2 SITE CHARACTERISTICS

2.1 Conduct of Review

Chapter 2 of the SAR discusses the geographical location of the PFS Facility and meteorological, hydrological, seismological, geological, and volcanological characteristics of the site and the surrounding area. It describes the population distribution within and around the Reservation, land and water uses, and associated site activities. Chapter 2 of the SAR also evaluates site characteristics with regard to safety and identifies assumptions that need to be applied when evaluating safety, establishing installation design, and providing design bases in other evaluations in the SAR.

The staff evaluated site characteristics by reviewing Chapter 2 of the SAR, documents cited in the SAR, and other relevant literature. 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.

The information and analyses in SAR Chapter 2 were reviewed with respect to the applicable siting evaluation regulations in 10 CFR Part 72, Subpart E, and 10 CFR 72.122(b). Where appropriate, findings of regulatory compliance are made for the 10 CFR Part 72 requirements that are fully addressed in Chapter 2 of the SAR. Because compliance with some regulations can only be determined by the integrated review of several sections in Chapter 2 and/or other Chapters within the SAR, a finding of regulatory compliance is not made in each major section unless the specific regulatory requirement is fully addressed. However, findings of technical adequacy and acceptability are made for each section in Chapter 2, as it relates to the regulatory requirements. Upon full consideration of information presented in all Chapters of the SAR and applicable regulatory requirements, the staff concludes that the PFS Facility is in compliance with the requirements of 10 CFR Part 72.

2.1.1 Geography and Demography

This section contains the review of Section 2.1, Geography and Demography, of the SAR. Subsections discussed include (i) site location, (ii) site description, (iii) population distribution and trends, and (iv) land and water uses. The staff reviewed the discussion on geography and demography 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 that the proposed sites with design basis external events for which adequate protection cannot be provided through ISFSI design shall be deemed unsuitable for the location of the ISFSI.

- 10 CFR 72.90(e) requires that pursuant to Subpart A of Part 51 of Title 10 for each proposed site for an ISFSI, the potential for radiological and other environmental impacts on the region must be evaluated with due consideration of the characteristics of the population, including its distribution, and of the regional environs, including its historical and aesthetic values.

- 10 CFR 72.90(f) requires the facility to be sited so as to avoid to the extent possible the long-term and short-term adverse impacts associated with the occupancy and modification of floodplains.

- 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 must be identified.

- 10 CFR 72.98(b) requires that the potential regional impact due to the construction, operation or decommissioning of the ISFSI must 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) of this section 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 72.100(a) requires that the proposed site must be evaluated with respect to the effects on populations in the region resulting from the release of radioactive materials under normal and accident conditions during operation and decommissioning of the ISFSI; in this evaluation both usual and unusual regional and site characteristics shall be taken into account.

- 10 CFR 72.100(b) requires that each site must be evaluated with respect to the effects on the regional environment resulting from construction, operation, and

decommissioning for the ISFSI; in this evaluation both usual and unusual regional and site characteristics must be taken into account.

2.1.1.1 Site Location

Section 2.1.1 of the SAR, Site Location, and relevant literature cited in the SAR describes the site location. The Facility will be located within the boundaries of the Reservation of the Skull Valley Band of Goshute Indians. The Reservation is geographically located in Skull Valley, Tooele County, Utah, about 50 miles southwest of Salt Lake City, Utah, and 14 miles north of the entrance to the Dugway Proving Ground in Tooele County, Utah.

The staff reviewed the description of the site location and found it acceptable because it clearly describes the geographic location of the site, including its relationship to political boundaries and natural anthropogenic features. The maps provided in the SAR are acceptable because they provide sufficient detail, which is needed for review of 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 the regulatory of requirements in 10 CFR 72.90(a), 72.90(e), and 72.98(a) with respect to this issue.

2.1.1.2 Site Description

Section 2.1.2 of the SAR, Site Description, and relevant literature cited in the SAR describe the site with maps to delineate the site boundary and controlled area. The proposed site is located on a typical valley floor of the local Basin and Range topography. The Stansbury Mountains lie to the east of the site and separate the site from Tooele City, Utah, about 27 miles to the northeast. The Cedar Mountains are approximately 14 miles to the west and separate the Facility from portions of the Utah Test and Training Range within the Great Salt Lake Desert. Skull Valley, Utah, has little population and limited agriculture, although a cattle ranch is located on the north border of the Facility. The site is located within the northern boundary of the Sevier B military operating area, utilized by military aircraft traveling to and from the Utah Test and Training Range and Hill Air Force Base.

Access to the controlled area will be restricted by typical range fencing, and ingress and egress of site personnel will be controlled. Skull Valley Road (Federal Aid Secondary Road 108) is about 1 mile east of the site and connects I-80 to the north with State Route 199 to the south. Traffic on this road is local, either to the Reservation or to the Dugway Proving Ground entrance about 14 miles south of the proposed site (the northern border of the Dugway Proving Ground is about 9 miles from the site). The orientation of the Facility structures with respect to nearby roads, railways, and waterways is shown on various maps and plots, and there is no obvious way in which traffic on adjacent transportation links can interfere with Facility operations.

The site (approximately 99 acres of restricted area for cask storage and a total controlled area of about 820 acres) has about a 15-foot elevation change across the facility with the south side higher than the north side. Local vegetation is sparse due to meager rainfall and extended drought periods. A shallow dry wash is found to the west of the site, and the primary floodway has been identified to the east of the proposed pad site.

The staff reviewed the site description and relevant literature cited in the SAR. The staff finds that the site description is adequate because the descriptive information and maps clearly delineate the site boundary and controlled area. The maps have a sufficient level of detail and are of appropriate scale and legibility that is required for the review of the site and Facility. The information is also acceptable to determine distances between the Facility and nearby facilities and cities. 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), 72.90(e), and 72.98(a) with respect to this issue.

2.1.1.3 Population Distribution and Trends

Section 2.1.3 of the SAR, Population Distribution and Trends, and relevant literature cited in the SAR describes the population distribution and trends. The population data used in the SAR were derived through local interrogation. Within 5 miles of the proposed site, there are two tribal homes approximately 2 miles southeast of the Facility, additional residences on the Reservation, about 3.5 miles east-southeast of the site, and two private farm residences located approximately 2.75 and 4.0 miles to the northeast of the site. The population within 5 miles of the site is about 36 people. Ten miles east-southeast of the proposed site is the small residential community of Terra with an estimated population of 120. The town of Dugway, with an estimated population of 1,700, is located about 12 miles south of the Facility. The permanent population of the immediate area during the period of operation of the Facility is not expected to grow. As described by the applicant, it is expected that the construction, operation, and decommissioning of the Facility will have a negligible effect on the overall population of the region. No transient or institutional populations are present within 5 miles of the Facility, and no public facilities are anticipated to be located in the vicinity. Based on this information, the applicant concludes that it is likely that the effect of the Facility on the population distribution and growth trends in Skull Valley, Utah, will be small, if any.

The staff reviewed the information presented in the SAR and has determined that the population distribution and trends in the region have been adequately described and assessed. The source of the population data used in the SAR is appropriate and the basis for population projections is reasonable. The staff found that 10 CFR 72.98(c)(1) is met because the region has been appropriately investigated with respect to the present and future character and distribution of the population. 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 regulatory requirements in 10 CFR 72.90(e), 72.98(a), 72.98(b), 72.100(a), and 72.100(b) with respect to this issue.

2.1.1.4 Land and Water Uses

Section 2.1.4 of the SAR , Uses of Nearby Lands and Waters, and relevant literature cited in the SAR (Bureau of Land Management, 1985, 1986, 1988, 1992) describe land water uses. Land use within the Reservation boundary includes residential use by tribal members. Until 1999, it also included the leased operation (since 1975) of the Tekoi Rocket Engine Test Facility on the south side of Hickman Knolls by Allied Techsystems. There is no current lessee

of the Tekoi facility.[1] Within the 5-mile radius of the proposed site there are approximately 28,000 acres of property owned by the Bureau of Land Management, 13,000 acres of Indian-owned land, and 9,000 acres of privately owned land. The principal land use is for grazing of livestock. The grazing quality is considered to be fair to poor, and in decline. Sheep and cattle are grazed seasonally on much of the acreage within the 5-mile radius and are sequentially pastured within the total acreage. Fifty-five percent of the Bureau of Land Management property within the 5-mile radius of the Facility is within the Pony Express Resource Area and is open to off-highway vehicle use, dispersed camping, and hunting. There are no designated camping areas, off-highway vehicle trails, or roads other than the Skull Valley Road within a 5-mile radius of the proposed site.

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 5,300 acre-feet of water per year. The applicant estimates water needs at no more than 9,300 gallons per day during construction and on average 1,729 gallons per day during operation. Assuming a conservative 365 days in a year, the water usage is estimated to be 10.4 acre-feet per year during construction activities and 1.93 acre-feet per year for operational activities. Therefore, the projected amount of water used during construction activities is about 0.2 percent (10.4 acre-feet divided by 5,300 acre-feet used per year) of current total water production estimated in Skull Valley and 0.036 percent (1.93 acre-feet divided by 5,300 acre-feet used per year) for operations. These water-use amounts attributed to the Facility are very small when compared to the total ground water budget and should have no perceptible impact on current water use.

The applicant indicates that there will be little projected population growth near the Facility because it is unlikely that the permanent population within 5 miles of the proposed Facility would change significantly during the proposed license period and due to the remoteness and extreme low population density of the area (36 persons within a 5-mile radius), no facilities such as hospitals, prisons, and recreational areas are located and planned within the 5-mile study area. Based on preliminary testing of the onsite monitoring well, the applicant determined that operation of the Facility water well will have no measurable offsite effects on existing groundwater quality or levels (Stone & Webster Engineering Corporation, 1999a). Thus, future impacts on water use are also considered to be minimal.

The staff reviewed the description of the land and water use in the SAR and information (Bureau of Land Management, 1985, 1986, 1988, 1992) cited in the SAR for the region and found that it has been adequately described and assessed. The staff accepts the use of land and water information provided by the Bureau of Land Management. The region has been

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[1]The Tekoi Rocket Engine Test Facility was in operation when the NRC initiated its review of the PFS Facility application; therefore, the potential impact of operations in the Tekoi facility is considered in this SER.

investigated as appropriate with respect to consideration of present and projected future uses of land and water within the region. 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 72.98(a), 72.98(b), and 72.98(c) with respect to this issue.

2.1.2 Nearby Industrial, Transportation, and Military Facilities

Section 2.2 of the SAR, Nearby Industrial, Transportation, and Military Facilities, and relevant literature cited in the SAR (Donnell, 1999a,b,c) describes nearby industrial, transportation, and military facilities and identifies potential hazards from these facilities. This information is necessary to evaluate credible scenarios involving manmade facilities that may endanger the PFS Facility site. The staff reviewed nearby industrial, transportation, and military facilities with respect to the following regulatory requirements:

- 10 CFR 72.94(a) requires that the region must be examined for both past and present man-made facilities and activities that might endanger the proposed ISFSI. The important potential man-induced events that affect the ISFSI design must be identified.

- 10 CFR 72.94 (b) requires that information concerning the potential occurrence and severity of such events must be collected and evaluated for reliability, accuracy, and completeness.

- 10 CFR 72.94 (c) requires that appropriate methods must be adopted for evaluating the design basis external man-induced events, based on 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 must be identified.

- 10 CFR 72.98(b) requires that the potential regional impact due to the construction, operation or decommissioning of the ISFSI must 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 72.100(a) requires that the proposed site must be evaluated with respect to the effects on populations in the region resulting from the release of radioactive materials under normal and accident conditions during operation and

decommissioning of the ISFSI; in this evaluation both usual and unusual regional and site characteristics shall be taken into account.

- 10 CFR 72.100(b) requires that each site must be evaluated with respect to the effects on the regional environment resulting from construction, operation, and decommissioning of the ISFSI; in this evaluation both usual and unusual regional and site characteristics must be taken into account.

Summary of Review

The identification of potential hazards includes identification of facilities and determination of credible scenarios that may endanger the PFS Facility. The facilities identified by the applicant include the Dugway Proving Ground, the Tekoi Rocket Engine Test Facility, the Utah Test and Training Range, the Tooele North Army Depot Area, and the Tooele South Army Depot Area.

The Dugway Proving Ground is a federal site that performs activities that include testing and disposing of chemical and biological agents. The site also contains the Michael Army Airfield, which is used by military aircraft and potentially for vehicle landings of the X-33 suborbital demonstrator. Its entrance is about 14 miles east-southeast of the PFS Facility. The Cedar Mountains (elevation greater than 5,300 feet) lie between this site and the PFS Facility. The Tekoi Rocket Engine Test Facility is a commercial facility that has performed high explosive and rocket motor testing. This facility is about 2.3 miles south-southeast of the PFS Facility on the south side of Hickmam Knolls (a rock formation with elevation greater than 4,600 feet). The Utah Test and Training Range is a federal site with activities that test air-to-ground and air-to-air munitions. The Tooele North Army Depot Area is a federal site located 17 miles east-northeast of the PFS Facility. The Tooele South Army Depot Area is also a federal site located about 22 miles east-southeast of the PFS Facility. It performs incineration of retired nerve agents. The Stansbury Mountains (elevation greater than 8,000 feet) lie between the Tooele site and the PFS Facility.

The applicant identified the potential crash of civilian or military aircraft onto the site as a potential hazard. Civilian aircraft with a potential to crash at the PFS Facility site include aircraft taking off and landing at Salt Lake City International Airport, aircraft flying along jet routes J-56 and V-257, and general aviation aircraft flying in the vicinity of the proposed site. Military aircraft that have a potential to crash at the PFS Facility site include aircraft taking off and landing at Michael Army Airfield at the Dugway Proving Ground, aircraft flying military route IR-420, aircraft flying to and from the Utah Test and Training Range and Hill Air Force Base, helicopters flying near the site, and the X-33 suborbital demonstrator vehicle landing at Michael Army Air Field.

The staff finds that all nearby military and industrial facilities that may present a hazard to the PFS Facility have been adequately identified. The potential hazards from these facilities, including from military aircraft flying through Skull Valley and helicopter flights over the Utah Test and Training Range and Skull Valley, are assessed in Chapter 15 of this SER.

2.1.3 Meteorology

The staff has reviewed the information presented in Section 2.3 of the SAR, Meteorology. Subsections discussed below include (i) regional climatology, (ii) local meteorology, and (iii) onsite meteorological measurement program. The staff reviewed the discussion on meteorology 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 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 shall be deemed unsuitable for the location of the ISFSI.

- 10 CFR 72.90(e) requires that, pursuant to Subpart A of Part 51 of Title 10, for each proposed site for an ISFSI, the potential for radiological and other environmental impacts on the region must be evaluated with due consideration of the characteristics of the population, including its distribution, and of the regional environs, including its historical and aesthetic values.

- 10 CFR 72.90(f) requires the facility to be sited so as to avoid to the extent possible the long-term and short-term adverse impacts associated with the occupancy and modification of floodplains.

- 10 CFR 72.92(a) requires that natural phenomena that may exist or that can occur in the region of a proposed site 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 must be identified.

- 10 CFR 72.98(c) requires that those regions identified pursuant to paragraphs 10 CFR 72.98(a) and (b) 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 72.122(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 be 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.

2.1.3.1 Regional Climatology

Section 2.3.1 of the SAR, Regional Climatology, and relevant literature cited in the SAR (National Oceanic and Atmospheric Administration, 1960, 1992; Ashcroft et al., 1992) describe the regional climatology associated with the Facility site. The applicant used climatologic data collected at the Salt Lake City International Airport (approximately 50 miles north of the proposed site) and at the Dugway Proving Ground (within 14 miles of the proposed site) to characterize the climate in Skull Valley, Utah. Regional data have been augmented with data collected at a meteorologic station established in Skull Valley, Utah, especially for the purpose of verifying the regional climatic and meteorologic data. Long-term weather data and severe weather data from the National Weather Service (National Oceanic and Atmospheric Administration, 1975–1995; Ramsdell and Andrews, 1986; Grazulis, 1993) are discussed. The information presented includes (i) weather influence of terrain; (ii) regional temperature, precipitation, atmospheric moisture, and winds; (iii) severe weather including maximum and minimum temperatures, temperature ranges, freeze-thaw cycle, degree days, design temperature, subsoil temperatures, extreme winds, tornadoes, dust devils, hurricanes, and tropical storms, precipitation extremes, thunderstorms and lightning, snow storms and snow

accumulation, hail and ice storms, and other phenomena; (iv) station pressure; and (v) air density (National Oceanic and Atmospheric Administration, 1975–1995).

The staff reviewed the regional climate data and discussions presented in the SAR, and found it acceptable. It is acceptable because reliable data sources, such as the National Weather Service, were used. In addition, all relevant data including weather data from nearby regional and local meteorological stations, were appropriately summarized to define the expected climatology of the site region. The information on severe weather data (National Oceanic and Atmospheric Administration, 1975–1995; Ramsdell and Andrews, 1986; Grazulis, 1993) is an acceptable source of data for the development of structural design criteria in Chapter 3, Principal Design Criteria, of the SAR regarding strong wind and windborne missiles.

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 the regulatory requirements of 10 CFR 72.90(a), 72.90(b), and 72.122(b) with respect to this issue.

2.1.3.2 Local Meteorology

Section 2.3.2 of the SAR, Local Meteorology, and relevant literature cited in the SAR (Ashcroft et al., 1992) describes local meteorology of the site. The SAR provides the maximum temperature data for Salt Lake City, Utah, (about 50 miles northeast to the site at an elevation of approximately 4,220 feet above mean sea level), as a part of the regional climatology information, based on the long-term meteorological data collected by the National Weather Service at the Salt Lake City International Airport (National Oceanic and Atmospheric Administration, 1992). Because the Stansbury and Oquirrh Mountains, with elevations exceeding 10,000 feet above mean sea level, are located between Salt Lake City, Utah, and the site, meteorological data collected in Skull Valley, Utah, are also needed to characterize the local conditions. The applicant provided the temperature data recorded at Dugway (approximately 12 miles south of the site at an elevation of 4,340 feet above mean sea level) and Iosepa South Ranch (about 12 miles north of the site at an elevation of 4,415 feet above mean sea level). These data are based on Ashcroft et al. (1992). The recorded period at Dugway was from 1950 to 1992 and the recorded period at Iosepa South Ranch was from 1951 to 1958. The applicant also provided annual average and average daily maximum temperatures for the month of July recorded at the proposed site meteorological tower during 1997 and 1998. Temperatures recorded at different sites are summarized in Table 2-1 of this SER.

Table 2.3-4 of the SAR provides the recorded temperatures at the site and at nearby locations. The annual average temperature measured at Salt Lake City is about 52 °F and at Skull Valley is 49 °F. Maximum and minimum average monthly temperatures recorded at Skull Valley are about 75 ° and 23 °F, respectively. Average daily maximum temperature recorded at the site is 90 °F, and the average daily minimum is 10 °F. As shown in Table 2.3-4 of the SAR and summarized in Table 2-1 of this SER, temperatures recorded at the site through the onsite measurement program during the period December 1996 and December 1998 correlate well with temperatures recorded at nearby sites for significantly longer periods.

Maximum solar radiation recorded during the onsite meteorological monitoring program at the site is 685 W/m^2 for a 12-hour period. Maximum solar insolation recorded at Salt Lake City during a 12-hour period in the 30-year Solar and Meteorological Surface Observational Network is 730 W/m^2. Relative humidity at the site varied from 3.9 to 98.6 percent in 1997. The average humidity in 1997 was 58.7 percent. The wind at the proposed site is similar to that at Salt Lake City. The prevailing wind direction is southeast or south-southeast throughout the year (Private Fuel Storage Limited Liability Company, 2000). Highest and lightest monthly average speeds recorded at the site are 9.6 and 7.4 mph. The average wind speed recorded for the two-year periods at the site is 8.7 mph.

The applicant did not develop atmospheric diffusion estimates for the facility based on the measurement program. However, the applicant assumed design basis atmospheric diffusion characteristics based on Regulatory Guide 1.145 (Nuclear Regulatory Commission, 1983). These design basis characteristics are a wind speed of 1 m/sec, atmospheric stability class F, and no consideration of plume meander.

The staff reviewed the local meteorological data and discussions presented in the SAR and found them acceptable because reliable data sources such as the National Weather Service were used, and the data from December 1996 to March 1998 are appropriately summarized.

Table 2-1. Temperature at the Private Fuel Storage Facility site and nearby cities

Temperature	Salt Lake City	Dugway	Iosepa South Ranch	Site Meteorological Tower
Average Daily Maximum (°F)				
June	82.3†/83*	84.7†/85*	86*	78*
July	92.2†/93*	94.4†/94*	95*	90*
August	89.4†/90*	91.3†/91*	93*	90*
Average Daily Minimum (°F)				
June	55.4†/53*	53.2†/53*	45*	46*
July	63.7†/62*	61.9†/62*	52*	54*
August	61.8†/60*	59.3†/59*	53*	57*
Annual Average (°F)	52*	51*	50*	49*
Monthly Average (°F)				
June	69.1†/68*	69.0†/69*	66*	63*
July	77.9†/78*	78.2†/78*	74*	74*
August	75.6†/75*	75.3†/75*	73*	75*
Record High (°F)				
June	104†	107†	NA	93.4*
July	107†	109†	NA	99.3*
August	104†	108†	NA	96.6*
Record Low (°F)				
December	−15†	−27†	NA	−4.7*
January	−22†	−25†	NA	−7.0*
February	−14†	−29†	NA	4.7*

† Ashcroft et al., 1992
* Private Fuel Storage Limited Liability Company, 2000
NA Not Available

The staff reviewed the topographic maps to determine the affects of meteorology on erosion at the site. The maps indicate that there is approximately 15 feet of relief across the proposed site and the site slopes from south to north. Staff analysis of the slope and the expected meteorologic environment indicates that the slopes will be stable and the site will not experience significant erosion. The staff determined that the current information presented in the SAR 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 the regulatory requirements of 10 CFR 72.92(a), 72.98(a), 72.98(c)(3), and 72.122(b).

2.1.3.3 Onsite Meteorological Measurement Program

Section 2.3.3 of the SAR, Onsite Meteorological Measurement Program, describes the onsite meteorological measurement program. The applicant described the meteorologic instrumentation that was used, including detail on its emplacement and operation. Actual siting, types of sensors, recordings of sensor output, instrument surveillance plans, and data acquisition and reduction methods are included in the SAR. Examples of data collected from the instruments are provided. As discussed in Section 2.1.3.2 of this SER, the applicant did not develop atmospheric diffusion estimates for the facility based on the measurement program.

The staff reviewed the information on the onsite meteorological measurement program and found it acceptable because:

- The onsite meteorologic measurement program has been adequately described such that potential meteorological effects on the Facility can be identified and assessed.

- The onsite meteorologic measurement program has been adequately described such that the regional extent of external phenomena, manmade or natural, used as a basis for the design of the Facility, can be identified.

- The onsite meteorologic measurement program has been adequately described such that the regional impact on the population or the environment due to the construction, operation, or decommissioning of the Facility can be identified.

The staff has determined that the current 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 the regulatory requirements of 10 CFR 72.92(a), 72.98(a), 72.98(c)(3), and 72.122(b) with respect to this issue.

2.1.4 Surface Hydrology

The staff has reviewed the information presented in Section 2.4 of the SAR, Surface Hydrology. Subsections discussed include (i) hydrologic description, (ii) floods, (iii) probable maximum flood on streams and rivers, (iv) potential dam failures, (v) probable maximum surge and seiche flooding, (vi) probable maximum tsunami flooding, (vii) ice flooding, (viii) flood protection requirements, and (ix) environmental acceptance of effluents. The staff reviewed the discussion on surface hydrology 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 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 72.90(e) requires that, pursuant to Subpart A of Part 51 of Title 10, for each proposed site for an ISFSI, the potential for radiological and other environmental impacts on the region must be evaluated with due consideration of the characteristics of the population, including its distribution, and of the regional environs, including its historical and aesthetic values.

- 10 CFR 72.90(f) requires the facility to be sited so as to avoid to the extent possible the long-term and short-term adverse impacts associated with the occupancy and modification of floodplains.

- 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 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 72.122(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 be 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.

2.1.4.1 Hydrologic Description

The Facility will be located in Skull Valley, Utah, between the Stansbury and East Cedar Mountain ranges. The Skull Valley watershed is approximately 50 miles long and 22 miles wide at its widest point, sloping gently northward to the Great Salt Lake. The site is situated near the center of the valley approximately 24 miles south of Interstate Highway 80 and the Great Salt Lake. The watershed through the central valley is an alluvium comprised of poorly sorted coarse to fine grained deposits resulting in a relatively high permeability. A U.S. Department of Agriculture (USDA) soil Curve Number of 70 was determined appropriate for the alluvium. No perennial streams are observed in the Skull Valley watershed. Most runoff infiltrates into the alluvium and recharges the subsurface groundwater. Surface runoff drains through channels formed during wet years; however, a continuous system of drainage channels is not apparent in the area adjacent to the Facility.

The tributary watershed of the Facility location drains approximately 334 sq mi, which includes the west slope of the Stansbury Mountains, west slope of the Onaqui Mountains, north slope of Lookout Mountain, east slope and south tip of the lower Cedar Mountain Range, and the valley lowlands. The watershed was subdivided into Basin A (tributary to the southeasterly side of the Facility) and Basin B (tributary to the southwesterly side of the Facility) as shown in Figure 1 of Donnell [1999f, calculation 0599602–G(B)–17, Revision 1]. A slight ridge line extends from the Facility northerly to Hickman Knolls and then westerly toward the East Cedar Mountain range. The ridge naturally segments the watershed into drainage basins of approximately 270 square miles (Basin A) and 64 square miles (Basin B) for flood analysis purposes.

Structures

The Facility will be situated on approximately 99 acres located near the center of the valley, approximately 26 miles from the southerly end of the watershed. The casks will be placed on storage pads at an elevation 4,475 feet above mean sea level at the southwest corner, falling to

elevation 4,462 feet above mean sea level at the northeast corner. Four predominant structures will be integrated into the Facility that have flood impact potential as shown in Figure 2 of Donnell [1999f, calculation 0599602–G(B)–17, Revision 1]. A berm will be constructed along the upstream side of the Facility to divert potential flood waters around the cask storage area. A railroad embankment will be constructed extending from the west side of the valley and linking into the diversion berm. An access road will link the roadway located on the east side of the valley to the Facility. In conjunction with the access road, a diversion berm (road berm) will be constructed perpendicular to the road (immediately east of the Facility) to span a gap in the natural ridge, thereby isolating the flood waters from Basins A and B. The structures are to be designed as an additional measure to ensure that flood water surface elevations remain below cask storage pad elevations.

The staff reviewed the hydrologic description and found it acceptable because the basic information regarding surface hydrology of the site and the vicinity has been described in sufficient detail for review of the license application. 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 72.90, 72.92(a), 72.98(a), and 72.98(b) with respect to this issue.

2.1.4.2 Floods

A probable maximum flood (PMF) analysis was performed for the proposed site based on state of the art procedures and practices outlined by the U.S. Army Corps of Engineers (1997). The analysis comprised delineation of the tributary drainage basins, determination of the appropriate rainfall depths, simulation of the storm and routing of the runoff hydrographs, determination of the flood water surface elevations near and through the PFS Facility site, and an evaluation of how the proposed structures affect site safety. Based on this analysis, the site is a flood dry site (i.e., the cask storage pads are elevated out of the adjacent flood plain), although the site will be temporarily isolated during a major flood event.

Little information is available pertaining to historic flooding in the Skull Valley watershed. The lack of a definitive, continuous stream channel or drainage feature throughout the basin indicates that drainage does not occur during frequent, low-intensity precipitation events (i.e., 2-year frequency or less storm events). The presence of segmented drainage channels in areas adjacent to the site indicates that drainage and subsequent channelization occurs, probably derived from less frequent, high-intensity precipitation events (i.e., 10-year storm events).

The staff reviewed the PMF analysis and found it acceptable because the surface water flooding that may directly affect the safety or environmental impact has been sufficiently investigated and assessed. 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 72.90(c), 72.90(d), 72.90(f), and 72.122(b) with respect to this issue.

2.1.4.3 Probable Maximum Flood on Streams and Rivers

The site is situated in an area that will be isolated (with the assistance of embankments) from the major floods derived from both Basins A and B. The site was evaluated in the SAR for the PMF scenario to ensure that a flood dry condition prevailed. The staff performed an independent flood analysis for both Basins A and B.

Probable Maximum Precipitation

The PMF is derived from the probable maximum precipitation (PMP) (rainfall) that may occur in each drainage basin. Using Hydrometeorological Report No. 49 (HMR 49) (National Weather Service, 1977), PMP values were determined to be 12.2 inches for a 72-hour General Storm and 10.2 inches for a 6-hour Local (thunderstorm) Storm. These precipitation amounts incorporate appropriate regional and elevation reduction factors. The staff reviewed the PMP values by using HMR 49 in accordance with NUREG–1623 (Nuclear Regulatory Commission, 1999), and determined the applicant's PMP values to be acceptable.

Curve Number

The Skull Valley alluvium was determined by the applicant to have a USDA soil Curve Number of 70 as indicated in the SAR. This value is based on the assumption that the soil is in a natural and unsaturated condition, thereby allowing the rainfall and runoff to infiltrate into the soil. However, a PMF analysis stipulates that the antecedent soil condition is saturated preventing infiltration and maximizing potential runoff. Therefore, the SAR elevated the Curve Number to 96 to account for the saturated soil conditions. A Curve Number of 96 was applied for the PMF computation. The staff computed the saturated soil Curve Number to be 85. Therefore, the staff considers the use of the conservative Curve Number of 96 as an acceptable value to describe the alluvium as proposed in the SAR.

Time of Concentration

An important factor in determining the magnitude of the PMF is estimating the time of concentration. The time of concentration is the time (hours) required for runoff to flow overland from the most distant point in the basin to the point of interest (site). The Kirpich (1964) method was applied resulting in times of concentration of 11 hours in Basin A and 4.2 hours in Basin B. The staff performed independent calculations for the times of concentration for both basins—Basin A resulted in a time of concentration of approximately 12 hours and Basin B yielded a time of concentration of approximately 4 hours. In Basin A, the applicant's 11-hour estimate will yield a more conservative peak flood discharge (larger) than the 12-hour value derived by the staff. In Basin B, the difference between 4 hours and 4.2 hours is considered negligible by the staff. Therefore, the staff considers the times of concentration presented in the SAR as acceptable input values for estimating the flood peak discharge.

Flood Peak Discharge Determination

The time of concentration, Curve Number, drainage area, and PMP rainfall depth were input into the HEC-I Flood Hydrograph Package (U.S. Army Corps of Engineers, 1990) to determine the PMF peak discharges, for both the General Storm and Local Storm options, for both Basin

A and Basin B. It was determined that the PMF for Basin A (Local Storm) yielded a peak flood discharge of 85,000 ft^3/sec. The staff independently performed a similar analysis yielding a PMF peak discharge of 85,800 ft^3/sec. Therefore, the staff considers the PMF peak discharge of 85,000 ft^3/sec as an acceptable value. The PMF peak discharge for Basin B (Local Storm) was determined by the applicant to be 102,000 ft^3/sec. The staff independently calculated the peak discharge yielding a peak discharge of approximately 100,000 ft^3/sec. The applicant's PMF is larger than that estimated by the staff and is therefore considered more conservative and acceptable for the flood impact analysis.

Flood Impacts on Site Structures

The PMF peak discharges for Basins A and B were routed to and through the site using HEC-RAS (U.S. Army Corps of Engineers, 1997). HEC-RAS is a numerical hydraulic routing simulation program that translates the PMF hydrograph output from HEC-I into water surface elevations throughout the site as a function of the site contours and structures. The water surface elevations from HEC-RAS are then compared to the proposed elevations of the critical components of the storage area to determine whether the components are flood dry. Flood water surface elevations were identified at each of the four structural components presented in Section 2.1.4.1 and potential impacts to the cask storage pads were determined as discussed below.

The SAR analysis indicates that the PMF (Basin A) peak discharge water surface elevation at the upstream face of the roadway embankment is 4,506.5 feet above mean sea level. The earth berm with top elevation of 4,507.5 feet above mean sea level contains the flood flow. The PMF overtops the low point of the access road embankment (elevation 4,502 feet) by approximately 4.5 feet. The cask storage pads are located downstream from the embankment where the flood water surface elevation is approximately 4.5 feet lower than the pad elevation. The PMF water surface elevation adjacent to the northeast corner of the Facility is approximately 4,456.8 feet above mean sea level. Therefore, the cask storage area is approximately 5 feet above the PMF water surface elevation and will be flood dry.

The staff has independently computed the PMF water surface elevations for a flood discharge of 85,800 ft^3/sec (Basin A) through the east channel area adjacent to the site. The staff computations yielded water surface elevations of 4,444.3 feet downstream of the roadway embankment, 4456.7 feet adjacent to the roadway embankment, and 4,477.4 feet upstream of the roadway embankment above mean sea level. The SAR presented water surface elevations of 4,444.2 feet, 4,456.7 feet, and 4477.4 feet above sea level for the same locations, respectively. Based upon the staff computation, the staff has determined that the cask storage area will remain flood dry. Therefore, the staff found the applicant's analysis for Basin A to be acceptable.

The SAR analysis indicates that the PMF (Basin B) will overtop the railroad embankment (elevation 4,475 feet above mean sea level) by approximately 3.2 feet at an elevation of 4,478.2 feet above mean sea level. The berm constructed immediately upstream of the Facility will have a top elevation of 4,480 feet above mean sea level and extend above the PMF water surface elevation approximately 1.8 feet (freeboard). Flood waters do not impact the south face of the berm. The PMF water surface elevation at the northeast corner of the site is at an elevation of 4,458 feet above mean sea level. The cask storage pad elevation is 4,462 feet

above mean sea level, indicating the pad is approximately 4 feet above the PMF water surface elevation. The staff reviewed the applicant's analysis and agrees that the cask storage pad will remain flood dry during the Basin B PMF event. Therefore the staff found the applicant's analysis for Basin B to be acceptable.

As discussed, the staff reviewed the analysis for PMF on streams and rivers and found it acceptable because the applicant has demonstrated that the potential flood scenarios will not result in flooding of the cask storage pad. 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 the regulatory requirements of 10 CFR 72.90(c), 72.90(d), 72.90(f), and 72.122(b) with respect to this issue.

2.1.4.4 Potential Dam Failures (Seismically Induced)

There are no water storage, flow control, or embankment structures in the watershed upstream of the site. Therefore, there are no potential impacts on the site from dam or control structure failure. The only potential embankment failures that may occur during a flooding event are the railroad embankment, the Facility berm, the access road embankment, and the road berm. Therefore, each scenario presents a different potential impact on the site as discussed below.

Failure of the railroad embankment will result in the flood waters concentrating through a breach and then resuming their northerly flow in the floodway. The water surface elevation over the embankment will be lowered as the cross sectional area of the flood flow increases. The flood flow water surface elevation will be a minimum of 4 feet below the cask pad top elevation. The railroad embankment failure will isolate the site until the embankment is repaired. Therefore, failure of the railroad embankment has no flood impact on the cask storage area.

Failure of the Facility upstream berm is highly unlikely since flood water will not contact the upstream face of the embankment. The only point of flood water contact with the Facility berm is at the interface of the railroad embankment and the west component of the Facility berm. Should the berm fail, the flood water surface elevation could potentially rise approximately 3 feet. However, there will be a minimum 1-foot differential between the water surface elevation and the cask pad top elevation, thereby keeping the casks dry. Therefore, failure of the Facility berm has no flood impact on the cask storage area.

In the event that the access road embankment fails, a breach will result, thereby funneling the flood flows through a concentrated area. The breach represents an increase in flood flow cross-sectional area, thereby reducing the water surface elevation in the vicinity of the breach. Flood flows will expand into the receiving flood plain downstream of the roadway resuming its course as outlined in the PMF analysis. The primary impact will be that the site will be isolated until road repairs can be performed. The flood water surface elevation will be a minimum of 4.5 feet below the cask pad top elevation. Therefore, the staff has determined that there will be no flooding impacts to the cask storage area.

The purpose of the road berm is to confine flows and maintain a separation of Basin A and Basin B flood waters. In the event that the road berm fails, flood water will reach the Facility berm and then be diverted back to the Basin A flood plain east of the Facility. In the event the road berm should fail, the water surface elevation will rise approximately 3 feet. However, at

least a 2-foot differential will exist from the water surface elevation to the cask pad top elevation. Therefore, failure of the road berm has no flood impact on the cask storage area.

Failure of any embankment or combination of embankments resulting from the PMF does not impact the safety of the cask storage because the pads remain dry in all scenarios. As a result, the cumulative effect of these embankments is not important to safety and, consequently, is not presented.

The staff reviewed the analysis for potential dam failures and found it acceptable because the applicant has demonstrated that the potential embankment failures will not result flooding of the cask storage pad. 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 the regulatory requirements of 10 CFR 72.90(c), 72.90(d), 72.90(f), and 72.122(b) with respect to this issue.

2.1.4.5 Probable Maximum Surge and Seiche Flooding

The nearest body of water is the Great Salt Lake, located over 24 miles north of the site. The site elevation is approximately 460 feet above the lake. A wave over 400 feet high would have to be generated at the Great Salt Lake and travel 24 miles to reach the site. Therefore, surge or seiche flooding would not impact the site.

The staff reviewed the discussion on probable maximum surge and seiche flooding and found it acceptable because this phenomenon will not impact the site.

2.1.4.6 Probable Maximum Tsunami Flooding

The site is not located adjacent to a coastal area. Therefore, flooding attributed to seismically-induced ocean waves is not applicable to the site.

The staff reviewed the discussion on probable maximum tsunami flooding and found it acceptable because this phenomenon will not impact the site.

2.1.4.7 Ice Flooding

The Facility is 24 miles from the nearest body of water. Closer to the site, the pooling or ponding of water is prevented by the semiarid climate and geologic conditions present. Therefore, ice flooding will not impact the site.

The staff reviewed the discussion on probable maximum ice flooding and found it acceptable because this phenomenon will not impact the site.

2.1.4.8 Flood Protection Requirements

The proposed cask storage pad elevations remain flood dry during the PMF event.

2.1.4.9 Environmental Acceptance of Effluents

The applicant states the sanitary sewer is the only liquid release during site operations and will not contain radioactive effluents.

The staff reviewed the discussion on environmental acceptance of effluents and found it acceptable because there will be no radioactive effluents.

2.1.5 Subsurface Hydrology

The staff has reviewed the information presented in Section 2.5, Subsurface Hydrology, of the SAR. Subsections discussed include (i) regional characteristics, (ii) site characteristics, and (iii) contaminant transport analysis. The staff reviewed the discussion on subsurface hydrology with respect to the following regulatory requirements:

- 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 impacts due to the construction, operation or decommissioning of the ISFSI must 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 72.122(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 be 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.

2.1.5.1 Regional Characteristics

Skull Valley, the proposed location of the Facility, is a north-trending valley that extends from the Onaqui Mountains to the southwest shore of the Great Salt Lake. This valley is bordered by the Cedar Mountains to the west and the Stansbury Mountains to the east. Most of the precipitation that falls in the higher elevations runs off the steep hillsides as spring snowmelt, with little infiltration into the mountain blocks. Water enters the valley-fill aquifers through an extensive recharge area consisting mainly of alluvial fans at the base of the mountains. The long-term average annual runoff from the uplands is about 32,000 acre-feet. The average annual groundwater discharge and recharge is between 30,000 and 50,000 acre-feet, with evapotranspiration accounting for 80–90 percent of the discharge.

Valley-fill consists of inter-stratified colluvium, alluvium, lacustrine, and fluvial deposits with minor ash and some Eolian material. The coarser deposits are generally near the perimeter of the valley, grading into well-sorted sand and gravel and interlayered with lacustrine silt and clay toward the center of the valley. Thick beds of clay exist in some areas and may create local, confined aquifers where they interfinger with sand and gravel along the alluvial fans. The Salt Lake Group of the Tertiary age comprises most of the valley-fill with a thickness ranging from 2,000 to over 8,000 feet. The Tertiary and older Quaternary deposits are slightly to highly permeable. The deeper deposits contain some volcanic deposits and have reduced permeability due to greater consolidation. The Tertiary and Quaternary deposits contain most of the usable groundwater in the valley. The valley floor is underlain by Quaternary and Holocene sediments that generally have low permeability. Most of the surface runoff ponds in discontinuous drainage channels until it evaporates.

Groundwater flows northward to the Great Salt Lake. The annual volume of underflow out of the valley is estimated at 800 acre-feet with a transmissivity of 2,675 ft/day^2. Annual discharge from pumping is estimated at 5,000 acre-feet and is not believed to have changed significantly in the past 30 years. Most of the domestic wells are developed in the unconsolidated alluvial fan deposits along the east side of the Skull Valley. This area provides most of the local recharge and yields high quality groundwater. Groundwater is generally between 110–160 feet below ground in this area. Some irrigation and stock wells show artesian conditions in the valley due to confining layers of lake clays. Most wells are drilled to depths of 250–500 feet but maintain a static water depth of 100 feet or less.

The groundwater quality depends on well proximity to the bordering mountain ranges. The groundwater along the base of the Stansbury Mountains contains the lowest dissolved solids content in the valley, with concentrations of 100 to 800 mg/L (0.006 to 0.050 lb/ft^3). In the southernmost part of the valley, the dissolved solids content concentrations range from 700 to about 900 mg/L (0.044 to about 0.056 lb/ft^3); however, dissolved solids content concentrations as high as 2,500 mg/L (0.156 lb/ft^3) have been observed in a well south of the Reservation.

The north end of the valley generally has high dissolved solids content concentrations in the range of 1,600–7,900 mg/L (0.100 to 0.493 lb/ft^3). The main ions in the groundwater are sodium and chloride.

The staff has reviewed the discussion and information regarding regional subsurface hydrology characteristics and found it acceptable because regional characteristics have been adequately described for further assessment of external events and the impact of the facility on present and future groundwater use in the region is negligible. 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 72.98(c)(2) and 72.122(b) with respect to this issue.

2.1.5.2 Site Characteristics

The applicant has classified the subsurface material at the proposed site as a relatively compressible top layer, approximately 25 to 30 feet thick, that is underlain by much denser and stiffer material. The underlaying layer is classified as dense sand and silt. The onsite boreholes, when drilled to a depth of 100 feet, did not intercept the water table. The groundwater table is greater than 100 feet below grade at the site. Based on regional studies in Skull Valley, the groundwater flows from the south to the north, toward Great Salt Lake. The hydraulic gradient is estimated at 9.5×10^{-4}. The permeability of silt soil in the Skull Valley ranges from 0.2 to 0.6 in/hr. The average estimated groundwater velocity ranges from 5.08×10^{-4} to 1.136×10^{-3} ft/day. The precipitation at the site does not contribute to the groundwater flow due to low permeability of surficial deposits and high rates of evapotranspiration. The groundwater flow beneath the site is mainly derived from precipitation at the higher elevations of the Stansbury Mountains.

The staff has reviewed the discussion and information regarding the site characteristics and found it acceptable because the groundwater characteristics have been adequately described for further assessment of external events. 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 72.98(c)(2) and 72.122(b) with respect to this issue.

2.1.5.3 Contaminant Transport Analysis

A hydrologic transport analysis was not included in the SAR because release of effluents from the Facility is not expected. The facility and cask designs are expected to preclude release of effluents for normal, off-normal, and accident conditions.

The staff has reviewed the discussion on contaminant transport analysis and has determined it to be acceptable because release of effluents from the Facility is not expected. 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 72.98(c)(2) and 72.122(b) with respect to this issue.

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 responses to Round 1 request for additional information (RAIs) 2-5 and 2-7 (Parkyn, 1999a; Donnell, 1999d) including two reports entitled "Fault Evaluation Study and Seismic Hazard Assessment Study—Final Report" (Geomatrix Consultants, Inc., 1999a) and "High-Resolution Seismic Shear Wave Reflection Profiling for the Identification of Faults at the Private Fuel Storage Facility, Skull Valley, Utah" (Bay Geophysical Associates, Inc., 1999). The review also includes the additional seismic evaluation (Appendix 2G) provided in the SAR (Private Fuel Storage Limited Liability Company, 2000). 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 be 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 the revised "Fault Evaluation Study and Seismic Hazard Assessment Study—Final Report." In 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 staff has reviewed the information presented in Section 2.6, Geology and Seismology, and accompanying Appendixes 2B, 2C, 2D, 2E, and 2G of the SAR (Parkyn, 1999a,b; Private Fuel Storage Limited Liability Company, 2000; Geomatrix Consultants, Inc., 1999a; Bay Geophysical Associates, Inc., 1999) 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), 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 subsequent 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 (Private Fuel Storage Limited Liability Company, 2000), 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, the peak horizontal acceleration and peak vertical acceleration values from the seismic event would be 0.72 and 0.80g, respectively (Geomatrix Consultants, Inc., 1999b). 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 proposes to design the Facility to the ground motions produced by 1,000-year return period earthquakes. These design-ground motions 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).

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 SAR and the result of the staff's independent investigation. 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 and complete. 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, staff finds the probabilistic assessment provided by Geomatrix Consultants, Inc. (1999a) acceptable, and possibly conservative, because the Geomatrix Consultants, Inc. (1999a) model considers the West fault as an active seismic source.

In summary, the staff found that the applicant's considerations of seismic source characteristics and associated uncertainties provide reasonable assurance that all significant source and capable faults have been identified and their characteristics and associated uncertainties are adequately 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.

Estimate of Ground Motion Attenuation

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 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. 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.

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 estimated.

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. 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. The mean peak horizontal acceleration is 0.40 g and 0.53 g for 1,000- and 2,000-year return periods, respectively. The mean peak vertical acceleration is 0.39 g and 0.53 g 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).

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 84th-percentile peak ground accelerations are 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 show that the ground motion from the East fault generally envelops those from the other sources. The 84th-percentile peak ground accelerations for the East fault are 0.72g in the horizontal direction and 0.80g in the vertical direction. When compared with the PSHA results, the controlling deterministic spectra generally are between the 5,000- and 10,000-year 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. (1999c) based on its site-specific PSHA results as reviewed in Stamatakos et al. (1999) and documented in detail in Geomatrix Consultants, Inc. (1999a). 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. (1999c). 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 84th-percentile peak acceleration and the 84th-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. (1999c). 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.

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} .

- On the basis of the foregoing, the mean annual probability of exceedance for the PFS Facility may be less 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 0.53 g. The 0.53 g 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 0.53 g. 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 95th-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.

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, 2000). The staff also reviewed information presented in Appendix 2A, Geotechnical Data Report, of the SAR (Private Fuel Storage Limited Liability Company, 2000) and other data and analyses provided by the applicant (Stone and Webster Engineering Corporation, 1998, 2000a,b,c; 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).

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.

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. Fourth, the possibility of soil collapse from dynamic loading was investigated

through cyclic triaxial testing of five specimens. The response of the five specimens suggests that the soils would not undergo collapse under dynamic loading that is equivalent to the design basis earthquake.

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.

Stability of Cask-Storage-Pad Foundation

The cask storage pads (each 30 feet wide, 64 feet long, and 3 feet thick) will be laid out in four clusters as shown in Figure 2.6-2 of the SAR. Each cluster consists of 130 pads arranged as thirteen 10-pad arrays with a 30-foot separation between arrays (in the width direction of the pads) and 5-foot separation between the pads in each array (in the length direction of the pads). Because of the small (5 feet) end-to-end spacing between pads in each array, the foundation of an array may be treated as a 30-foot wide strip footing. Because of the 30-foot width-wise separation between the arrays, the zones of influence of the foundation loading for adjacent arrays can be assumed to be independent to a depth of 30 feet below the base of the pads. The eolian silt (layer 1A soil) in the storage pad area will be completely excavated and mixed with cement and water and compacted to form a soil-cement subgrade over the surface of the storage-pad area (Section 2.6.1.7 of the SAR). Each storage pad will be founded at a depth of 2 feet below the surface of the soil-cement subgrade (Figure 4.2-7 of SAR; Figure 3 of Stone and Webster Engineering Corporation, 2000b) such that lateral movement of a pad cannot occur without rupture of the soil-cement wall around it. The soil-cement layer will extend to a depth of at least 1 foot, but up to 3–5 feet, below the base of each pad (SAR, p. 2.6-62). The soil-cement mixture will be designed to have an unconfined compressive strength of at least 36 ksf (250 psi). Also, the storage pads will be constructed so that their top surfaces are at least 3.5 in above the finished grade to accommodate the estimated postconstruction settlement of the pads (SAR, Revision 13, p. 26–51).

Each storage pad will be loaded to a static bearing pressure of 1.94 ksf, considering the dead load plus long-term live load for a 30 feet × 64 feet × 3 feet concrete pad loaded with eight casks. Potential dynamic loading of the pads was characterized by vertical and horizontal ground accelerations of 0.53 g and 0.53 g, 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: 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 4.35 ksf based on the undrained analysis, or 9.44 ksf based on the drained analysis. Both values of allowable bearing pressure exceed the actual bearing pressure of 1.94 ksf under static loading (dead load plus long-term live load).

The staff reviewed the applicant's evaluation regarding the estimated allowable bearing pressure under static loading and found it acceptable. 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. 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.

Stability Against Excessive Settlement Under Static 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. 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. Additional calculations made by PFS, of the storage-pad settlement were conducted by PFS using the Q_t data and a procedure developed by Schmertmann (1970, 1978). The additional calculations gave values of settlement smaller than 1.0 inch 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 at least 3.5 inches above the finished grade to accommodate a settlement of this magnitude (p. 2.6-51 of the SAR).

Therefore, the staff concludes that the proposed cask-pad design is acceptable considering the potential for excessive settlement under static loading.

Stability Against Sliding Under Dynamic Loading

The proposed design of the pad foundations relies on the shear resistance of the soil-cement subgrade to prevent sliding of the pads. To achieve this objective: (i) each storage pad will be embedded in the soil-cement subgrade to a depth of 2 feet below the subgrade surface, (ii) the soil-cement layer will extend to a depth of at least 1 foot below the base of the pad, (iii) the soil-cement subgrade will extend over the entire storage-pad area and at least 4 feet from the edge of the exterior storage pads, and (iv) the soil-cement material will have an unconfined compressive strength of at least 36 ksf (250 psi). These specifications were determined by the applicant, except for the requirement that the soil-cement subgrade extend to at least 4 feet from the edge of each pad, which was determined by NRC staff through a consideration of the minimum cross-sectional area of soil-cement required to prevent sliding of a storage pad.

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.

Therefore, the staff concludes that the proposed cask-pad design is acceptable considering the potential for base sliding under dynamic loading.

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 2-2. In each load case, the static load (dead load plus long-term live load for a 30 feet × 64 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 2-2 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.

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.

Table 2-2. 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	7.68	3.63
IIIA	0.4	0.4	−1.0	10.35	1.52
IIIB	0.4	1.0	−0.4	8.10	3.06
IIIC	1.0	0.4	−0.4	9.56	2.37
IVA	0.4	0.4	1.0	10.83	3.42
IVB	0.4	1.0	0.4	9.03	3.53
IVC	1.0	0.4	0.4	9.89	3.09

Stability of the Canister Transfer Building Foundation

The footprint of the proposed Canister Transfer Building foundation is 272 feet long in the north-south direction and 145 feet wide in the east-west direction, except for a 60-foot long section near the south end of the building that is 215 feet wide. A rectangle 165 feet wide by 265 feet long was determined by the applicant to have approximately the same area and moments of inertia as the Canister Transfer Building footprint. Stability analyses of the foundation were performed using the equivalent rectangle. The foundation is a 5-foot thick reinforced concrete slab and will be founded at a depth of 5 feet below the ground surface. The perimeter of the foundation will be extended to a depth of 6 feet (using a 1-foot deep concrete key) to constrain potential sliding surfaces to pass through the underlying soil. The 6-foot deep excavation around the foundation will be backfilled using eolian silt (layer-1A soil), which will be compacted to a unit weight of 80 lb/ft^3 and friction angle of at least 30°.

The foundation loading was determined through a lumped-mass analysis of the Canister Transfer Building, which gave a vertical static load of 72,988 kips and dynamic load of 57,139 kips vertical, 62,040 kips north-south, and 67,572 kips east-west (Stone and Webster Engineering Corporation, 1999b). The dynamic loads were calculated using vertical and horizontal ground accelerations of 0.53 g and 0.53 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 (Section 2.1.6.2). 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 1.67 ksf, considering the vertical static load of 72,988 kips supported by a total bearing area of 43,640 ft^2 (Stone and Webster Engineering Corporation, 2000c). 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 calculated over a depth of 23 feet below the foundation 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 6.31 ksf based on the undrained analysis, or 45.0 ksf based on the drained analysis. Both values of allowable bearing pressure exceed the actual bearing pressure of 1.67 ksf under static loading. The staff reviewed the applicant's evaluation regarding the estimated allowable bearing pressure under static loading and found it acceptable.

Therefore, the staff concludes that the Canister Transfer Building foundation design is acceptable considering the potential for bearing-capacity failure under static loading.

Stability Against Excessive Settlement Under Static 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. 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. The effect of the estimated settlement on the structural design of the Canister Transfer Building is evaluated in Section 5.1.3.4, Structural Analysis for Reinforced Concrete Structures.

Therefore, the staff concludes that the Canister Transfer Building foundation design is acceptable considering the potential for excessive settlement under static loading.

Stability Against Sliding Under Dynamic Loading

The proposed design of the Canister Transfer Building foundations relies on a 1-foot deep perimeter key at the base of the foundation to constrain potential sliding surfaces to pass through the underlying layer-1B soil. Because of this design, the resistance to base sliding can be determined using the undrained shear strength of layer-1B soil. The applicant presented base-sliding analyses for six combinations of static load, 100 percent dynamic load in one coordinate direction, and 40 percent dynamic load in the other two coordinate directions, with vertical dynamic load upward and downward (Table 2.6-13 of SAR). Changing the vertical

dynamic load has no effect, however, because the potential sliding surface is horizontal and the sliding resistance is not evaluated in terms of friction. Therefore, only the horizontal-load combinations need to be varied, and information presented in the SAR indicates that the case with 100 percent east-west and 40 percent north-south dynamic loads is critical. The applicant obtained a factor of safety against sliding of 1.1 for this case, using a c_u value of 1.8 ksf interpreted from direct shear test results. The factor of safety increases to 1.3 when the calculation is performed using c_u=2.2 ksf, which is the acceptable undrained shear strength for layer-1B soil.

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 2.6-83). 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.

Therefore, the staff concludes that the proposed Canister Transfer Building foundation design is acceptable considering the potential for base sliding under dynamic loading.

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 2-3. In each load case, the vertical static load of 72,988 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 (57,139 kips vertical; 62,040 kips north-south; and 67,572 kips 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 calculated for a depth of 23 feet below the foundation 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 average c_u would be larger if the relatively stiff layer-2 soil, which lies 20–25 feet below the Canister Transfer Building foundation, 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 2-3 shows, the calculated bearing pressure for each load case is smaller than the allowable bearing pressure.

Therefore, the staff concludes that the proposed Canister Transfer Building foundation design is acceptable considering the potential for bearing-capacity failure under dynamic loading.

Table 2-3. Results of bearing capacity analysis of Canister Transfer Building foundation under dynamic loading (from Private Fuel Storage Limited Liability Company, 2000)

Load Case	Dynamic Load Factors			Bearing Pressure (ksf)	
	North-South	East-West	Vertical	Allowable	Calculated
II	1.0	1.0	0.0	9.98	3.59
IIIA	0.4	0.4	−1.0	7.95	2.50
IIIB	0.4	1.0	−0.4	9.04	3.31
IIIC	1.0	0.4	−0.4	13.12	2.15
IVA	0.4	0.4	1.0	15.64	3.44
IVB	0.4	1.0	0.4	12.70	3.42
IVC	1.0	0.4	0.4	14.22	2.97

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). Specifically:

- 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.

- Sufficient investigation was conducted to evaluate the potential for instability resulting from possible occurrence of collapsible soils at the proposed site. Results of the investigation indicate that the potential for such instability is negligible.

- The stability of the cask storage pads was evaluated 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.

- The stability of the cask storage pads was evaluated with respect to the potential for excessive settlement under static loading. 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. This conclusion is based in part on the following storage pad design feature specified in the SAR: The storage pads will be constructed such that their top surface is at least 3.5 inches above the finished grade to accommodate the estimated post construction settlement of the pads.

- The stability of the cask storage pads was evaluated with respect to the potential for sliding under dynamic loading. 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. This conclusion is based in part on the following storage pad design features specified in the SAR: (i) the eolian-silt layer (layer 1A soil) in the storage-pad area will be completely replaced with a cement-reinforced subgrade over the surface of the storage-pad area; (ii) each storage pad will be founded 2 feet below the surface of the subgrade and will be embedded in the subgrade, such that lateral movement of the pad cannot occur without rupture of the cement-reinforced wall around it; (iii) the cement-reinforced subgrade will extend at least 4 feet from the edge of the storage pads; and (iv) the cement-reinforced subgrade material will have an unconfined compressive strength of at least 36 ksf (250 psi). The unconfined compressive strength of the subgrade material must be demonstrated by testing and is, therefore, included in the Technical Specifications.

- The stability of the cask storage pads was evaluated 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.

- The stability of the Canister Transfer Building foundation was evaluated 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.

- The stability of the Canister Transfer Building foundation was evaluated with respect to the potential for excessive settlement under static loading. The evaluation was performed using appropriate techniques, foundation loading, and material properties; and it was demonstrated that the estimated settlement can be tolerated without damage to the Canister Transfer Building structure.

- The stability of the Canister Transfer Building foundation was evaluated with respect to the potential for sliding under dynamic loading. The evaluation was performed using appropriate techniques, foundation loading, and material properties. The staff considers the analysis of the stability of the Canister Transfer Building foundation acceptable. This conclusion is based in part on the following design feature specified in the SAR: A 1-foot deep concrete key will be installed around the perimeter of the Canister Transfer Building foundation to constrain potential sliding surfaces to pass through the underlying soil.

- The stability of the Canister Transfer Building foundation was evaluated 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.

- The risk of liquefaction or other soil instability due to vibratory ground motion was sufficiently assessed.

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

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