presently affected by active tectonic extensional strain and, therefore, will be subjected to future seismicity and deformation.

Historical Seismicity

Geomatrix Consultants, Inc. (1999a) used the earthquake catalog compiled by the University of Utah, which includes historical earthquakes from about 1850 to 1962 and instrument recorded earthquakes from the University of Utah network of 26 statewide stations from 1962 to 1996. The compiled catalog was filtered by Arabasz et al. (1989) to remove duplicates and manmade events such as quarry and mining blasts. All magnitudes were also converted by Arabasz et al. (1989) to a common magnitude scale. Foreshocks and aftershocks were removed following the methodology of Youngs et al., (1987). The largest earthquake in the catalog is the 1909 M 6.0 event. Seismicity is generally concentrated along the Wasatch Front east of the site and in the Central Nevada Belt west of the site.

Because the reporting techniques improved through time, the catalog was incomplete; small magnitude events below about M 5.0 are absent from the record until primitive instruments became available in the early 1930s. As instrumentation improved, the record of smaller and smaller earthquakes became more complete. Completeness of the catalog for different magnitude scales was assessed using the methodology recommended by Stepp (1972) and reported in Youngs et al. (1987). The maximum likelihood technique (Weichert, 1980) was used by Geomatrix Consultants, Inc. (1999a) to derive recurrence parameters.

The staff reviewed the information provided by the applicant and evaluated the applicant's analyses of historical seismicity. The staff found no evidence of historic seismicity in the vicinity of the site. The staff believes that the analyses and information in the SAR provide reasonable assurance that an adequate set of data was used in developing seismic recurrence relationships and determining the maximum earthquake potential in the hazard analyses.

Potential Seismic Sources and Their Characteristics

The seismic source characterization of the Facility was developed from examination of the available literature integrated with detailed site geological studies, including site stratigraphy, geologic mapping, cross-sectional construction, and geophysical investigations (Geomatrix Consultants, Inc.,1999a; Bay Geophysical Associates, Inc., 1999). The most important aspects for the evaluations of seismic hazards were identification and characterization of active faults derived from paleoseismic and geophysical investigations. Identification of a detailed Quaternary stratigraphy was also essential because it provided critical constraints on faulting activity. Based on detailed site investigations and review of the seismotectonic setting, Geomatrix Consultants, Inc. (1999a) identified 29 fault sources and 4 areal sources. A logic tree approach was used to combine alternative models of source geometry, activity, and seismicity to formulate the PSHA.

The staff reviewed the seismic source characterization and found it acceptable because it is thorough, complete, and conservative. Models used by the applicant for the hazard assessment were appropriate. For example, Geomatrix Consultants, Inc. (1999a) conservatively considered all faults to be planar and to extend through the thickness of the brittle crust rather than considering the possibility that the primary faults could be listric and sole into a seismic detachment above the base of the seismogenic crust. Uncertainties in other aspects of fault geometry and seismic activity were incorporated into the probabilistic assessment. Upper ranges of those parameters that describe fault geometry or seismic activity were constructed to adequately bound geologic and geophysical observations. The historic seismic record was appropriately used to develop b-values for recurrence relationships and to develop the background areal source zone.

One aspect of the staff review included the interpretations of fault geometries for newly discovered East and West faults in Skull Valley based on reflection seismic data and forward modeling of gravity data in Geomatrix Consultants, Inc. (1999a). Staff review of the alternative models shows that the Geomatrix Consultants, Inc. assessment may have led to an overly conservative hazard result. Reanalysis (Stamatakos et al., 1999) of the proprietary industry gravity data does not support the interpretation that the West fault is an independent seismic source. Rather, the staff interprets the West fault as a splay of the East fault, incapable of independently generating large magnitude earthquakes. Therefore, the staff found the probabilistic assessment provided by Geomatrix Consultants, Inc. (1999a) to be acceptable, albeit conservative because the Geomatrix Consultants, Inc. (1999a) model considers the West fault as an active seismic source.

The conservative nature of the applicant's source characterization and PSHA results presented in the SAR is evident when the results are compared to PSHA results for other sites in Utah, especially those in and around Salt Lake City. Such a comparison shows that the seismic hazard in Skull Valley was calculated by the applicant to be higher than seismic hazard assessments that have been performed for sites at, or near, Salt Lake City, despite the fact that fault sources near Salt Lake City are larger and more active than fault sources near the PFS site. For example, the results of the applicant's PSHA for Skull Valley (Geomatrix Consultants, Inc., 2001a) suggest that it is 1.5 times more likely that a ground motion of 0.5g horizontal peak ground acceleration or greater will be exceeded at the PFS site (assuming hard rock site conditions), than at Salt Lake City, based on the USGS National Earthquake Hazard Reduction Program (Frankel et al., 1997). Similarly, the 2000-yr horizontal peak ground acceleration for Skull Valley (soil hazard) as estimated by the applicant, is higher than the 2500-yr ground motions for the nine sites along the Wasatch Front that were evaluated as part of the Utah Department of Transportation I-15 Reconstruction Project (Dames & Moore, Inc., 1996). The ground motions estimated by the applicant in Skull Valley are higher than those for the I-15 corridor, despite the close proximity of Salt Lake City to the Wasatch fault, which has a slip rate nearly ten times larger than the Stansbury or East Faults (cf., Martinez et al., 1998; Geomatrix Consultants, Inc., 1999a) and is capable of producing significantly larger magnitude earthquakes than the faults near the PFS Facility site in Skull Valley (cf., Machette et al., 1991; Geomatrix Consultants, Inc., 1999a).

Slip Tendency

Another aspect of the seismic source characterization that appears to be conservative, is the site-to-source models used in the ground motion attenuation relationships and the development of distributions of maximum earthquake magnitude based on the dimensions of fault rupture. This conclusion of additional conservatism is derived from a slip tendency analysis of the Skull Valley fault systems performed by the staff.

A slip tendency analysis (Morris et al., 1996) was completed using an interactive stress analysis program (3DStress™) that assesses potential fault activity relative to crustal stress. For Skull Valley, the stress tensor is defined with a vertical maximum principal stress (σ_1), a horizontal intermediate principal stress (σ_2) with azimuth of 355°, and a horizontal minimum principal stress (σ_3) with an azimuth of 085°. The stress magnitude ratios are $\sigma_1/\sigma_3 = 3.50$ and $\sigma_1/\sigma_2 = 1.56$. This orientation for the principal stresses was based on recent global positioning satellite information (Martinez, et al., 1998a). The slip tendency analysis assumed a normal-faulting regime, with rock density equal to 2.7 g/cc, fault dip equal to 60°, water table at a depth of 40 m, and a hydrostatic fluid pressure gradient.

In slip tendency analysis, the underlying assumption is that the regional stress state controls slip tendency and that there are no significant deviations due to local perturbations of the stress conditions. This assumption is supported by a similar slip tendency analysis of the Wasatch fault, which shows highest slip tendency values for the segments of the fault considered to be most active (Machette et al., 1991).

The slip tendency analysis shows that segments of the East fault and the East Cedar Mountain fault nearest the PFS site have relatively low slip tendency values compared to segments farther north in Skull Valley. As discussed in the following sections on site-to-source distances and maximum magnitudes, these results indicate that the seismic source characterization of the PSHA study conducted by Geomatrix Consultants, Inc. (1999a, and 2001a) is conservative. Three areas of conservatism are the distribution of site-to-source distance, maximum magnitude earthquakes, and potential of the West fault as a seismogenic source (discussed in Stamatakos et al., 1999).

Distributions of Site-to-Source Distances

Results of the slip tendency analysis indicate that fault segments with approximately North-South strikes (azimuth = 175°) are optimally oriented for future fault slip. Faults with north northeast-south southwest strikes have high slip tendency values. In contrast, fault segments with northwest-southeast strikes, such as the East fault near the PFS Facility site and the southern segments of the East Cedar Mountain fault also near the PFS Facility site, have relatively low slip tendency values. Therefore, these fault segments are less likely to slip in the future than fault segments further from the site. Fault rupture close to the site greatly influence the seismic hazard. The closer the earthquake is to the site, the larger the resulting ground motions compared to an equal magnitude earthquake on a fault segment farther away from the site.

In the site-to-source distributions used in the ground motion attenuation equations, Geomatrix Consultants, Inc. (1999a) assumed uniform distributions of earthquake ruptures along active fault segments. Given the slip tendency analysis described above, this assumption by Geomatrix Consultants, Inc. (1999a) is conservative. The staff concludes that seismic source models that incorporate slip tendency would result in a lower ground motion hazard than the one developed by the applicant.

Maximum Magnitude

The slip tendency results suggest that Geomatrix Consultants, Inc. (1999a) may have overestimated the maximum magnitude of the East and East Cedar Mountain faults near the PSFS site. In the SAR, the applicant first developed conceptual models of the physical dimensions of fault rupture—either rupture area or trace length of surface fault rupture—based on the geologic record (Geomatrix Consultants, Inc.,1999a). Second, the applicant developed distributions of maximum magnitudes for each active fault using empirical scaling relationships developed from the magnitudes and associated rupture dimensions of historical earthquakes (e.g., Wells and Coppersmith,1994). In developing the fault segment models, the applicant conservatively assumed that the entire mapped length of the surface trace length represents active fault segments. Thus, these maximum fault dimensions produce conservative estimates of maximum magnitude.

The slip tendency analysis indicates that parts of the East and East Cedar Mountain faults near the PFS Facility site have relatively low slip tendency values. Thus, these faults may be smaller

than in the fault models used by the applicant to estimate maximum magnitude. Fault rupture models developed using slip tendency analysis would therefore lead to fault segment models with smaller rupture dimensions (length or area) than those used by Geomatrix Consultants, Inc. (1999a). Because distributions of maximum magnitude for each active fault are derived from empirical scaling relationships of rupture area or rupture length (e.g., Wells and Coppersmith,1994), application of the slip tendency analysis would thereby result in smaller predicted maximum magnitudes than those developed by the applicant. Smaller maximum magnitudes would reduce the overall ground motion hazard.

In summary, the staff found that the applicant's considerations of seismic source characteristics and associated uncertainties provide reasonable assurance that all significant sources of future seismic activity have been identified and their characteristics and associated uncertainties are adequately or conservatively described and appropriately included in the evaluation of the seismic ground motion hazard. Stamatakos et al. (1999) provides more details of PFS's seismic source characterization and the staff's independent sensitivity analyses.

Further, the staff concludes that the seismic source characterization performed by the applicant is conservative (perhaps by as much 50% or more based on a comparison to Salt Lake City PSHA results). The staff does not attempt here to explicitly quantify the degree of conservatism in the seismic source characterization. Quantitative estimates of the degree of conservatism would require the staff to essentially recalculate the PFS PSHA, which is not necessary under the NRC Standard Review Plan (1997a). Nevertheless, this qualitative assessment of potential conservatism provides additional confidence that the applicant's seismic source characterization is acceptable. Because the applicant's seismic source characterization is conservative, it provides reasonable assurance that the seismic hazard has been adequately determined and is sufficient to assess safety of the PFS Facility. Therefore, the staff concludes that the information presented in Section 2.6.1.1 of the SAR is acceptable because the basic geologic and seismic characteristics of the site and vicinity have been adequately (albeit conservatively) described in detail to allow investigation of seismic characteristics of the proposed Facility site. The staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Facility, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR 72.92(a), 72.92(b), 72.102(e), and 72.122(b) with respect to this subject.

Estimate of Ground Motion Attenuation

Yucca Mountain Approach

For purposes of estimating earthquake ground motions that may occur at the proposed site, the applicant utilized results of the PSHA conducted for the proposed high-level waste repository site at Yucca Mountain (Civilian Radioactive Waste Management System Management and Operating Contractor, 1998). The Yucca Mountain study developed and implemented a methodology for evaluating earthquake ground motions in the Basin and Range that includes the results of scientific evaluations and expert elicitations from seven ground motion experts. The

staff found that the use of the Yucca Mountain methodology for the Facility PSHA ground motion analysis is appropriate, in general, because (i) it represents the state-of-the-art knowledge and (ii) both the PFS Facility site and site of the proposed geologic repository at Yucca Mountain have seismotectonic characteristics of the Basin and Range.

Geomatrix Consultants, Inc. (1999a) selected the published median ground motion attenuation models and weighted them according to the Yucca Mountain Seismic Hazard Study (Civilian Radioactive Waste Management System Management and Operating Contractor, 1998).

The Yucca Mountain PSHA used a sophisticated methodology for modeling and quantifying the epistemic uncertainty in ground motions. The Yucca Mountain analysis attempted to quantify all of the sources of uncertainty involved in the estimation of strong ground motion. As part of the Facility PSHA, Geomatrix Consultants, Inc. (1999a) elected to consider only that part of the epistemic uncertainty associated with the choice of different median ground motion models and not the uncertainty in the models themselves. As a consequence, sources of epistemic uncertainty that were quantified in the Yucca Mountain PSHA were not considered in the PFS Facility analysis. This leads to an underestimate of the total epistemic uncertainty and, therefore, an underestimate of the mean seismic hazard at the site. The staff performed sensitivity calculations and determined that the mean frequency of exceedance of ground motions changes by less than a factor of two. Therefore, the staff concludes this effect to be insignificant.

Revisions to the Ground Motion Modeling in 2001

In March 2001, Geomatrix Consultants, Inc. (2001a) published the revised probabilistic seismic hazard analysis result for the PFS Facility. The revision was motivated by the analysis of site-specific soils and velocity data obtained subsequent to the submittal of the initial PSHA results (Geomatrix Consultants, Inc., 1999a). In particular, the applicant provided additional shear wave velocity measurements of the upper 106.5 ft of strata in the soil column at the PFS Facility site in the SAR. The additional data were acquired from downhole geophysical measurements in two borings (Northland Geophysical Limited Liability Company, 2001) and 16 test pits excavated at the site. The applicant used the results to derive alternative interpretations of the shear wave velocity profiles that were used to develop site response models Calculation G(PO18)-2 of Parkyn, 2001.

The applicant provided revised dynamic properties of the soil strata above 106.5 feet in the SAR (Private Fuel Storage Limited Liability Company, 2001). Parkyn (2001) documents several changes in dynamic soil properties compared to those reported in the former revision of the SAR (Private Fuel Storage Limited Liability Company, 2000). These changes include:

(1) Small adjustment of the depths of the boundaries of several layers including the two prominent soil horizons.

(2) Incorporation of the downhole shear-wave velocity measurements from two boreholes, CTB-5(OW) and CTB-5A (Northland Geophysical. L.L.C., 2001).

(3) Alternative multi-step methodology to develop statistical models of shear wave velocity profiles from the 16 cone penetrometer tests and the CTB-5(OW) and CTB-5A borehole data.

(4) Direct measurement of shear wave velocities in the upper layers of the Tertiary Salt Lake Group strata, which lies just below the Quaternary-Tertiary unconformity.

(5) Revision of site response to include lower damping and lower levels of modulus reduction based on results of the resonant column tests leading to a more linear modulus reduction and damping relationship.

These revisions led to development of a nine-layer shear-wave soil profile used to calculate the site response. This change in the shear-wave profile and site response model led to a significant increase in estimated ground motions at the PFS site. As shown in Table 2-2, these changes significantly affect higher frequencies, but have much less effect on lower frequencies of ground motion (Appendix F of Geomatrix Consultants, Inc., 2001a).

Based on the new site velocity data, Geomatrix Consultants, Inc. (2001a) made several revisions to its assessment of the ground motions at the PFS site. These revisions included modifications to the site velocity model, the ground motion attenuation relationships adopted from the Yucca Mountain study, and the approach used in the site response analysis. In the aggregate, these changes resulted in an increase in the ground motion hazards estimated at the PFS site. Table 2-2 compares the estimated 2000-year PSHA accelerations as estimated in Revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000) and the updated 2000-year PSHA accelerations in Revision 22 of the SAR, for horizontal and vertical ground motions at selected periods.

Table 2-2. Comparison of PSHA for 2,000-Year Return Period Spectral Acceleration (with 5% Damping)

Period (sec)	Horizontal Ground Motion (g)		Vertical Ground Motion (g)	
	SAR Revision 22	SAR Revision 18 (former design)	SAR Revision 22	SAR Revision 18 (former design)
PGA	0.711	0.528	0.695	0.533
0.1	1.541	1.046	1.752	1.369
0.5	1.045	1.166	0.509	0.476
2.0	0.164	0.272	0.088	0.088

The process used to estimate the ground motion at the PFS site in the original PSHA (SAR Revision 18, Private Fuel Storage Limited Liability Company, 2000), as well as in the revised analysis, consisted of the following elements.

- Median Ground Motion Attenuation Models – the ground motion models used in the Yucca Mountain study (Civilian Radioactive Waste Management System Management and Operating Contractor, 1998) were adopted in the PFS analysis to define the median ground motion and the epistemic uncertainty in the median, as a function of earthquake magnitude and distance. These are empirical models derived from ground motions recorded principally in California.

- Faulting Type – an adjustment factor was used to account for differences in the type of faulting between faults in California and Skull Valley. This adjustment factor was used to scale the California median ground motion attenuation models.

- Regional Attenuation – an anelastic attenuation model was used to remove the effects of regional attenuation of seismic waves in the crust in California and to account for the regional attenuation as it would be expected to occur in Utah.

- Site-Specific Response – the effects of California surficial materials were removed and the response of the PFS soils were computed and incorporated in the analysis to model the response of the near surface geologic deposits on ground motion at the PFS site.

- Near-Source Effects – adjustment factors were used to account for the near-source effects of faulting kinematics on ground motions at the PFS site. While the elements of the process of developing site-specific ground motion estimates for the PFS site were the same in the original PSHA and in the revised analysis,

there are differences in the implementation of two of the four elements. Table 2-3 tabulates the elements of the ground motion model and how they were implemented in Revisions 18 and 22 of the SAR.

The revised PSHA used the same adjustment factors for the effects of faulting type, regional attenuation, and near-source effects. However, the median ground motion attenuation models and the evaluation of site response changed in the revision of the PSHA.

In the original PSHA, Geomatrix Consultants, Inc. (1999a) used a set of California empirical ground motion models applicable to soil sites. These models were the companion empirical models to the rock attenuation models selected by the Yucca Mountain study experts. The choice to use soil ground motion attenuation models as a starting point was based on the original observation that the PFS velocity profile compared favorably with California soil sites (Geomatrix Consultants, Inc., Appendix F, 1999a). Following revision of the soil profile data, Geomatrix Consultants, Inc. (2001a) concluded that the PFS velocity profiles now compared more favorably with California rock sites rather than California soil sites. On this basis, the California empirical rock ground motion attenuation models selected by the Yucca Mountain study experts were chosen. A total of 20 rock horizontal attenuation models with associated probability weights were used in the PFS analysis. For the vertical motions, 11 models were used. The model weights were derived from the weights assigned by the ground motion experts that participated in the Yucca Mountain study. Based on a review of the current site data, the staff agrees that the PFS site conditions compare more favorably with the California rock site conditions. Further, the staff notes that the process used in the Yucca Mountain study and in the PFS analysis is designed to remove the California regional and site-specific effects that are inherent in empirical ground motion attenuation models and to incorporate appropriate regional and site-specific effects for the site in question (in this case, Utah and Skull Valley).

By virtue of this modeling approach, the issue as to whether rock or soil median ground motion attenuation models should be used is not significant. The staff agrees, however, based on the current PFS site-specific information, that the use of the empirical rock attenuation models for the PFS site is reasonable.

Table 2-3. Comparison of Ground Motion Modeling and Soil Velocity Profiles

<table>
<tr><th colspan="3"></th><th>SAR Revision 22</th><th>SAR Revision 18
(former design)</th></tr>
<tr><td colspan="3">Median Ground Motion Attenuation Model</td><td>California rock models</td><td>California soil models</td></tr>
<tr><td colspan="3">Faulting-Type Effect
(Strike-slip to normal faulting)</td><td>Yucca Mountain scaling factors (re-normalized weights for rock models)</td><td>Yucca Mountain scaling factors (re-normalized weights for rock models)</td></tr>
<tr><td colspan="3">Regional Attenuation
(Crustal Path Effect)
(California motion to Utah motion)</td><td>Yucca Mountain technique</td><td>Yucca Mountain technique</td></tr>
<tr><td colspan="3">Near-Source Effects</td><td>Conservative application of Sommerville et al. (1997) factors</td><td>Conservative application of Sommerville et al. (1997) factors</td></tr>
<tr><td rowspan="6">Site Effect</td><td colspan="2">Empirical Approach</td><td>New</td><td>-</td></tr>
<tr><td rowspan="5">Modeling Approach</td><td>Input Motion</td><td>Rock recordings</td><td>Rock recordings</td></tr>
<tr><td>Soil Velocity Profile</td><td>New 9-layer model</td><td>3-layer average velocity model
1-layer average velocity model</td></tr>
<tr><td>Deconvolution</td><td>To a depth of 5 km</td><td>To a depth of 3 km</td></tr>
<tr><td>Response Analyses</td><td>PFS multilayer profiles
Western US generic rock profiles</td><td>PFS average profiles
Westurn US generic soil profiles</td></tr>
</table>

Site Response Effects

A final step in the assessment of site-specific ground motions for the PFS site requires that the response of near-surface geologic deposits be considered. The effects of site response are included in the estimates of ground motion by means of frequency (or period) dependent site-response factors. In the revision of the PFS PSHA, two approaches were used to derive the site-adjustment factors. The first approach is empirical and the second is based on site-response calculations for the PFS site soils. Geomatrix Consultants, Inc. (2001a, b) assigned

probability weights to each approach, based on their interpretation of the credibility in each method.

The empirical approach, used by Geomatrix Consultants, Inc. (2001a, b), was assigned 1/3 weight in PSHA calculation. The empirical approach is based on two assumptions: (i) the PFS site can be classified as a shallow soil site, and (ii) PFS soil velocity characteristics are similar to those of western United States shallow soil sites. In the empirical approach, a set of strong motion recordings obtained at shallow soil sites were selected. The selected ground motion recordings were scaled to the desired ground motion levels at the PFS site. A set of empirical site response factors was determined from the distribution of spectral ratios that were determined from the set of shallow site recordings and the selected empirical hard-rock ground-motion models.

The second approach used by Geomatrix Consultants, Inc. (2001a) in the revision to the PSHA involved the calculation of site response factors using the SHAKE model and the PFS site data. The same approach was used in the original analysis. Based on the results of the site soils and velocity data obtained subsequent to the original submission of the PSHA, significant modifications were made to the site model. Geomatrix Consultants, Inc. (2001a, b, c) abandoned the 3-layer average velocity model used in the original study (Geomatrix Consultants, Inc., 1999a) and developed a new 9-layer soil velocity model above the Tertiary strata. The Tertiary strata in Skull Valley are part of the Salt Lake group, which is a ~500-700 ft thick sequence of semi-consolidated siltstones, claystones, and sandstones of Middle to Late Miocene Age (5.3 to 16.6 Ma). In the 2001 revisions, Geomatrix Consultants, Inc, (2001a) used both a constant velocity model and an increasing velocity model for these Tertiary strata, whereas a 1-layer average velocity model was used in the Geomatrix Consultants, Inc. (1999a) study.

The differences between the results of the empirical and site response analyses are considerable for periods less than about 0.3 s (see Fig. F-17 in Appendix F, Geomatrix Consultants, Inc., 2001a). At these periods, the site response analysis predicts higher scaling factors. However, at periods greater than about 1.0 s, the empirical factors are higher. In its revised PSHA report (Appendix F, Geomatrix Consultants, Inc., 2001a) Geomatrix Consultants, Inc. also concludes that the use of the empirical site response scaling factors is appropriate because they are based on actual strong motion recordings at shallow soil sites. At the same time, they recognize these factors are not site-specific and thus assign a lower weight to this approach.

Staff Review of Ground Motion Attenuation Models

The staff reviewed the characterization of strong ground motion in the Facility seismic hazard analysis and the approach taken to model the epistemic uncertainty, and found them acceptable. The approach to modeling strong ground motion provides reasonable assurance that the site hazard is adequately (albeit conservatively) estimated.

The staff agrees with the applicant that revision of the dynamic soil properties presented in the SAR was necessary because of the acquisition of new velocity data (Northland Geophysical, L.L.C, 2001), which was collected by the applicant after publication of the original SER. The revision of the original 3-layer shear-wave velocity profile to the current 9-layer model led to a large increase in the peak ground accelerations. However, the revised data are well within the uncertainty bands provided in the original 3-layer model. The staff considers the overall shear wave profile results, as revised, to be acceptable and conservative. In this regard, the staff notes that incorporation of the new shear-wave velocity data from the boreholes (Northland Geophysical Limited Liability Company, 2001) into the existing shear wave velocity profiles (Private Fuel Storage Limited Liability Company, 2000) or equal weighting of the original and new statistical methodologies would lead to a site response model with lower ground motions than the model presented in Revision 22 of the SAR.

The staff also agrees with the applicant's approach to estimate regional and site-specific ground motions based on the site response calculations for PFS Soils. There are sufficient technical bases for ground motion modeling based on this approach for use in development of the site specific PSHA and ultimately in development of the design basis earthquake. In contrast, the staff finds that PFS did not provide sufficient technical basis for use of empirical site response factors. These factors are based on strong-motion recordings obtained at California sites for which no information is provided that supports a comparison to the PFS site, other than a general shallow soil site characterization. However, sensitivity results provided by PFS (Parkyn, 2001) show that inclusion of the empirical site response factors approach has a small effect on the PGA values (~12%), and an even smaller effect on the predicted ground motions at lower frequencies. The small increase in ground motions that would occur if the applicant did not use the empirical site response approach is more than compensated for by other conservatisms in the PSHA results, including the noted conservatism in the seismic source characterization.

In summary, the staff concludes that there is sufficient information on shear wave velocity profiles in the soil strata and ground motion attenuation modeling for use in other sections of the SAR to develop the design bases of the proposed Facility, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR 72.90(b-d), 72.92(a-c), 72.98(b), 72.98(c)(3), and 72.122(b) with respect to this issue.

Probabilistic Seismic Ground Motion Hazard

The Geomatrix Consultants, Inc. (1999a) PSHA uses a well-established methodology and basic equations (e.g., Cornell, 1968, 1971; McGuire, 1976, 1978; Civilian Radioactive Waste Management System Management and Operating Contractor, 1998). Calculation of probabilistic seismic ground motion hazard requires specification of three basic inputs: (i) geometric characteristics of potential sources, (ii) earthquake recurrence characteristics for each potential source, and (iii) ground motion attenuation estimates. Details of these inputs to the PSHA at Skull Valley have been evaluated in Stamatakos et al. (1999) and summarized in previous sections of this SER. PSHA calculations include the seismic hazard from each individual source and the total hazard from all potential sources. Such calculations establish hazard curves that

depict the relationship between levels of ground motion and probabilities (frequencies) at which the levels of ground motion are exceeded. In Geomatrix Consultants, Inc. (1999a) computations, fault sources were modeled as segmented planar surfaces. Areal sources were modeled as a set of closely spaced parallel fault planes occupying the source regions. The distance density functions were computed assuming that a rectangular rupture area for a given size earthquake is uniformly distributed along the length of the fault plane and located at a random point on the fault plane. Depth distribution for earthquakes was based on depth distribution of recorded historical earthquakes along the Wasatch Front. The rupture size (mean rupture area) of an event was estimated based on the empirical relation of Wells and Coppersmith (1994). The basis for using the mean rupture area is the study of Bender (1984) that shows nearly equal hazard results using the mean estimates of rupture size and considering statistical uncertainty in rupture size. The minimum earthquake magnitude considered in the Geomatrix PSHA was M 5 (Geomatrix Consultants, Inc., 1999a).

Mean and percentile (95, 85, 50, 15, and 5th) peak ground motion and 1-Hz spectral (5-percent damped) acceleration hazard curves were calculated and presented in Geomatrix Consultants, Inc. (1999a) for horizontal and vertical motions. In Revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000), the mean peak horizontal accelerations were 0.40g and 0.53g and the mean peak vertical accelerations were 0.39g and 0.53g for 1,000- and 2,000-year return periods, respectively. Equal-hazard response spectra for return periods of 1,000 and 2,000 year (mean annual probabilities of exceedance of 1×10^{-3} and 5×10^{-4}, respectively) were calculated and presented in Geomatrix Consultants, Inc. (1999c). In Revision 22 of the SAR (Private Fuel Storage Limited Liability Company, 2001), mean peak horizontal and vertical accelerations for the 2000-yr return period were calculated to be 0.711g and 0.695g, respectively.

Contributions of individual seismic sources were calculated and the results show that the dominating sources are the Stansbury, East-Springline, and East Cedar Mountain faults for peak ground acceleration for return periods greater than 1,000 years and for 1-Hz spectral acceleration for a return period greater than 2,000 years. Deaggregation results show that the total hazard is dominated by ground motions from nearby M 6 to 7 events. Sensitivity results indicate that the choice of attenuation relationship is a major contributor to uncertainty in the hazard calculation. Geomatrix Consultants, Inc. (1999a) sensitivity results also indicate (i) alternative models for the geometry and extent of the West fault have little effect on the total hazard because the East fault dominates the hazard from the Skull Valley faults as a result of its higher estimated slip rate, and the alternative models for the West fault have only minor effects on the parameters of the East fault, (ii) the West fault, considered as an independent source or as a secondary feature, has a minimal influence on the hazard, and (iii) the East and Springline faults, combined as a single source, produces slightly higher hazard at low probabilities of exceedance and for longer period motions than separating them as individual fault sources. The Geomatrix Consultants, Inc. (1999a) summary of contributions to the uncertainty in the total hazard at the proposed Skull Valley site for a return period of 2,000 years shows that the major contributors to the total uncertainty in the hazard are the selection of attenuation relationships, assessment of maximum magnitude, recurrence rate, and magnitude distribution.

Deterministic Seismic Ground Motion Hazard

Site-specific deterministic ground motion hazard for the Facility was assessed by Geomatrix Consultants, Inc. (1997), in which two potentially capable fault sources were identified to be within 7 miles of the site—the East Cedar Mountain and Stansbury faults. Their closest distances to the site were estimated to be about 6 miles to the Stansbury fault and 5.5 miles to the East Cedar Mountain fault. The potential for a random nearby earthquake was considered by including an areal source within 16 miles of the site. Maximum earthquake magnitudes for the two fault sources were estimated using empirical relationships of Wells and Coppersmith (1994) and Anderson et al. (1996) based on estimated maximum rupture dimensions (rupture length and rupture area). The resulting mean estimates of maximum magnitudes are M 7.0 for the Stansbury fault and M 6.8 for the East Cedar Mountain fault. The maximum magnitude for the areal source was estimated to range from M 5.5 to 6.5, with a mean value of 6, based on the Wells and Coppersmith (1993) study on the relationship between earthquake magnitude and the occurrence of associated surface faulting and the assumption that these random earthquakes do not produce significant surface faulting. A mixture of attenuation relationships for strike-slip faults in California and for extensional stress regimes were used to account for uncertainties. These include Abrahamson and Silva (1997), Campbell (1997), Sadigh et al. (1993, 1997), Idriss (1991), and Spudich et al. (1997). In the Geomatrix DSHA, uncertainties were included for maximum magnitude, minimum source-to-site distance, and the selection of attenuation relationships. The recommended 84^{th}-percentile peak ground accelerations were calculated to be 0.67g in the horizontal direction and 0.69g in the vertical direction. These accelerations envelop the calculated accelerations for a rock site and a deep soil site.

The Geomatrix Consultants, Inc. (1999b) DSHA considers the two new faults (i.e., the East and West faults) near the proposed site and in-depth characterization of other capable faults. The detailed characteristics of the two new faults as well as other fault sources are reviewed in Stamatakos et al. (1999). In its updated DSHA, Geomatrix Consultants, Inc. (1999b) considered four nearby fault sources—the Stansbury, East, West, and East Cedar Mountains faults. The mean maximum magnitudes of these fault sources were estimated to be M 7.0, 6.5, 6.4, and 6.5, respectively, based on distributions for maximum magnitude of each source developed in Geomatrix Consultants, Inc. (1999a). The closest distances to the Canister Transfer Building from the surface traces of these faults were estimated to be 9, 0.9, 2.0, and 9 km, respectively. The ground motion models used in the updated DSHA were the set of 17 horizontal and 7 vertical attenuation relationships used in the PSHA (Geomatrix Consultants, Inc., 1999a). These relationships were reviewed and discussed in Stamatakos et al. (1999). The ground motion attenuation relationships were adjusted for near-source effects using the empirical model developed by Somerville et al. (1997). The updated DSHA results in 2000 showed that the ground motion from the East fault generally envelops those from the other sources. In Revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000), the 84^{th}-percentile peak ground accelerations for the East fault were calculated to be 0.72g in the horizontal direction and 0.80g in the vertical direction. When compared with the PSHA results in Revision 18 of the SAR, the controlling deterministic spectra generally were between the 5,000- and 10,000-year return period equal-hazard response spectra. In revision 22 of the SAR (Private Fuel Storage

Limited Liability Company, 2001), the 84th percentile peak ground accelerations for the East fault were calculated to be 1.15g in the horizontal direction and 1.17g in the vertical direction (Geomatrix Consultants Inc., 2001d). As in revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000), the revised controlling deterministic spectra (Geomatrix Consultants Inc., 2001d) in revision 22 of the SAR generally fall between the 5,000-yr and 10,000-yr return period equal-hazard response spectra.

Design-Basis Ground Motion

The design ground motion response spectra for the proposed Skull Valley site were developed by Geomatrix Consultants, Inc. (2001b) based on its site-specific PSHA results as reviewed in this SER and Stamatakos et al. (1999) and documented in detail in Geomatrix Consultants, Inc. (1999a, 2001a). The Geomatrix Consultants, Inc. development of design spectra is based on the procedures outlined in Regulatory Guide 1.165 (Nuclear Regulatory Commission, 1997c) and incorporates near-source effects.

The assessment of design ground motions for the Facility is described in Geomatrix Consultants, Inc. (2001b). The design ground motions were determined using the procedure described in Regulatory Guide 1.165 (Nuclear Regulatory Commission, 1997c). However, prior to implementing the Regulatory Guide 1.165 procedure, the site seismic hazard results were modified to account for the near-source effects of rupture directivity and the polarization of ground motions. Adjustments to the PSHA results that account for these effects were made using empirical models developed by Somerville et al. (1997). Based on its review, the staff determined that the deterministic approach of shifting the seismic hazard results to account for rupture directivity and ground motion directional effects is conservative for the frequencies to which these adjustments were applied. Based on the results of Somerville et al. (1997), adjustments were not made for the peak ground acceleration seismic hazard results or for spectral accelerations greater than 1.0 Hz. There is empirical evidence that suggests peak ground accelerations and high frequency ground motions may also be influenced by rupture directivity and source radiation. In addition, there is limited empirical evidence to verify the Somerville et al. (1997) model and to predict, in an absolute sense, the systematic effect of rupture directivity on strong ground motion. However, as discussed in Stamatakos et al. (1999) and Geomatrix Consultants, Inc. (1999c), the random effects of rupture directivity are accounted for as part of the aleatory variability in ground motion. Therefore, it is an effect that is accounted for in the PSHA. In fact, for frequencies less than 1.0 Hz, these effects are double counted in the Facility estimate of design motions.

The Regulatory Guide 1.165 process for determining design basis ground motion spectra involves computing the contributions to the total hazard at the specified design return period (or reference probability) from events in discrete magnitude and distance bins. In the Geomatrix Consultants, Inc. (1999c) calculation, a magnitude bin size of 0.25 was selected. The distance bin size increases gradually from 3 to 32 miles as the source-to-site distance increases from 0 to 150 km. From these contributions and the average magnitude and distance for each bin, a weighted average magnitude, $\overline{M}$, and log average distance, $\overline{D}$, of the events contributing to the

design level hazard were determined for spectral frequency ranges of 5–10 Hz and 1–2.5 Hz. Free-field ground surface response spectral shapes were developed using the 84^{th}-percentile peak acceleration and the 84^{th}-percentile response spectra for each of the $\overline{M}$ and $\overline{D}$ pairs using a weighted combination of the same ground motion attenuation relationships used for the PSHA (Geomatrix Consultants, Inc., 1999a). These response spectral shapes were scaled to the appropriate equal hazard spectra. Design ground motion response spectra were defined to be the envelope of the scaled spectra and equal hazard spectra. This envelope was further scaled by the adjustment factors for near-fault effect as described in Stamatakos et al. (1999). The final response spectra can be found in Geomatrix Consultants, Inc. (2001b). In Revision 18 of the SAR (Private Fuel Storage Limited Liability Company, 2000),these studies resulted in the following design ground motion accelerations: (1) for a 1,000-year return period earthquake, a peak horizontal acceleration of 0.40 g and a peak vertical acceleration of 0.39 g; and (2) for a 2,000-year return period earthquake, a peak horizontal acceleration of 0.53 g and a peak vertical acceleration of 0.53 g for a 2,000-year return period. In Revision 22 of the SAR (Private Fuel Storage Limited Liability Company, 2001), mean peak horizontal and vertical accelerations for the 2000-yr return period were calculated to be 0.711 g and 0.695 g respectively.

The applicant's exemption request specified a 1,000-year return period to calculate design basis ground motions with the PSHA methodology. The applicant (Parkyn, 1999b) stated (i) a 1,000-year return period is the same as that selected by the U.S. Department of Energy (1997) for preclosure seismic design of important to safety structures, systems, and components for NRC Frequency Category 1 design basis events at the proposed Yucca Mountain high-level waste geologic repository, and (ii) the consequences of a major seismic event at the Facility can be bounded using the HI-STORM 100 system technology and are limited to a storage cask-tipover event, which would result in a dose below regulatory limits. A Frequency Category 1 design basis ground motion refers to a mean recurrence interval of 1,000 years and a Frequency Category 2 design basis ground motion refers to a mean recurrence interval of 10,000 years. As discussed below, the staff has determined that a 2000-year return period is the appropriate value for the PFS Facility site.

Staff Review of Ground Vibration and Request for Exemption to 10 CFR 72.102(f)(1)

The staff found the applicant's seismic hazard results to be conservative, based on the review of geological and seismotectonic setting, historical seismicity, potential seismic sources and its characteristics, estimate of ground attenuation, estimates of probabilistic and deterministic ground motion hazards, development of design basis ground motion, and independent staff analyses. The staff also found that in the application:

- Seismic events that could potentially affect the site were identified and the potential effects on safety and design were adequately assessed.

- Records of the occurrence and severity of historical and paleoseismic earthquakes were collected for the region and evaluated for reliability, accuracy, and completeness.

- Appropriate methods were adopted for evaluations of the design basis vibratory ground motion from earthquakes based on site characteristics and current state of knowledge.

- Seismicity was evaluated by techniques of 10 CFR Part 100, Appendix A. Seismic hazard, however, was evaluated using a probabilistic approach as stated in the Request for an Exemption to 10 CFR 72.102(f)(1).

- Liquefaction potential or other soil instability from vibratory ground motions was appropriately evaluated.

- The design earthquake has a value for the horizontal ground motion greater than 0.10g with the appropriate response spectrum.

- The applicant's considerations with respect to the approach taken to model the epistemic uncertainty in ground motions and near-source effects are adequate.

- As discussed in Stamatakos et al. (1999), the applicant adequately applied adjustment factors for the near-fault effect using the state-of-the-art techniques and applied procedures described in Regulatory Guide 1.165 (Nuclear Regulatory Commission, 1997c) for developing design-basis ground motion. The associated response spectra and design basis motion levels are adequate.

The staff reviewed the applicant's exemption request to use the PSHA methodology with a 1,000-year return period value by evaluating the technical basis of the PSHA methodology and its use in other Title 10 regulations regarding nuclear facilities and materials. Although 10 CFR Part 72 requires a deterministic approach for the seismic design of an ISFSI site west of the Rocky Mountain Front, a probabilistic approach for seismic design is acceptable by the 1997 amendments to 10 CFR Parts 50 and 100 that apply to new nuclear power plants, and 10 CFR Part 60 that applies to the disposal of high-level waste in geologic repositories. Also, the NRC issued Regulatory Guide 1.165 to provide guidance on PSHA methodology (Nuclear Regulatory Commission, 1997c). In addition, NRC has reviewed and approved the Request for Exemption to 10 CFR 72.102(f)(1) seismic design requirements to allow seismic design using PSHA results of 2,000-year return period earthquakes for the Three Mile Island Unit-2 (TMI-2) ISFSI (Nuclear Regulatory Commission, 1998b; Chen and Chowdhury, 1998). DSHA considers only the most significant earthquake sources and events with a fixed site-to-source distance. PSHA, on the other hand, considers contributions from all potential seismic sources and integrates across a range of source-to-site distances and magnitudes. Furthermore, DSHA is a time-independent statement, whereas PSHA estimates the likelihood of earthquake ground motion occurring at the location of interest within the time frame of interest. The staff concludes that there are sufficient regulatory and technical bases to accept the PSHA methodology for seismic design of the Facility.

The design basis ground motion for a particular structure, system, and component depends on the importance of that particular structure, system, and component to safety. As described in the NRC rulemaking plan for 10 CFR Part 72 (Nuclear Regulatory Commission, 1998a), an individual structure, system, and component may be designed to withstand only Frequency Category 1 events (1,000-year return period) if the applicant's analysis provides reasonable assurance that the failure of the structure, system, and component will not cause the Facility to exceed the radiological requirements of 10 CFR 72.104(a). If the applicant's analysis cannot support this conclusion, then the designated structures, systems, and component should have a higher importance to safety, and the structures, systems, and component should be designed such that the Facility can withstand Frequency Category 2 events (10,000-year return period).

The staff reviewed the applicant's request and supporting analysis to use the 1,000-year return period value and does not find this value acceptable because of the following reasons: (i) the DOE classification of Yucca Mountain proposed high-level waste geologic repository structures, systems, and components to design for Frequency Category 1 and Frequency Category 2 events as it applies to the proposed Yucca Mountain repository has not been reviewed or accepted by the NRC staff; (ii) the applicant has provided no technical basis for classifying all the important to safety structures, systems, and components for the Facility as those that could be designed for NRC Frequency Category 1 design basis events; and (iii) the consequence analysis using the HI-STORM 100 systems technologies includes only a single accident scenario (i.e., cask tipover) that is independent of ground motion level. The applicant did not demonstrate that the cask-tipover event envelops other unanalyzed conditions such as the effect of collapse of the Canister Transfer Building on canisters or the effects of sliding and bearing failures of the foundation and concrete pad on storage casks.

However, the staff has determined that a 2,000-year return value with the PSHA methodology can be acceptable for the following reasons:

- The radiological hazard posed by a dry cask storage facility is inherently lower and the Facility is less vulnerable to earthquake-induced accidents than operating commercial nuclear power plants (Hossain et al., 1997). In its Statement of Consideration accompanying the rulemaking for 10 CFR Part 72, the NRC recognized the reduced radiological hazard associated with dry cask storage facilities and stated that the seismic design basis ground motions for these facilities need not be as high as for commercial nuclear power plants (45 FR 74697, 11/12/80; SECY-98-071; SECY-98-126).

- Seismic design for commercial nuclear power plants is based on a determination of the Safe Shutdown Earthquake ground motion. This ground motion is determined with respect to a reference probability level of 10^{-5} (median annual probability of exceedance) as estimated in a probabilistic seismic hazard analysis (Reference Reg Guide 1.165). The reference probability, which is defined in terms of the median probability of exceedance, corresponds to a mean annual probability of exceedance of 10^{-4} (Murphy et al., 1997). That is, the same design

ground motion (which has a median reference probability of 10^{-5}) has a mean annual probability of exceedance of 10^{-4} . Further, analyses of nuclear power plants in the western United States show that the estimated average mean annual probability of exceeding the safe shutdown earthquake is 2.0×10^{-4} (U.S. Department of Energy, 1997).

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

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

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

In summary, the staff agrees that the use of the PSHA methodology is acceptable. A 2,000-year return period is acceptable for the seismic design of the PFS Facility. As discussed in the subsequent chapters of this SER, the design analyses use a spectrum that envelops the 2,000-year return period uniform hazard spectra.

Additional Information on the East Great Salt Lake Fault

The staff reviewed additional information and analyses provided in Appendix 2G of the SAR (Private Fuel Storage Limited Liability Company, 2000) regarding reported fault characterization data for the East Great Salt Lake fault. Recent high-resolution seismic data collected from the Great Salt Lake and reported in Dinter and Pechmann (1999a,b) indicate a Holocene vertical slip rate for the East Great Salt Lake fault of 1 mm/yr (average recurrence period of 3000–6000 years). The applicant assessed the possibility of the East Great Salt Lake fault being linked with the Oquirrh fault and also with the Topliff-Hill and Mercur faults, which collectively could form a Wasatch-scale fault zone.

The applicant showed in Appendix 2G, that the information about slip in the East Great Salt Lake fault does not significantly change the existing PSHA given in Geomatrix Consultants, Inc. (1999a). The applicant reiterated that the possibility of a linked East Great Salt Lake-Oquirrh fault was already accounted for in the existing PSHA analyses. In the existing PSHA model, the mean slip rate for the East Great Salt Lake fault was 0.38 mm/yr. The data of Dinter and

Pechmann (1999a,b) indicate a higher slip rate of 1 mm/yr. The applicant stated that this increase will have little effect on the PSHA because the East Great Salt Lake and the Oquirrh faults are located too far from the site to generate significant ground motion. The applicant concluded that compared to all seismic sources, the East Great Salt Lake fault contributes only a small fraction to the total hazard, including an assumption of a 1 mm/yr slip rate.

The staff agrees the applicant's analyses are acceptable. The contribution of the East Great Salt Lake fault to the PFS seismic hazard is not significant, including the possible connection with the Oquirrh fault.

Co-Seismic Rupture of Stansbury and East Faults

The staff reviewed information and analyses provided in the SAR (Appendix 2G) regarding possible co-seismic rupture of the Stansbury and East faults or East/West fault and the potential impact of co-seismic rupture on ground motion hazard at the proposed PFS Facility. The staff agrees that co-seismic rupture of the East/West faults with the Stansbury fault is not supported by historic earthquakes, nor is it supported by recent geomorphic or geologic observations. Consequently, co-seismic rupture of these faults during the license period are unlikely. Thus, co-seismic rupture scenario would likely be given a very low weight in fault tree analysis and its contribution to the total hazard would be negligible.

The applicant estimated the potential effect of co-seismic rupture of the Stansbury and East faults on ground motion hazard at the proposed Facility based on scaling factors similar to those proposed for co-seismic rupture at Yucca Mountain, Nevada [developed by the expert elicitation for the Yucca Mountain PSHA (Civilian Radioactive Waste Management System Management and Operating Contractor, 1998)]. In its assessment, the applicant stated that because both Yucca Mountain and the proposed Facility are within the same tectonic setting (extension in the basin and range), the effects of coseismic rupturing on the characteristics of ground motion attenuation is similar. The staff agreed and found using Yucca Mountain scaling factors for the Facility to be acceptable. This finding, however, is specific to the proposed Facility because it is based on specific site conditions and regulatory requirements for the proposed Facility. It is not necessarily applicable to evaluations of co-seismic rupture at other spent nuclear fuel-related facilities.

The effects of simultaneous multiple-fault ruptures on ground motions at Yucca Mountain were estimated as an increase in the median ground motion and an increase in the standard error (Civilian Radioactive Waste Management System, Management and Operating Contractor, 1998). The increase in the median ground motion is expressed as a multiple of the median. The increase in the standard error is expressed as either a multiple of the standard error or as an additional error incorporated using the square root of the sum of the squares. These scaling and additional factors for peak ground acceleration obtained by seven ground motion teams are summarized in tabular format in Appendix 2G. From this table, PFS computed the geometric means of the scale factors from all seven ground motion teams (Civilian Radioactive Waste Management System Management and Operating Contractor, 1998) for both the median

ground motion and standard error and used these mean factors to estimate changes in the contributions of maximum magnitude earthquakes on Stansbury and East faults to the total hazard at the proposed PFS Facility. The calculations show that, without co-seismic rupture, a M 6.5 earthquake on East fault and a M 7.0 earthquake on Stansbury fault (the maximum expected magnitudes on these faults, respectively, Geomatrix Consultants, Inc., 1999a) have probabilities of approximately 0.35 and 0.32, respectively, of producing a peak ground acceleration in excess of 0.53g. The 0.53g is the 2000-year return period peak ground motion (Geometrix Consultants, Inc., 1999a). Considering that events of M 6.5 and larger on each fault have expected frequencies of occurrence of approximately 3×10^{-4} per year (Geomatrix Consultants, Inc., 1999a), these two earthquakes would contribute $0.35 \times (3 \times 10^{-4}) + 0.32 \times (3 \times 10^{-4}) = 2.0 \times 10^{-4}$ events per year to the annual frequency of exceeding 0.53 g. With co-seismic rupture of the East and Stansbury faults (i.e., assuming instead that the maximum earthquakes on the two faults occur as a single M 7.05 co-seismic rupture, M 7.05 was obtained using the combined moment for a M 6.5 and a M 7.0 earthquake), scaling the median ground motion level and the standard error produced by this earthquake by the mean factors results in a probability of approximately 0.62 of exceeding a peak ground acceleration of 0.53 g. Considering the frequency of the combined event remains to be 3×10^{-4}, the event would contribute $0.62 \times (3 \times 10^{-4}) = 1.8 \times 10^{-4}$ event per year to the annual frequency of exceeding 0.53g. This contribution does not exceed the contribution by two independent earthquakes.

The staff concludes that a co-seismic rupture for the Stansbury and the East faults is unlikely and will not impact the existing PSHA results. Therefore, a design earthquake analyses based on the 2000-year return period ground motion is acceptable.

2.1.6.3 Surface Faulting

Geomatrix Consultants, Inc. (1999a) documented several small faults in and around the site. These faults are all considered secondary faults related to deformation of the hanging wall above the larger East and West faults. These faults are too small to be independent seismic sources but large enough to be considered in the fault displacement analysis.

Similar to the seismic hazard evaluation, Geomatrix Consultants, Inc. (1999a) developed a probabilistic fault displacement hazard. The fault displacement hazard analysis was built on two methodologies developed for the Yucca Mountain PSHA (Civilian Radioactive Waste Management System Management and Operating Contractor, 1998). These methodologies, termed the earthquake approach and displacement approach, use Basin and Range empirical relationships with site-specific data to generate fault displacement hazard curves similar to seismic hazard curves.

Probabilistic fault displacement hazard results were calculated for three potential secondary faults that are under or near the site. These faults—informally named the C, D, and F faults—were identified from detailed seismic reflection profiles and confirmed by boreholes. The seismic profiles document offset of the unconformity between Promontory soil, deposited between 130–28 Ka, and Bonneville lacustrian deposits, deposited between 28–12 Ka. Vertical

separation across the largest strands of the F fault (F-1 and F-4) is approximately 5 feet in the last 60 Ka and 2 feet in the last 20 Ka. A critical observation is that these faults show evidence of repeated fault slip. This is important because it suggests that future faulting events will likely occur along these same faults and not on new faults under the site. In addition, these observations of repeated slip events allowed Geomatrix Consultants, Inc. (1999a) to constrain the average displacement per event for each fault.

Faulting recurrence rates and displacement per event were quantified based on vertical separation of the Quaternary marker horizons. The results show that based on the 95^{th}-percentile curve, significant displacements, above 0.04 inch, are expected to occur only with an annual frequency of less than 3×10^{-4}, or once in 3,333.3 years. Significant displacements of 4 inches or more are expected to occur only with an annual frequency of less than 2×10^{-4}, or once in 5,000 years. For a 2,000-year return period (annual frequency of 5×10^{-4}), displacements due to faulting are smaller than 0.04 inch, which is less than the settlement allowance for concrete foundations.

Geomatrix Consultants, Inc. (1999a) also considered other possible distributed faulting between the mapped faults. These displacements were small. For example Geomatrix Consultants, Inc. (1999a) measured only 2 inches of cumulative displacement across 88 m of exposure in Trench T-2, with a fracture spacing between 3 and 5 feet. This suggests vertical displacement of less than 1 m accumulated across the entire width of the proposed site (approximately 5,000 feet) during the last several million years.

Based on its revisions to the soil velocity models presented in the SAR, the applicant examined whether the new shear wave velocity data or re-interpreted velocity profiles would alter existing conclusions regarding the shallow seismic surveys (Bay Geophysical Associates, Inc., 1999). The shallow seismic surveys were used in part by Geomatrix Consultants, Inc.(1999a) in its probabilistic fault displacement hazard assessment. The applicant stated (Parkyn 2001) that Bay Geophysical Associates, Inc. reviewed the Northland Geophysical, L.L.C. (2001) report and found the shear wave velocities were consistent with those used in the shallow seismic surveys. The average shear wave velocities from the borehole geophysical measurements (Northland Geophysical, L.L.C.,2001) are compatible with the values used by Bay Geophysical Associates, Inc. (1999) in processing the shear wave seismic data.

The staff reviewed the discussion and analysis and found the displacement approach is representative of site conditions, and that these results are acceptable for use in assessing the faulting hazard at the proposed site. The staff found the applicant's faulting hazard results conservative and representative of the best estimates. Using a 2,000-year return period to calculate fault displacement is appropriate and consistent with the return period for estimating seismic ground motion hazard and for seismic design. The investigations and materials presented by the applicant provide reasonable assurance that the displacements due to faulting are smaller than 0.04 inch for a 2,000-year return period (annual frequency of 5×10^{-4}), which is less than the settlement allowance for concrete foundations. Therefore, the facility is not required to be designed for a potential surface faulting hazard.

In sum, the staff reviewed the applicant’s discussion on surface faulting and found it acceptable because:

- Surface geological structures at the proposed site were adequately described such that the safety of the site can be assessed and the design basis for surface faulting developed.

- Potential surface faulting that directly affects site conditions and the likely environmental impacts of activities at the site were sufficiently investigated and assessed.

- Surface faulting near or at the site will be too small to affect site safety. Therefore, no specific designs or mitigation actions with respect to surface faulting are required.

- Surface faulting will not directly influence potential consequences of a release of radioactive material during the operational lifetime of the Facility.

- No specific design is necessary for structures, systems, and components to withstand the effects of surface faulting.

This information is also acceptable for use in other sections of the SAR to develop the design bases of the Facility, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR 72.90(b-d), 72.92(a-c), 72.98(b), 72.98(c)(3), and 72.122(b) with respect to this issue.

2.1.6.4 Stability of Subsurface Materials

The staff has reviewed information presented in Section 2.6.4, Stability of Subsurface Materials, of the SAR, which refers to the following sections of the SAR for details: 2.6.1.5, Facility Plot Plan and Geologic Investigations; 2.6.1.6, Relationship of Major Foundations to Subsurface Materials; 2.6.1.7, Excavations and Backfill; 2.6.1.11, Static and Dynamic Soil and Rock Properties at the Site; 2.6.1.12, Stability of Foundations for Structures and Embankments; and 2.6.2.1, Engineering Properties of Materials for Seismic Wave Propagation and Soil-Structure Interaction Analyses (Private Fuel Storage Limited Liability Company, 2001). The staff also reviewed information presented in Appendix 2A, Geotechnical Data Report, of the SAR (Private Fuel Storage Limited Liability Company, 2001) and other data and analyses provided by the applicant (Stone and Webster Engineering Corporation, 1998, 2001a,b,c,d; ConeTec, Inc., 1999).

Geotechnical Site Characterization

Geotechnical characterization of the site was performed through a combination of field and laboratory testing. The site investigation included 32 borings for sampling and standard penetration testing (20 in the pad emplacement area, 10 in the canister transfer building area, and 2 along the access road). The boring locations are described in Figures 2.6-2 and 2.6-18 of the SAR. Also, 39 cone penetrometer tests (CPTs) and 16 dilatometer tests were performed at locations described in Figures 2.6-18 and 2.6-19 of the SAR, Revision 13. The CPTs gave continuous profiles of tip resistance and sleeve friction, which were interpreted to obtain profiles of relative soil strength and compressibility (ConeTec, Inc., 1999). Sixteen of the CPTs included down-hole compressional and shear wave velocity measurements. The borings were used mainly for conducting standard penetration tests (SPTs). In addition, several split-spoon samples were obtained along with the SPT. The split-spoon samples were used for laboratory index testing, such as Atterberg limits and percentage of fine fraction. Undisturbed (Shelby-tube) samples were also obtained and used for laboratory triaxial, direct shear, and odometer testing to obtain strength and compressibility data. Laboratory specimens and the test results are listed in Tables 2-6 of Stone and Webster Engineering Corporation (2000a). Sixteen test pits were excavated in the proposed pad emplacement area in January, 2001 for sampling and *in-situ* examination of the near-surface soil layers (Parkyn, 2001).

The water-table depth was estimated to be approximately 125 feet below the ground surface (i.e., at about elevation 4,350 feet above mean sea level), based on data from an observation well. A depth to groundwater of about this value is also implied by P-wave velocities from a seismic refraction survey that change from about 2,780 ft/sec to about 5,525 ft/sec at a depth of 90–131 feet.

Soil classification was performed using information from three sources: (i) visual field classification of drill cuttings and split-spoon samples following ASTM D2488–93 (American Society for Testing and Materials, 1999), (ii) Atterberg limits and percentage of fine fraction from laboratory testing of split-spoon samples, and (iii) interpretation of CPT logs. Based on information from these sources, the subsurface materials at the site were classified by the applicant as consisting of a relatively compressible top layer (layer 1) that is approximately 25–30 feet thick. Layer 1 is underlain by much denser and stiffer material (layer 2) classified as dense sand and silt. The strength and stiffness of layer-2 soil, interpreted from SPT values that exceed 100, indicate that the soil is not a likely source of instability for the proposed structures. Therefore, geotechnical site investigation was focused on determining the engineering characteristics of layer-1 soil, a mixture of clayey silt, silt, and sandy silt with occasional silty clay and silty sand. A detailed description of layer-1 soil is provided through 17 cross sections in the SAR Figures 2.6-5 (Sheets 1–14) and 2.6-21 through 2.6-23. Fourteen of the cross sections were developed along lines that cross the proposed storage-pad area and consist of six east-west lines, six north-south lines, and two diagonal lines (Figure 2.6-19 of the SAR). The other three cross sections were developed along east-west lines that cross the proposed Canister Transfer Building area (Figure 2.6-18 of the SAR). Based on these cross sections, layer-1 soil was subdivided into four sublayers, (in top-down order): layer 1A, classified as eolian silt, is

typically about 3–5 feet thick; layer 1B, a silty clay/clayey silt mixture that varies in thickness from about 5 to 10 feet; layer 1C, a mixture of clayey silt, silt, and sandy silt, with thickness of about 7.5–12 feet; and layer 1D, a silty clay/clayey silt mixture with maximum thickness of about 5 feet. Information provided in the SAR indicates that the eolian silt (layer 1A soil) will be excavated, mixed with sufficient Portland cement and water, and re-compacted to form a soil-cement subgrade in the proposed pad-emplacement and canister transfer building areas.

Profiles of cone tip resistance from the CPT [Figure 2.6-5 (Sheet 1–14) of SAR; ConeTec, Inc., 1999, Appendix A] indicate that the strength of the silty clay/clayey silt layers (layers 1B and 1D) is smaller than the strength of layer 1C (clayey silt, silt, and sandy silt). The value of tip resistance in layers 1B and 1D is typically about 0.5 and 0.75, respectively, of layer 1C tip resistance. Information from the CPT tip resistance profiles, which indicate the variation of relative strength with depth, was combined with laboratory compression test results from layer-1B specimens to obtain values of undrained shear strength for layer 1B, layer 1C, and layer 1D soils.

Soil compressibility was determined using a combination of laboratory compressibility data for layer 1B soils and CPT data. Cone tip resistance profiles (from the CPT) show the relative compressibility of the soil layers, with layer 1B being the most compressible and layer 1C the least. This variation of relative compressibility indicates that values of settlement calculated using the compressibility data for layer 1B soil represent the upper bound for the entire soil profile. Settlement of the entire soil profile can also be calculated directly from the cone tip resistance values using an empirical approach developed by Schmertmann (1970, 1978). The approach is described in detail in Lunne et al. (1997).

The potential for significant additional settlement owing to collapsible soils was explored by the applicant. The occurrence of collapsible soils at the site is suggested by the high values of void ratio reported for several specimens in the SAR. Collapsible soils may undergo a relatively large decrease in volume when wetted or subjected to dynamic loading. Therefore, the occurrence of significant quantities of such soils under the foundation of a structure requires analysis on the potential for relatively high settlements if the foundation soil is wetted or subjected to dynamic loading. The following information presented in Section 2.6.1.11.4 of the SAR, demonstrates that the risk of significant additional settlement owing to soil collapse is negligible. First, results of laboratory testing on five specimens with high-void ratio (1.95–2.51) indicate the additional vertical strain that resulted from inundating the specimens with water is only about 0.001 (i.e., an additional settlement of about 0.12 inch for a 10-foot thick soil layer). Second, the top 5–7 feet soil layer at the pad emplacement area will be replaced with a low-permeability soil/cement mixture. Furthermore, the ground surface in the pad area will be graded to promote run-off toward the north. This arrangement is expected to make water influx into the pad foundation soil unlikely. Also, the pad emplacement area is at an elevation of at least 4 feet above the probable maximum flood level. Third, there is no known record of excess settlement resulting from collapsible soils occurring in the Skull Valley area. The only known occurrence of collapsible soil in Utah is in Cedar City, which is far from the site. Any occurrence of excess settlement in the Skull Valley area would likely have been mentioned in the County Soil Report (a USDA

unpublished report), which deals with the suitability of the various soil types for septic-systems construction.

The staff reviewed the geotechnical site characterization information provided in the SAR and concluded that:

- The depth and thicknesses of soil layers and the water-table depth at the site are described in sufficient detail to support engineering analyses of the proposed structures.

- The index properties and strength and compressibility of the soil layers were determined using an appropriate combination of field and laboratory testing. The information presented is sufficient to support appropriate engineering analyses of the proposed structures.

- The potential for instability resulting from possible occurrence of collapsible soils at the proposed site was investigated in sufficient detail. Results of the investigation indicate that the potential for such instability is negligible.

The staff concludes that the geotechnical site characterization information presented in the SAR is adequate for use in other sections of the SAR to develop the design bases for the Facility and perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c, d) and 72.122(b).

Stability of Cask-Storage-Pad Foundation

The cask storage pads (each 30 ft wide, 67 ft long, and 3 ft thick) will be laid out in two clusters: a north cluster separated from a south cluster by a 90-foot wide space (Private Fuel Storage Limited Liability Company, 2001, Figure 1.2-1). Each cluster consists of 25 north-south columns of storage pads. Each pad column is separated from the adjacent column by a 35-foot wide space (in the east-west direction) and consists of ten pads arranged end-to-end in the north-south direction with a 5-foot separation between the adjacent pads. Each pad cluster, therefore, consists of 250 pads, giving a total of 500 pads in the two clusters. An east-west vertical section through a typical storage pad (Private Fuel Storage Limited Liability Company, 2001, Figure 4.2-7) indicates that the pad would be embedded in soil cement of a maximum thickness of 4 feet and 4 inches. The soil cement consists of two layers: an upper layer with the minimum unconfined compressive strength of 250 psi underlain by a lower layer with the maximum thickness of 2 ft, the minimum unconfined compressive strength of 40 psi, and the maximum elastic modulus of 75,000 psi. The base of the upper soil-cement layer is flush with the base of the pad. The soil cement is overlain by an 8-inch thick layer of compacted aggregate, the top surface of which is flush with the top surface of the pad.

Each storage pad will be loaded to a static bearing pressure of 1.87 ksf, considering the dead load plus long-term live load for a 30 feet × 67 feet × 3 feet concrete pad loaded with eight

casks. The 35-foot width-wise (east-west) separation between pad columns is considered large enough that the zones of influence of the static foundation loading from adjacent pad columns can be assumed to be independent to a depth of 30 ft below the base of the pads. Potential dynamic loading of the pads was characterized by vertical and horizontal ground accelerations of 0.711g and 0.695g, respectively, calculated based on consideration of a 2,000-year return-period earthquake. The stability of the pads was evaluated with respect to the potential for bearing-capacity failure or excessive settlement under static loading, and the potential for base sliding or bearing-capacity failure under dynamic loading. These aspects of the stability evaluation are reviewed in the following sections.

Stability Against Bearing-Capacity Failure Under Static Loading

Stability of the storage pads under static loading was determined through the allowable bearing pressure calculated using a factor of safety of 3.0. This is a standard procedure for the design of shallow foundations (e.g., Terzaghi et al., 1996). Two calculations of the allowable bearing pressure under static loading were provided (Stone and Webster Engineering Corporation, 2001b): one based on undrained analysis, using an undrained shear strength (c_u) of 2.2 ksf; and another based on drained analysis using a friction angle of 30° (with zero cohesion). The c_u value of 2.2 ksf was obtained from compression tests on specimens of layer 1B soil (silty clay/clayey silt), which, as described earlier, is the weakest soil layer, with a CPT tip resistance typically about 0.5 times the tip resistance of layer 1C soil. The friction angle of 30° is a lower bound estimate from the CPT data. Values of friction angle from the CPT data are generally greater than 35°. Therefore, either of these strength-parameter values (i.e., c_u value of 2.2 ksf, or friction angle of 30° with zero cohesion) is accepted as representing the average strength of layer 1 soil for the purpose of determining the allowable bearing pressure for the specified dimensions and embedment depth of the cask storage pad. The allowable bearing pressure was determined to be 4.36 ksf based on the undrained analysis, or 9.73 ksf based on the drained analysis (Table 2.6-6 of the SAR). Both values of allowable bearing pressure exceed the actual bearing pressure of 1.87 ksf, based on consideration of the foundation dead load plus long-term live load.

The staff reviewed the applicant's evaluation of the stability of the cask storage pads with respect to the potential for bearing-capacity failure under static loading. The evaluation was performed using appropriate techniques, foundation loading, and material properties; and an acceptable safety factor was demonstrated. Independent calculations were performed by the staff using a procedure suggested by Meyerhof (1956, 1965) to determine the SPT values (N) or CPT tip resistance values (Q_t) that are required to satisfy a safety factor of 3.0 against bearing failure under the cask-pad bearing pressure of 1.94 ksf, which bounds the bearing pressure of 1.87 ksf. The calculations gave the required values as N = 0.9 and Q_t = 7.05 ksf, which are much smaller than the measured N and Q_t values [Appendix 2A of the SAR, Revision 13, and Appendix A of ConeTec, Inc. (1999)]. Therefore, the staff concludes that the proposed cask-pad design is acceptable considering the potential for bearing-capacity failure under static loading, and the information provided in the SAR regarding the static bearing capacity of the storage

pads is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c, d) and 72.122(b).

Stability Against Excessive Settlement Under Static and Dynamic Loading

The settlement of the cask storage pad under the bearing pressure of 1.94 ksf is given in the SAR as 3.3 inches, which is considered an upper bound estimate, having been calculated using laboratory compressibility data for layer 1B soil and a bearing pressure larger than the static foundation bearing pressure of 1.87 ksf. The estimated settlement of 3.3 inches can be accepted as the upper bound considering the Q_t profiles for the site (discussed under *Geotechnical Site Characterization*), which indicate that layer 1B is the most compressible soil layer. An alternative estimate of the storage-pad settlement made by PFS using the Q_t data and a procedure developed by Schmertmann (1970, 1978) gave values of settlement smaller than 1.0 in. for the storage pads. Based on these calculations, the storage pads would be expected to undergo post construction settlement of not more than about 3 inches. The storage pads will be constructed such that their top surface is flush with the top surface of the compacted-aggregates layer.

The applicant estimated the potential settlement owing to dynamic compaction of the subsurface materials using the empirical procedure of Tokimatsu and Seed (1987) for the evaluation of settlements in sand from earthquake shaking (Stone and Webster Engineering Corporation, 2001d). The primary inputs to the analysis are the SPT blow count (corrected for the effects of overburden pressure), the earthquake magnitude, and the soil thickness that may undergo vibratory compaction. The applicant estimated a dynamic settlement of 0.15 in. using a corrected blow count of 28, earthquake magnitude of 7, and soil thickness of 15 ft. The values for blow count and soil thickness may be difficult to justify, but an examination of the calculation by the staff indicates that the settlement would increase to about 1.2 in. if the corrected blow count was decreased to 10 and the soil thickness increased to the maximum of about 30 ft for layer 1 soil.

The applicant indicated that changes caused by potential settlement of the pad would be corrected by scraping the aggregates from between the pads to maintain the top surface of the aggregates at the same elevation as the top surface of the pads (SAR p. 2.6-51; Enclosure 2 of Parkyn, 2001).

The staff reviewed the applicant's evaluation of the stability of the cask storage pads with respect to the potential for excessive settlement under static and dynamic loadings. The evaluation was performed using appropriate techniques, foundation loading, and material properties. The staff considers the analysis of the stability of the cask storage pads acceptable. In addition, the applicant committed to perform maintenance repair of the pad-emplacement area as necessary to correct any changes caused by settlement of the pad, such as by scraping aggregates from between the pads to maintain the top surface of the aggregate layer at the same elevation as the top surface of the pads. The staff concludes that the information provided in the SAR regarding potential settlement of the storage pads is adequate for use in other

sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c, d) and 72.122(b).

Stability Against Sliding Under Dynamic Loading

As shown in Figure 4.2-7 of the SAR, each storage pad would be surrounded by soil cement that consists of two layers as follows: an upper layer with the minimum unconfined compressive strength of 250 psi, and a lower layer having the minimum unconfined compressive strength of 40 psi, and the maximum elastic modulus of 75000 psi. The base of the upper soil-cement layer is flush with the base of the pad. The applicant provided sliding stability analyses that rely on the shear strength of the natural soil underlying the lower layer of soil cement to resist sliding of the pads (Stone and Webster Engineering Corporation, 2001b, p. 18–35). For these analyses, the applicant assumed that a sufficient bond would develop at the interfaces between the upper and lower soil cement layers, the concrete pad and lower soil cement layer, and the lower soil cement layer and the underlying natural soil. Therefore, the applicant concluded that failure of the natural soil would be more likely than failure of any of the interfaces. The applicant has also committed to perform laboratory tests during the design of the soil cement to demonstrate that the required shear strengths can be achieved at the various interfaces, and to perform field tests during construction to demonstrate that the required shear strengths at these interfaces have been achieved (PFS - SAR, section 2.6.1.12.1).

The applicant also presented analyses to assess stability against sliding on two deep-seated failure surfaces: one at the base of the soil-cement subgrade, and another located within layer 1C soil (SAR, pp. 2.6-63 through 2.6-72). Such analyses require an examination of several potential failure surfaces and an assessment of stability using the conditions on the most critical failure surface. This approach was not followed in the analyses presented by the applicant for stability against sliding on deep-seated surfaces. The NRC staff concluded that an explicit analysis of deep-seated sliding is not necessary for the proposed facility because of the following reasons: (i) subsurface investigations conducted at the site do not indicate the occurrence of any deep-seated and relatively weak soil layer in which sliding may be localized; (ii) the assessment of stability against bearing-capacity failure (evaluated next) is based on the bearing capacity theory (e.g., Terzaghi et al., 1996, pp. 258–261), which considers sliding on a series of failure surfaces that may develop in a thick soil deposit of uniform shear strength; and (iii) the subsurface conditions at the site, (i.e., shear strength increasing with depth) satisfy the assumptions used to develop the bearing capacity theory.

The applicant also provided a set of analyses that rely on the frictional resistance of the interfaces and the passive resistance of the natural soil at the north or south boundaries of the soil-cement layers to resist sliding of the pads (Stone and Webster Engineering Corporation, 2001b, p. 36–42). Each of the interfaces was assigned a friction coefficient of 0.306 (friction angle of 17°) in the analyses. This value of friction coefficient is consistent with the values recommended in the literature for interfaces between concrete and fine-grained soils. For example, Terzaghi et al. (1996, p. 328) suggest a maximum value of about 0.364 (friction angle of 20°) for such interfaces. The values of safety factor obtained from the analyses indicate that

ground motion from the design-basis earthquake could cause sliding of the pads (or pad-foundation system). The applicant determined that the magnitude of sliding displacement would not exceed about 6 inches (Stone and Webster Engineering Corporation, 2001b, p. 43–45) and stated that such sliding displacement would not constitute a safety hazard because there are no external safety-related connections to either the pads or the casks. This statement was supported by additional analyses provided by the applicant (Holtec International, 2001, Attachment 1), which also indicate that sliding of the pads would reduce the tendency for sliding or tipping over of the casks.

The staff agrees with the applicant's conclusion that sliding of the pads would not constitute a safety hazard because pad sliding tends to increase the stability of the casks (against sliding or tip over) and there are no safety-related external connections to the pads or casks that may rupture or be misaligned as a result of pad sliding. Therefore, the staff concludes that the proposed cask-pad design is acceptable considering the potential for instability resulting from sliding of the pads under dynamic loading, and the information provided in the SAR regarding potential sliding of the pads is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c, d) and 72.122(b).

Stability Against Bearing Capacity Failure Under Dynamic Loading

The assessment of stability against bearing capacity failure of the storage pads under dynamic loading was based on bearing-capacity analyses for the load cases shown in Table 2-4. In each load case, the static load (dead load plus long-term live load for a 30 feet × 67 feet × 3 feet concrete pad loaded with eight casks) was combined with dynamic-load components determined using the load factors shown in the table. The dynamic load applied in a given direction is equal to the product of the load factor and the design basis earthquake load for that direction. A negative load factor for vertical force indicates that the vertical force is applied upward. The combinations of dynamic-load factors shown in the table satisfy NRC requirements given in Newmark and Hall (1978). The table shows values of the calculated and allowable bearing pressures for each load case. The allowable bearing pressure was determined using a factor of safety of 1.1 and a value of undrained shear strength (c_u) of 2.2 ksf. This value of c_u is the minimum for layer-1 soil and, consequently, is accepted as an average value along potential failure surfaces that may develop in this soil layer. Values of the calculated and allowable bearing pressures vary because of changes in the effective bearing area of the pads caused by the eccentricity of the resultant applied loading for each load case. The magnitude of dynamic horizontal force transmitted from the casks to the pad was calculated using a value of 0.8 for the cask-on-pad friction coefficient. As Table 2-4 shows, the calculated bearing pressure for each load case is smaller than the allowable bearing pressure.

PFS also presented stability analyses for partially loaded pads under dynamic loading. Analyses were presented for pads loaded with two or four casks (instead of the full load of eight casks) and subjected to 100 percent dynamic loading (load factor of 1.0) in every direction. The dynamic loadings were obtained from finite element analyses of a pad loaded with two or four

casks and subjected to vertical and horizontal acceleration time histories representative of the design earthquake. Results of the bearing capacity analysis (Table 2.6-8 of the SAR) indicate adequate safety factors against bearing capacity failure under dynamic loading for pads loaded with two or four casks.

The staff reviewed the applicant's evaluation of the stability of the cask storage pads with respect to the potential for bearing-capacity failure under dynamic loading. The evaluation was performed using appropriate techniques, foundation loading, and material properties, and an acceptable safety factor was demonstrated. Based on the results of the analyses, the staff concludes that the proposed cask-pad design is acceptable considering the potential for bearing-capacity failure under dynamic loading, and the information provided in the SAR regarding the dynamic bearing capacity of the storage pads is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c) and 72.102(d).

Table 2-4. Results of bearing capacity analysis of storage pads under dynamic loading (from Private Fuel Storage Limited Liability Company, 2000)

Load Case	Dynamic Load Factors			Bearing Pressure (ksf)	
	North-South (Pad Long Dimension)	East-West (Pad Short Dimension)	Vertical	Allowable	Calculated
II	1.0	1.0	0.0	4.85	4.56
IIIA	0.4	0.4	−1.0	8.21	2.13
IIIB	0.4	1.0	−0.4	4.78	4.55
IIIC	1.0	0.4	−0.4	8.59	2.61
IVA	0.4	0.4	1.0	10.51	3.76
IVB	0.4	1.0	0.4	7.73	4.09
IVC	1.0	0.4	0.4	9.45	3.83

Stability of the Canister Transfer Building Foundation

The proposed Canister Transfer Building will be founded on a rectangular reinforced concrete mat 240-ft wide (east-west direction), 279.5-ft long (north-south direction), and 5-ft thick (Figure 4.7-1 of SAR). The perimeter of the foundation mat to a distance of 6.5 ft from the edge will be extended to a depth of 1.5 feet below the base of the mat to form a shear key into the underlying

soil. The natural soil around the foundation will be replaced by soil cement to a depth of 5 ft below the top of the foundation mat and laterally to a distance of one mat dimension from the edge of the mat in every direction {i.e., 240 ft out from the mat in the east and west directions and 279.5 ft out from the mat in the north and south direction (Stone and Webster Engineering Corporation, 2001c; Enclosure 2 of Parkyn, 2001)}. The soil cement will have a minimum unconfined compressive strength of 250 psi.

The foundation loading was determined through a lumped-mass analysis of the Canister Transfer Building, which gave a vertical static load of 97,749 kips and dynamic load of 79,779 kips vertical, 111,108 kips north-south, and 99,997 kips east-west (Table 2.6-11 of the SAR). The dynamic loads were calculated using vertical and horizontal ground accelerations of 0.711 g and 0.695 g, respectively, which represent the 2,000-year return-period earthquake for the facility design. The lumped-mass analysis has been reviewed and accepted by the NRC staff (Chapter 5). The stability of the Canister Transfer Building foundation was evaluated with respect to the potential for bearing-capacity failure or excessive settlement under static loading, and the potential for base sliding or bearing-capacity failure under dynamic loading. These aspects of the stability evaluation are reviewed in the following sections.

Stability Against Bearing-Capacity Failure Under Static Loading

The Canister Transfer Building foundation will be loaded to a bearing pressure of 1.46 ksf, considering the vertical static load of 97,749 kips supported by a total bearing area of 240 x 279.5 ft^2 (Stone and Webster Engineering Corporation,2001c). The stability of the Canister Transfer Building foundation under static loading was determined through the allowable bearing pressure using a factor of safety of 3.0. This is a standard procedure for the design of shallow foundations (e.g., Terzaghi et al., 1996). Two calculations of the allowable bearing pressure under static loading were provided: one based on undrained analysis using an undrained shear strength (c_u) of 3.18 ksf; and another based on drained analysis using a friction angle of 30° (with zero cohesion). The c_u value of 3.18 ksf is a depth-weighted average for layer 1 soil from the base of the Canister Transfer Building foundation to a depth of 20-25 ft below the foundation. The average was calculated using the c_u value of 2.2 ksf for layer 1B soil from laboratory compression test and the variation of relative strength with depth from CPT data. The relatively stiff layer 2 soil, which lies at a depth of 20–25 feet below the Canister Transfer Building foundation, was not included in the calculation of average strength. The friction angle of 30° is a lower bound estimate from the CPT data. Values of friction angle from the CPT data are generally greater than 35°. Therefore, either of these strength-parameter values (i.e., c_u value of 3.18 ksf, or friction angle of 30° with zero cohesion) is accepted as representing the average strength of layer 1 soil for the purpose of determining the allowable bearing pressure for the Canister Transfer Building foundation. The allowable bearing pressure was determined to be 6.54 ksf based on the undrained analysis, or 56.6 ksf based on the drained analysis (Table 2.6-9 of the SAR). Both values of allowable bearing pressure exceed the actual bearing pressure of 1.46 ksf under static loading. The staff reviewed the applicant's evaluation regarding the estimated allowable bearing pressure under static loading and found it acceptable.

The staff reviewed the applicant's evaluation of the stability of the Canister Transfer Building foundation with respect to the potential for bearing-capacity failure under static loading. The evaluation was performed using appropriate techniques, foundation loading, and material properties; and an acceptable safety factor was demonstrated. Therefore, the staff concludes that the proposed design of the Canister Transfer Building foundation is acceptable considering the potential for bearing-capacity failure under static loading, and the information provided in the SAR regarding the static bearing capacity of the Canister Transfer Building foundation is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c, d) and 72.122(b).

Stability Against Excessive Settlement Under Static and Dynamic Loading

The settlement of the Canister Transfer Building foundation under the bearing pressure of 1.67 ksf is given in the SAR as 3 inches, which is considered an upper bound estimate having been calculated using laboratory compressibility data for layer 1B soil and a bearing pressure larger than the estimated static bearing pressure of 1.46 ksf for the foundation. The estimated settlement of 3 inches can be accepted as the upper bound considering the Q_t profiles for the site (discussed under *Geotechnical Site Characterization*), which indicate that layer 1B is the most compressible soil layer. As discussed under "Stability of Cask-Storage-Pad Foundation" subsection titled "Stability Against Excessive Settlement Under Static and Dynamic Loadings," the applicant's calculation indicates that a maximum settlement of about 1.2 in. can be expected from soil compaction owing to the design-basis earthquake.

The staff reviewed the applicant's evaluation of the stability of the Canister Transfer Building foundation with respect to the potential for excessive settlement under static and dynamic loadings. The evaluation was performed using appropriate techniques, foundation loading, and material properties. The staff considers this stability analysis acceptable, and concludes that the information provided in the SAR regarding potential settlement of the Canister Transfer Building foundation is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c,d) and 72.122(b).

Stability Against Sliding Under Dynamic Loading

The proposed design of the Canister Transfer Building foundation relies on two features to resist foundation sliding. First, a 1.5-foot deep perimeter key at the base of the foundation would constrain potential sliding surfaces to pass through the underlying soil, such that the strength of the soil can be relied upon to resist sliding. Second, sliding of the foundation would be resisted by the compression strength of the surrounding soil cement, such that passive resistance of the soil cement can be relied upon to contribute to the overall sliding resistance. Two assessments of the sliding stability of the foundation are provided in the SAR and in the supporting calculation package (Stone and Webster Engineering Corporation, 2001c). One assessment is based on combining the residual strength of the natural soil with the full passive resistance of the soil cement, and the other on a combination of the peak strength of the natural soil with 50 percent of the passive resistance of the soil cement. The first assessment gave a minimum safety factor

of 1.26 for all the load cases examined whereas the second assessment gave a minimum safety factor of 1.15 for the same load cases. Six load cases (IIIA, IIIB, IIIC, IVA, IVB, and IVC in Table 2-4) were used in the assessment. The load case with a load factor of 1.0 for the north-south component and 0.4 for the other two components produced the minimum safety factor. The applicant varied the vertical load factors as well as the horizontal factors in its assessment of the sliding stability as shown in Table 2.6-13 of the SAR. Changing the vertical dynamic load should have no effect, however, because the potential sliding surface is horizontal and the sliding resistance used in the analyses is independent of vertical loading. Therefore, the factor of safety against sliding may vary with the horizontal load factors but not with the vertical. Information presented in the SAR indicates that the case with 100 percent north-south and 40 percent east-west dynamic loads is bounding. The factor of safety obtained for this load case is larger than the minimum acceptable value of 1.1. This assessment of the sliding stability assumes that the soil cement around the foundation has a minimum unconfined compressive strength of 250 psi in every direction and at every point within the soil cement layer.

The applicant also presented analyses to assess stability against sliding on a deep-seated failure surface localized within layer-1C soil (SAR, pp. 2.6-79 through 2.6-81). Such analyses generally require an examination of several potential failure surfaces and an assessment of stability using the conditions on the most critical failure surface. This approach was not followed in the analyses presented by the applicant for stability against sliding on a deep-seated surface. The NRC staff, concluded that an explicit analysis of deep-seated sliding is not necessary for the proposed facility because of the following reasons: (i) subsurface investigations conducted at the site do not indicate the occurrence of any deep-seated and relatively weak soil layer in which sliding may be localized; (ii) the assessment of stability against bearing-capacity failure (evaluated next) is based on the bearing capacity theory (e.g., Terzaghi et al., 1996, p. 258–261), which considers sliding on a series of failure surfaces that may develop in a thick soil deposit of uniform shear strength; and (iii) the subsurface conditions at the site, (i.e., shear strength increasing with depth) satisfy the assumptions used to develop the bearing-capacity theory.

The staff reviewed the applicant's evaluation of the stability of the Canister Transfer Building foundation with respect to the potential for sliding under dynamic loading. The evaluation was performed using appropriate techniques, foundation loading, and material properties; and an acceptable safety factor was demonstrated. Therefore, the staff concludes that the proposed design of the Canister Transfer Building foundation is acceptable considering the potential for sliding under dynamic loading, and the information provided in the SAR regarding the sliding stability of the Canister Transfer Building foundation is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c, d) and 72.122(b).

Stability Against Bearing Capacity Failure Under Dynamic Loading

The assessment of stability against bearing capacity failure of the Canister Transfer Building foundation under dynamic loading was based on bearing-capacity analyses for the load cases

shown in Table 2-7. In each load case, the vertical static load of 97,749 kips was combined with dynamic-load components using the load factors shown in the table. The dynamic force applied in a given direction is equal to the product of the load factor and the appropriate component of dynamic load (79,779kips vertical;111,108 kips north-south; and 99,997kips east-west). A negative load factor for vertical force indicates that the vertical force is applied upward. The combinations of dynamic-load factors shown in the table satisfy NRC requirements in Newmark and Hall (1978). The table shows values of the calculated and allowable bearing pressures for each load case. The allowable bearing pressure was determined using a factor of safety of 1.1 and a value of undrained shear strength (c_u) of 3.18 ksf. This value of c_u is a depth-weighted average for layer 1 soil from the base of the Canister Transfer Building foundation to a depth of 20-25 feet below the foundation. The average was calculated using the c_u value of 2.2 ksf for layer 1B soil from laboratory compression test and the variation of relative strength with depth from CPT data. The relatively stiff layer 2 soil, which lies at a depth of 20–25 ft below the Canister Transfer Building foundation, was not included in the calculation of average undrained strength. The average c_u would be larger if layer 2 soil was included in the calculation. Therefore, this value of c_u is accepted as an estimate of the average c_u value along a potential failure surface that may result from Canister Transfer Building foundation loading. Values of the calculated and allowable bearing pressures vary because of changes in the effective bearing area of the foundation caused by the eccentricity of the resultant applied loading for each load case. As Table 2-5 shows, the calculated bearing pressure for each load case is smaller than the allowable bearing pressure.

The staff reviewed the applicant's evaluation of the stability of the Canister Transfer Building foundation with respect to the potential for bearing-capacity failure under dynamic loading. The evaluation was performed using appropriate techniques, foundation loading, and material properties; and an acceptable safety factor was demonstrated. Therefore, the staff concludes that the proposed design of the Canister Transfer Building foundation is acceptable considering the potential for bearing-capacity failure under dynamic loading, and the information provided in the SAR regarding the dynamic bearing capacity of the Canister Transfer Building foundation is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c, d) and 72.122(b).

Table 2-5. Results of bearing capacity analysis of Canister Transfer Building foundation under dynamic loading (from Private Fuel Storage Limited Liability Company, 2001)

Load Case	Dynamic Load Factors			Bearing Pressure (ksf)	
	North-South	East-West	Vertical	Allowable	Calculated
II	1.0	1.0	0.0	11.97	2.39
IIIA	0.4	0.4	−1.0	12.54	0.99
IIIB	0.4	1.0	−0.4	12.82	1.70
IIIC	1.0	0.4	−0.4	13.67	1.65
IVA	0.4	0.4	1.0	16.26	2.92
IVB	0.4	1.0	0.4	14.19	2.50
IVC	1.0	0.4	0.4	14.53	2.47

Liquefaction Potential

The subsurface materials are not likely to undergo liquefaction. The relatively compressible soil layers within the top 25–30 feet depth would not undergo liquefaction because of the depth of the water table (125 feet below the ground surface). Also, the material below 25–30 feet consists of dense granular soil with high (> 50) *N* values. Such materials experience dilation when subjected to shear strain, decreasing the pore pressure (e.g., Lambe and Whitman, 1969, Figure 29.6 and Table 7.4). As a result, the materials within the saturated zone are not likely to undergo liquefaction.

Staff Evaluation

The staff has reviewed Section 2.6.4, Stability of Subsurface Materials, of the SAR and concludes that the information presented in this section is adequate for use in other sections of the SAR to develop the design bases of the Facility, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR 72.102(c) and 72.102(d).

2.1.6.5 Slope Stability

There are no natural slopes close enough to the proposed Facility that require stability evaluation. The foundation excavations would be backfilled to the current ground-surface elevation, so there will not be any excavated slopes at the site.

The site layout includes four embankments: the railroad embankment, the Facility berm, the access road embankment, and the road berm. However, these embankments have been classified as not important to safety in Section 2.5.4.4 of the SAR. Also, evaluations in Section 2.1.4.4 of this SER show that failure of the embankments would not affect any structures important to safety. Consequently, the geotechnical design of the embankments is not presented or evaluated.

The staff reviewed the applicant's discussion of slope stability and found it acceptable because:

- The slopes and slope materials of the site and vicinity have been adequately described such that safety of the site can be assessed and design bases for slope stability during external events can be developed.

- The slope stability that directly affects site conditions and the likely environmental impact of activities at the site have been sufficiently investigated and assessed.

- The severity of slope instability that may directly affect site safety has been sufficiently investigated and assessed.

- Slope stability is not a safety concern during natural or man-induced events. Therefore, no specific designs or mitigation actions with regard to slope stability are required.

- There is no known landslide area near the site that may affect site safety.

This information is acceptable for use in other sections of the SAR to develop the design bases of the Facility, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR 72.90(a-d), 72.92(a-c), and 72.122(b) with respect to this issue.

2.1.6.6 Volcanism

The staff has reviewed information presented in Section 2.6.1 and Appendix 2E of the SAR with regard to volcanism. Chemical analyses of ash layers exposed in trenches and boreholes at the Facility indicate they are chemically similar to the Walcot Tuff, which erupted approximately 6.4 Ma near Heise, Idaho (see Appendix 2E of the SAR). The closest Quaternary volcanic activity (which occurred between 950 and 880 Ka) is located more than 50 miles south of the Facility at Fumarole Butte. Therefore, volcanism is not deemed a credible event at the site.

The staff reviewed the discussion on volcanism and found it acceptable because the applicant demonstrated that volcanism is not a credible phenomenon at the Facility. This information is acceptable for use in other sections of the SAR to develop the design bases of the Facility, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR 72.92(a-c) and 72.122(b) with respect to this issue.

2.2 Evaluation Findings

The staff has reviewed the site characteristics presented in the SAR. The staff finds that the SAR provides an acceptable description and safety assessment of the site on which the PFS Facility is to be located, in accordance with 10 CFR 72.24(a). The staff also finds that the proposed site complies with the criteria of 10 CFR 72 Subpart E, as required by 10 CFR 72.40(a)(2).

2.3 References

Abrahamson, N.A., and W.L. Silva. 1997. Empirical response spectral attenuation relations for shallow crustal earthquakes. *Seismological Research Letters* 68(1): 94–127.

American Society for Testing and Materials. 1999. Standard practice for description and identification of soils (Visual-manual procedure). *Annual Book of ASTM Standards* D2488–93e1: West Conshohocken, PA: American Society for Testing Materials. 228–238.

Anderson, J.G., S.G. Wesnousky, and M.W. Stirling. 1996. Earthquake size as a function of fault slip rate. *Bulletin of the Seismological Society of America.* 86(3): 683–690.

Arabasz, W.J., J.C. Pechmann, and E.D. Brown. 1987. *Evaluation of Seismicity Relevant to the Proposed Siting of a Superconducting Supercollider (structures, systems, and component) in Tooele County, Utah*. Tooele County, UT: Dames and Moore.

Arabasz, W.J., J.C. Pechmann, and E.D. Brown. 1989. Evaluation of seismicity relevant to the proposed siting of a Superconducting Supercollider (structures, systems, and component) in Tooele County, Utah. *Utah Geological and Mineral Survey Miscellaneous Publications* 89(1): 107.

Ashcroft, G.L., D.T. Jensen, and J.L. Brown. 1992. *Utah Climate*. Logan, UT: Utah State University, Utah Climate Center.

Axen, G.J., W.J. Taylor, and J.M. Bartlet. 1993. Space-time patterns and tectonic controls of tertiary extension and magmatism in the Great Basin of the western United States. *Geological Society of America Bulletin* 105: 56–76.

Bay Geophysical Associates, Inc. 1999. *High-Resolution Seismic Shear-Wave Reflection Profiling for the Identification of Faults at the Private Fuel Storage Facility, Skull Valley, Utah—Final Report*. Traverse City, MI: Bay Geophysical Associates, Inc.

Bender, B. 1984. Seismic hazard estimation using a finite fault rupture model. *Bulletin of the Seismological Society of America* 74: 1,899–1,923.

Bishop, K.M. 1997. Miocene rock-avalanche deposits, Halloran/Sihrian Hills area, southeastern California. *Environmental and Engineering Geosciences* III: 501–512.

Bureau of Land Management. 1985. *Skull Valley Allotment Management Plan*. Salt Lake City, UT: U.S. Department of Interior, Salt Lake District, Bureau of Land Management.

Bureau of Land Management. 1986. *Skull Valley Allotment Management Plan*. Salt Lake City, UT: U.S. Department of Interior, Salt Lake District, Bureau of Land Management.

Bureau of Land Management. 1988. *Draft Pony Express Resource Management Plan and Environmental Impact Statement*. Salt Lake City, UT: U.S. Department of Interior, Salt Lake District, Bureau of Land Management.

Bureau of Land Management. 1992. *Horseshoe Springs Habitat Management Plan*. UT–020–WHA–T–7. Salt Lake City, UT: U.S. Department of Interior, Salt Lake District, Bureau of Land Management.

Campbell, K.W. 1997. Empirical near-source attenuation relationships for horizontal and vertical components of peak ground acceleration, peak ground velocity, and pseudo-absolute acceleration response spectra. *Seismological Research Letters* 68(1): 154–179.

Chen, R., and A.H. Chowdhury. 1998. *Seismic Ground Motion at Three Mile Island Unit 2 Independent Spent Fuel Storage Installation Site in Idaho National Engineering and Environmental Laboratory—Final Report.* CNWRA 98-007. San Antonio, TX: Center for Nuclear Waste Regulatory Analyses.

Civilian Radioactive Waste Management System Management and Operating Contractor. 1998. *Probabilistic Seismic Hazard Analyses for Fault Displacement and Vibratory Ground Motion at Yucca Mountain, Nevada*. DE–AC04–94AL85000. Las Vegas, NV: U.S. Department of Energy.

ConeTec, Inc. 1999. *Presentation of Cone Penetration Testing Results of Soils at the Private Fuel Storage Facility, Skull Valley, Utah*. 05996.02–G (P030) Revision 1. Salt Lake City, Utah. ConeTec, Inc.

Cornell, C.A. 1968. Engineering seismic risk analysis. *Bulletin of the Seismological Society of America* 58: 1,583–1,606.

Cornell, C.A. 1971. Probabilistic analysis of damage to structures under seismic loads. *Dynamic Waves in Civil Engineering*. D.A. Howells, I.P. Haigh, and C. Taylor eds. London: Wiley Interscience.

Cowan, D.S., and R.L. Bruhn. 1992. Late Jurassic to early Cretaceous geology of the U.S. Cordillera. The Cordilleran Orogen: Conterminous U.S. B.C. Burchfiel, P.W. Lipman, and M.L. Zoback, eds. Boulder, CO: *Geological Society of America. The Geology of North America* G-3: 407–479.

Currey, D.R., and C.G. Oviatt. 1985. Durations, average rates, and probable causes of Lake Bonneville expansions, stillstands, and contractions during the last deep-lake cycle, 32,000 to 10,000 years ago. *Problems of and Prospects for Predicting Great Salt Lake Levels*. P.A. Kay, P.A., and H.F. Diaz, eds. Salt Lake City, UT: University of Utah, Center for Public Affairs and Administration: 9–24.

Dames & Moore, Inc. 1996. *Final Report Seismic Hazard Analysis of the I-15 Corridor 10600 South to 500 North Salt Lake County, Utah*. UT-47026. Report submitted to Parsons Brinckerhoff. Salt Lake City, UT: Dames & Moore.

dePolo, C.M., D.G. Clark, D.B. Slemmons, and A.R. Ramelli. 1991. Historical surface faulting in the Basin and Range province, western North America: Implications for fault segmentation. *Journal of Structural Geology* 13: 123–136.

Dinter, D. A., and J. C. Pechmann. 1999a. Multiple Holocene earthquakes on the East Great Salt Lake fault, Utah: Evidence from high-resolution seismic reflection data: *EOS, Transactions of the American Geophysical Union* 80 (46) (Supplement): F934.

Dinter, D. A., and J. C. Pechmann. 1999b. Sublacustrine paleoseismology: Reflections of seismic evidence of recent earthquakes on the East Great Salt Lake Fault, Utah. *Association of Engineering Geologists, Program with Abstracts, 42nd Annual Meeting, Salt Lake City, UT, Sept. 26-29, 1999*: p.62-63.

Donnell, J.L. 1998. *Supplemental Response to RAIs*. Letter (June 15) to Director, Office of Nuclear Material Safety and Safeguards, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999a. *Commitment Resolution Letter No. 7*. Letter (June 30) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999b. *Commitment Resolution Letter No. 9*. Letter (July 14) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999c. *Commitment Resolution Letter No. 10*. Letter (July 22) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999d. *Submittal of Request for Additional Information Calculations/Reports*. Letter (February 11) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999e. *Submittal of Commitment Resolution Information*. Letter (March 25) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999f. *Submittal of Commitment Resolution Information.* Letter (May 18) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999g. *Submittal of Commitment Resolution Information* Letter (March 24) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999h. *Submittal of Commitment Resolution Information.* Letter (March 31) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999i. *Submittal of Commitment Resolution #4 Information.* Letter (April 22) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 1999j. *Submittal of Commitment Resolution #4 Information.* Letter (May 28) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Donnell, J.L. 2001. *Response to April 18, 2001 Meeting Issue regarding PFSF License Application Amendment #22.* Letter (May 1) to Nuclear Regulatory Commission. Englewood, CO: Private Fuel Storage Limited Liability Company.

England P., and J. Jackson. 1989. Active deformation of the continents. *Annual Reviews of Earth and Planetary Sciences* 17: 197,226.

Frankel, A., S. Harmsen, C. Mueller, T. Barnhard, E.V. Leyendecker, D. Perkins, S, Hanson, N. Dickman, and M. Hooper. 1997. USGS National Seismic Hazard Maps: uniform hazard spectra, de-aggregation, and uncertainty. *Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Facilities, NCEER Technical Report 97-0010*: 39–73.

Geomatrix Consultants, Inc. 1997. *Deterministic Earthquake Ground Motions Analysis, Private Fuel Storage Facility, Skull Valley, Utah.* GMX #3801.1. Revision 0. San Francisco, CA: Geomatrix Consultants, Inc.

Geomatrix Consultants, Inc. 1999a. *Fault Evaluation Study and Seismic Hazard Assessment Private Fuel Storage Facility, Skull Valley, Utah*. San Francisco, CA: Geomatrix Consultants, Inc.

Geomatrix Consultants, Inc. 1999b. *Update of Deterministic Ground Motion Assessments, Private Fuel Storage Facility, Skull Valley.* San Francisco, CA: Geomatrix Consultants, Inc.

Geomatrix Consultants, Inc. 1999c. *Development of Design Ground Motions for the Private Fuel Storage Facility*, *Skull Valley, Utah*. Project No. 4790. San Francisco, CA: Geomatrix Consultants, Inc.

Geomatrix Consulatnts Inc., 2001a. *Fault evaluation study and seismic hazard assessment study-final report*. Revision 1. Oakland, CA: Geomatrix Consultants, Inc.

Geomatrix Consultants, Inc. 2001b. *Development of Design Ground Motions for the Private Fuel storage Facility*, Revision 1. Oakland, CA: Geomatrix Consultants, Inc.

Geomatrix Consultants, Inc. 2001c, *Soil and foundation parameters for dynamic soil-structure interaction analysis*: Geomatrix Calculation 05996.02-G(PO18)-2, Rev 1. prepared for Stone and Webster Engineering Corporation. 1. Oakland, CA: Geomatrix Consultants, Inc.

Geomatrix Consultants, Inc. 2001d. *Update of Deterministic ground motion assessments,* Revision 1. Oakland, CA: Geomatrix Consultants, Inc.

Livermore Software Technology Corporation. 2000. LS-DYNA: A Program for Nonlinear Dynamic Analysis of Structures in Three Dimensions. Version 950e. August. Livermore, CA: Livermore Software Technology Corporation.

Grazulis, T.P. 1993. *Significant Tornadoes: 1680–1991.* St. Johnsbury, VT: The Tornado Project of Environmental Films.

Hintze, L.F. 1988. Geologic Map of Utah. Scale 1:500,000. Salt Lake City, UT: *Utah Geological and Mining Survey*.

Holtec International. 2001. *Multi Cask Response At PFS ISFSI From 2000-Yr Seismic Event (Rev. 2)*. Holtec Report No. HI-2012640. Marlton, NJ: Holtec International.

Hossain, Q.A., A.H. Chowdhury, M.P. Hardy, K.S. Mark, J.E. O'Rourke, W.J. Silva, J.C. Stepp, and F.H. Swan, III. 1997. *Seismic and Dynamic Analysis and Design Considerations for High-Level Nuclear Waste Repositories*. J.C. Stepp, ed. New York: American Society of Civil Engineers.

Idriss, I.M. 1991. *Selection of Earthquake Ground Motions at Rock Sites*. Davis, CA: National Institute of Standards and Technology, Department of Civil Engineering, University of California.

Jones, C.H., J.R. Unruh, and L.J. Sonder. 1996. The role of gravitational potential energy in active deformation in the southwestern United States. *Nature* 381: 37–41.

Kirpich, P.Z. 1964. Dimensionless constants for hydraulic elements of open-channel cross-sections. *Civil Engineering* 18(10): 47.

Lambe, T.W., and R.V. Whitman. 1969. *Soil Mechanics*. New York: John Wiley & Sons, Inc.

Lunne, T., P.K. Robertson, and J.J.M. Powell. 1997. *Cone Penetration Testing in Geotechnical Engineering, 1st Edition*. London, England: Blackie Academic & Professional.

Machette, M.N., and W.E. Scott. 1988. Field trip introduction—A brief overview of research on lake cycles and neotectonics of the eastern Basin and Range province. *Utah Geological and Mineral Survey Miscellaneous Publication* 88(1): 7–14.

Machette, M.N., S.F. Personius, A.R. Nelson, D.P. Schwartz, and W.R. Lund. 1991. The Wasatch fault zone, Utah-segmentation and history of Holocene earthquakes. *Journal of Structural Geology* 13: 137-149.

Martinez, L., C. M. Meertens, and R. B. Smith. 1998. Anomalous intraplate deformation of the Basin and Range-Rocky Mountain transition from initial GPS measurements, *Geophysical Research. Letters 24*: 2741-2744.

McGuire, R.K. 1976. *FORTRAN Computer Program for Seismic Risk Analysis*. OFR 76-67. Reston, VA: U.S. Geological Survey.

McGuire, R.K. 1978. *FRISK: Computer program for seismic risk analysis using faults as earthquake sources*. OFR 78-1007. Reston, VA: U.S. Geological Survey.

Meyerhof, G.G. 1956. Penetration tests and bearing capacity of cohesionless soils. *American Society of Civil Engineers Journal of the Soil Mechanics and Foundations Division* 82(SM1): 1–19.

Meyerhof, G.G. 1965. Shallow foundations. *ASCE Journal of Soil Mechanics and Foundations Division* 91(SM2): 21–31.

Morris, A., D.A. Ferrill, and D.B. Henderson. 1996. Slip-tendency analysis and fault reactivation. *Geology 24*, 275-278.

Murphy, A.J., N.C. Chokshi, R. McMullen, R. Kenneally, L.C. Shao, and R. Rothman. Revision of seismic and geologic siting criteria. *Transactions of the 14th International Conference on Structural Mechanics in Reactor Technology (SmiRT 14*), Lyon, France, 1997.

National Oceanic and Atmospheric Administration. 1960. *Climatography of the United States: No. 60, Climate of Utah.* Salt Lake City, UT: National Oceanic and Atmospheric Administration, National Climatic Data Center.

National Oceanic and Atmospheric Administration. 1992. *Local Climatologic Data, Annual Summary with Comparative Data for 1991.* Salt Lake City, UT: National Oceanic and Atmospheric Administration, National Environmental Satellite Data and Information Service, National Climatic Data Center.

National Oceanic and Atmospheric Administration. 1975–1995. *Storm Data and Unusual Weather Phenomena with Late Reports and Corrections.* Asheville, NC: National Oceanic and Atmospheric Administration, National Climatic Data Center.

National Weather Service. 1977. *Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages*. HMR 49. Silver Springs, MD: U.S. Department of Commerce, National Oceanic and Atmospheric Administration.

Newmark, N.M., and W.J. Hall. 1978. *Development of Criteria for Seismic Review of Selected Nuclear Power Plants*. NUREG/CR–0098. Washington, DC: Nuclear Regulatory Commission.

Northland Geophysical, L.L.C., 2001. *Report on Downhole Seismic Geophysical Testing, Report No. 05996.02-GG9Po37)-1*. Revision 1. Snohomish, WA: Northland Geophysical, L.L.C.

Nuclear Regulatory Commission. 1983. *Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants*. Regulatory Guide 1.145. Revision 1. Washington, DC: Nuclear Regulatory Commission, Office of Standards Development.

Nuclear Regulatory Commission. 1997a. *Standard Review Plan for Dry Cask Storage Systems*. NUREG–1536. Washington, DC: Nuclear Regulatory Commission.

Nuclear Regulatory Commission. 1997b. *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants*. NUREG–0800. Washington, DC: Nuclear Regulatory Commission.

Nuclear Regulatory Commission. 1997c. *Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion*. Regulatory Guide 1.165. Washington, DC: Nuclear Regulatory Commission.

Nuclear Regulatory Commission. 1998a. *Rulemaking Plan: Geological and Seismological Characteristics for Siting and Design of Dry Cask Independent Spent Fuel Storage Installation, 10 CFR Part 72.* SECY–98–126. Letter (June 4) from L.J. Callan (EDO) to the Commissioners. Washington, DC: U.S. Government Printing Office.

Nuclear Regulatory Commission. 1998b. *Exemption to 10 CFR 72.102(f)(1) Seismic Design Requirement for Three Mile Island Unit 2 Independent Spent Fuel Storage Installation.* SECY–98–071, Letter (April 8) from L.J. Callan (EDO) to the Commissioners. Washington, DC: U.S. Government Printing Office.

Nuclear Regulatory Commission. 1999. *Design of Erosion Protection for Long-Term Stabilization*. NUREG–1623, Draft Report. Washington, DC: Nuclear Regulatory Commission, Office of Nuclear Material Safety and Safeguards.

Ofoegbu, G.I., and D.A. Ferrill. 1997. Mechanical analysis of listric normal faulting with emphasis on seismicity assessment. *Tectonophysics* 284: 65–77.

Oviatt, C.G., 1997. Lake Bonneville fluctuations and global climate change: *Geology* 25: 155–158.

Parkyn, J.D. 1997. *Submittal of Calculation Package*. Letter (July 14) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Parkyn, J.D. 1998. *Response to Request for Additional Information*. Letter (May 19) to Director, Office of Nuclear Material Safety and Safeguards, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Parkyn, J.D. 1999a. *Response to Request for Additional Information*. Letter (February 10) to Director, Office of Nuclear Material Safety and Safeguards, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Parkyn, J.D. 1999b. *Request for Exemption to 10 CFR 72.102(f)(1) Seismic Design Requirement*. Letter (April 2) to M.S. Delligatti, Nuclear Regulatory Commission. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Parkyn, J.D., 2001. Data needed for NRC review of License Amendment #22. Letter (May 31) to Nuclear Regulatory Commission. LaCrosse, WI: Private Fuel Storage Limited Liability Company.

Private Fuel Storage Limited Liability Company. 2000. *Safety Analysis Report for the Private Fuel Storage Facility*. Revision 18. Docket No. 72-22. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Private Fuel Storage Limited Liability Company. 2001. *Safety Analysis Report for the Private Fuel Storage Facility*. Revision 22. Docket No. 72-22. La Crosse, WI: Private Fuel Storage Limited Liability Company.

Ramsdell, J.V., and G.L. Andrews. 1986. *Tornado Climatology of the Contiguous United States*. NUREG/CR–4461. Washington, DC: Nuclear Regulatory Commission.

Rigby, J.K. 1958. Geology of the Stansbury Mountains, Tooele County, Utah: *Utah Geological Society Guidebook*. Salt Lake City, UT: Utah Geological Society. 13.

Sack, D. 1993. *Quaternary Geological Map of Skull Valley, Tooele County, Utah*: Utah Geological Survey, Map 150. Scale 1:100,000. Tooele County, UT: Utah Geological Survey.

Sadigh, K., C.Y. Change, N.A. Abrahamson, S.J. Chiou, and M.S. Power. 1993. Specification of long-period ground motions: updated attenuation relationships for rock site conditions and adjustment factors for near-fault effects. *Proceedings of ATC–17–1 Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control, March 11–12*. Redwood City, CA: Applied Technology Council. 59–70.

Sadigh, K., C.-Y. Change, J.A. Egan, F. Makdisi, and R.R. Youngs. 1997. Attenuation relationships for shallow crustal earthquakes based on California strong motion data. *Seismological Research Letters* 68(1): 154–179.

Schmertmann, J.H. 1970. Static cone to compute static settlement over sand. *ASCE Journal of Soil Mechanics and Foundations Division*. 96(SM3): 1011–1043.

Schmertmann, J.H. 1978. *Guidelines for Cone Penetration Test, Performance and Design*. FHWA TS-78-209. Washington, D.C: U.S. Federal Highway Administration.

Simiu, E., M.J. Changery, and J.J. Filliben. 1979. *Extreme Wind Speeds at 129 Stations in the Contiguous United States.* NBS Building Science Series 118. Washington, DC: U.S. Department of Commerce, National Bureau of Standards.

Somerville, P.G., N.F. Smith, R.W. Graves, and N.A. Abrahamson. 1997. Modification of empirical strong ground motion attenuation relations to include the amplitude and duration effects of rupture directivity. *Seismological Research Letters* 68(1): 199–222.

Spudich, P., J.B. Fletcher, M. Hellweg, J. Boatwright, C. Sullivan, W.B. Joyner, T.C. Hanks, D.M. Boore, A. McGarr, L.M. Baker, and A.G. Lindh. 1997. SEA96—A new predicative relation for earthquake ground motions in extensional tectonic regimes. *Seismological Research Letters* 68(1): 190–198.

Stamatakos, J., R. Chen, M. McCann, and A.H. Chowdhury. 1999. *Seismic Ground Motion at the Private Fuel Storage Facility Site in the Skull Valley Indian Reservation*. San Antonio, TX: Center for Nuclear Waste Regulatory Analyses.

Stepp, J.C. 1972. Analysis of completeness of the earthquake sample in the Puget Sound area and its effect on statistical estimates of earthquake hazard. *Proceedings of the International Conference on Microzonation* 2: 897–910.

Stewart, J.H. 1978. *Basin-Range Structure in Western North America: A Review. Cenozoic Tectonic and Regional Geophysics of the Western Cordillera*. R.B. Smith and G.P. Eaton, eds. *Geological Society of America Memoir* 152: 1–31.

Stone and Webster Engineering Corporation. 1998. *Static Settlement of the Canister Transfer Building Supported on a Mat Foundation*. PFSF Calculation No. 05996.02.GC-14. Revision 0. Denver, CO: Stone and Webster Engineering Corporation.

Stone & Webster Engineering Corporation. 1999a. *Determination of Aquifer Permeability from Constant Head Test and Estimation of Radius of Influence for the Proposed Water Well*. PFSF Calculation No. 05996.02–G(B)–15. Revision 0. Denver, CO: Stone and Webster Engineering Corporation.

Stone and Webster Engineering Corporation. 1999b. *Seismic Analysis of Canister Transfer Building*. PFSF Calculation No. 05996.02.SC–5. Revision 1. Denver, CO: Stone and Webster Engineering Corporation.

Stone and Webster Engineering Corporation. 2000a. *Document Bases for Geotechnical Parameters Provided in Geotechnical Design Criteria*. PFSF Calculation No. 05996.02.GB–05. Revision 2. Denver, CO: Stone and Webster Engineering Corporation.

Stone and Webster Engineering Corporation. 2000b. *Stability Analyses of Storage Pads*. PFSF Calculation No. 05996.02.GB–04. Revision 6. Denver, CO: Stone and Webster Engineering Corporation.

Stone and Webster Engineering Corporation. 2000c. *Stability Analyses of the Canister Transfer Building Supported on a Mat Foundation*. PFSF Calculation No. 05996.02.GB-13. Revision 3. Denver, CO: Stone and Webster Engineering Corporation.

Stone and Webster Engineering Corporation. 2001a. *Soil and foundation parameters for dynamic soil-structure analyses, 2,000-yr return period design ground motions*. PFSF Calculation No. 05996.02.G(PO18)-2. Revision 1. Denver, CO: Stone and Webster Engineering Corporation.

Stone and Webster Engineering Corporation. 2001b. *Stability Analyses of Storage Pads*. PFSF Calculation No. 05996.02.GB–04. Revision 9. Denver, CO: Stone and Webster Engineering Corporation.

Stone and Webster Engineering Corporation. 2001c. *Stability Analyses of the Canister Transfer Building*. PFSF Calculation No. 05996.02.GB-13. Revision 6. Denver, CO: Stone and Webster Engineering Corporation.

Stone and Webster Engineering Corporation. 2001d. *Dynamic Settlement of the Soils Underlying the Site*. PFSF Calculation No. 05996.01.GB–11. Revision 3. Denver, CO: Stone and Webster Engineering Corporation.

Suppe, J. 1983. Geometry and kinematics of fault-bend folding. *American Journal of Science* 282: 684–721.

Teichert, J.A. 1959. Geology of the southern Stansbury range, Tooele County, Utah: *Utah Geological and Mineral Society Bulletin* 65.

Terzaghi, K., R.B. Peck, and G. Mesri. 1996. *Soil Mechanics in Engineering Practice, 3rd Edition*. New York: John Wiley & Sons, Inc.

Thatcher, W., G.R. Foulger, B.R. Julian, J. Svarc, E. Quilty, and G.W. Bawden. 1999. Present-day deformation across the Basin and Range province, western United States. *Science* 283: 1,714–1,718.

U.S. Army Corps of Engineers. 1990. Flood Hydrograph Package HEC–1. Davis, CA: U.S. Army Corps of Engineers, Hydrologic Engineering Center.

U.S. Army Corps of Engineers. 1997. *HEC-RAS River Analysis System*. Davis, CA: U.S. Army Corps of Engineers, Hydrologic Engineering Center.

Tokimatsu, A.M. and H.B. Seed. 1987. Evaluation of settlements in sand due to earthquake shaking. *ASCE Journal of Geotechnical Engineering Division*. 113(8): 861–878.

U.S. Department of Energy. 1994. *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*. DOE–STD–1020–94. Washington, DC: U.S. Department of Energy.

U.S. Department of Energy. 1997. *Preclosure Seismic Design Methodology for a Geologic Repository at Yucca Mountain*. Yucca MountainR/TR–003–NP. Revision 2. Washington, DC: U.S. Department of Energy.

Weichert, D.H. 1980. Estimation of the earthquake recurrence parameters for unequal observations periods for different magnitudes. *Bulletin of the Seismological Society of America* 70: 1,337–1,346.

Wells, D.W., and K.J. Coppersmith. 1993. Likelihood of surface rupture as a function of magnitude. *Seismological Research Letters* 64(1): 54.

Wells, D.W., and K.J. Coppersmith. 1994. New empirical relationships among magnitude, rupture length, rupture width, rupture area, and surface displacement. *Bulletin of the Seismological Society of America* 84: 974–1,002.

Wernicke, B.P. 1992. Cenozoic extensional tectonics of the western Cordillera. The Cordilleran Orogen: Conterminous U.S. B.C. Burchfiel, P.W. Lipman, and M.L. Zoback, eds. Boulder, CO: *Geological Society of America*. *The Geology of North America* G-3: 553–581.

Wernicke, B.P. 1995. Low-angle normal faults and seismicity: A review. *Journal of Geophysical Research* 100: 20,159–20,174.

Wernicke, B.P., and B.C. Burchfiel. 1982. Modes of extensional tectonics. *Journal of Structural Geology* 4: 104–115.

Woodward, N.B., S.B. Boyer, and J. Suppe. 1989. Balanced geological cross-sections: An essential technique in geological research and exploration. *American Geophysical Union Short Course in Geology* 6: 132.

Yarnold, J.C. 1993. Rock-avalanche characteristics in dry climates and the effects of flow into lakes: Insights from mid-tertiary sedimentary breccias near Artillery Peak, Arizona. *Geological Society of America Bulletin* 105: 345–360.

Youngs, R.R., F.H. Swan III, M.P. Power, D.P. Schwartz, and R.K. Green. 1987. Analysis of earthquake ground shaking hazard along the Wasatch Front. P.L. Gori and W.W. Hays, eds. *Assessment of Regional Earthquake Hazards and Risk Along the Wasatch Front, Utah*. Volume II. USGS Open-File Report 87-585: 1–110.