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2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

The PFSF site is situated in the northwest corner of the Skull Valley Indian Reservation in Tooele County, Utah. The Reservation consists of approximately 18,000 acres, of which the PFSF site area is approximately 820 acres, or less than 5% of the reservation area. The PFSF site location was selected by the Skull Valley Band of Goshute Indians in order to avoid disruption of tribal roads, housing or cultural facilities. Figure 1.1-1 shows the facilities and locations addressed in this section.

The area surrounding the PFSF site is very sparsely populated, with the nearest residence 2 miles southeast of the site. The Skull Valley Band of the Goshute Village, with a population of about 30, is 3.5 miles east-southeast of the PFSF site. Terra, a small residential community with a population of 120 (Tooele County Commission, 1995), is located 10 miles east-southeast of the PFSF.

2.2.1 Hazards from Facilities and Ground Transportation

Tekoi Rocket Engine Test Facility

The only industrial, transportation or military facility within 5 miles of the PFSF is the Tekoi Rocket Engine Test facility, located about 2.5 miles south-southeast of the PFSF. This facility, located on leased reservation lands, was used in the past by Alliant Techsystems Inc. to periodically test rocket engines mounted on stationary bases and explosive charges. Hickman Knolls, with an elevation of approximately 4873 ft, is situated directly between the PFSF (approximate elevation 4465 ft) and the Tekoi Test facility (elevation 4600 ft). The relative location of Hickman Knolls between the PFSF and Tekoi Test facility, and the distance of 2.5 miles would substantially deflect and disperse overpressures from an explosion at the Tekoi Test facility, precluding any hazard to the PFSF. In order for this facility to be used by Alliant Techsystems Inc. in the future, the lease agreement between Alliant and the Goshute Band would need to be renegotiated. There are no other facilities which could present the threat of an explosion or other hazard within 5 miles of the PFSF.

Major Transportation Corridors

Interstate Highway 80 and the Union Pacific Railroad main line are located 24 miles north of the PFSF site. Any events associated with either the interstate highway or the railroad will not present a hazard to the PFSF due to the relatively large distance involved. The Skull Valley Road runs essentially north-south between Interstate 80 and the town of Dugway, population 1,700, 12 miles south of the PFSF. Dugway is a residential community supporting the nearby Dugway Proving Ground and has no facilities which could present a hazard to the PFSF.

Dugway Proving Ground

The U.S. Army's Dugway Proving Ground is a 1,315 square mile range and test facility located west of the town of Dugway. The Dugway Proving Ground performs testing of all types of military equipment in chemical and biological environments, as well as smoke, obscurant and incendiary testing, and munitions testing. Open air testing is not permitted by law, and there have been no accidents or releases of toxic gas from the facility or associated transportation activities. The Proving Ground has a mean elevation of 4,350 ft above sea level and is surrounded on three sides by mountain ranges. The Cedar Mountains, with an elevation of 5,300 ft or greater, lie between the Proving Ground and the PFSF. The following potential threats posed by the Dugway Proving Grounds were assessed: 1) the firing of conventional ground weapons in military testing and training; 2) the testing, storage, and disposal of chemical munitions and agents; 3) the testing of biological materials; 4) the transportation of biological, chemical and hazardous materials to and from Dugway Proving Ground; and 5) unexploded ordnance. The PFSF will be located over 8 miles north from the northeastern boundary of Dugway Proving Ground and will be approximately 20 miles from the locations where most of the activities involving chemical agent and biological materials take place at of Dugway Proving Ground. By virtue of the distance between the PFSF and the locations on of Dugway Proving Ground where the ostensibly hazardous activities take place, the nature of the activities, and the safety precautions that are taken with respect to all potentially dangerous activities at of Dugway Proving Ground, those activities would not pose a significant hazard to the PFSF, as discussed in the following paragraphs.

Military training exercises and the firing and testing of conventional weapons will not pose a hazard to the PFSF because 1) the firing of weapons is covered by rigid procedures, 2) the closest firing position to the PFSF is more than 15 miles away, 3) the ranges of most of the weapons are insufficient to reach the PFSF from those distances, and 4) the weapons are fired toward the south and northwest, away from the PFSF. It is not credible that a conventional munition fired from Dugway would strike the PFSF.

Chemical munitions and agent at Dugway will pose no significant hazard to the PFSF. Open air testing of chemical munitions and agents was prohibited by law in 1969 (50 U.S.C. § 1512), and has not been conducted since 1969. Thus, activities at Dugway Proving Ground involving chemical agent and munitions are limited to indoor testing of chemical agent, storage of agent and unexploded chemical munitions recovered from the firing ranges, and disposal of chemical agent. None of these activities pose a hazard to the PFSF because of their distance from the PFSF and the limited quantities of agent whose release would be credible. The in-door testing of chemical agent is done in facilities – located close to 20 miles from the PFSF – designed to preclude the release of chemical agent, and thus would pose no credible hazard to the PFSF. Similarly, the locations at which chemical munitions and agent are stored on Dugway Proving Ground are located more than 17 miles from the PFSF and are stored under a strict set of rules governing their storage, including State regulations under RCRA for the storage of chemical munitions and related agent. By virtue of the distance to the PFSF and the many controls designed to protect public health and safety, the release of chemical agent from chemical munitions or agent stored at Dugway does not pose a credible hazard to the PFSF. The worst credible threat posed by chemical agent at Dugway would arise from the accidental detonation of a previously unexploded 8-inch projectile filled with chemical agent GB (which is an extremely unlikely event). The distance at which such an event would pose a threat is on the order of several miles, far less than the distance to the PFSF. Likewise, the disposal of chemical munitions and agents is done under rigorous control, including regulation by the State under RCRA, and would pose no credible hazard to the PFSF.

Biological materials present at Dugway Proving Ground would likewise not pose a credible hazard to the PFSF because all use of biological materials at Dugway Proving Ground is conducted in the Life Sciences Test facility – more than 20 miles from the PFSF – under engineering and procedural controls designed to prevent the release of material to the environment. The United States destroyed its biological agents and munitions after a presidential decree in 1969 and Dugway Proving Ground does not test biological agents for use in warfare. All testing done at the Life Sciences Test Facility is for defensive purposes. Further, even if biological material at the test facility were to escape, it would pose no significant hazard to the PFSF, in that it would have almost no chance of surviving in the environment long enough to be carried the 20 miles from the facility to the PFSF. Thus, the use of biological materials at Dugway Proving Ground poses no credible hazard to the PFSF.

The transportation of chemical agent or biological materials to or from Dugway does not pose a significant hazard to the PFSF. Larger shipments of such material are performed with extraordinary safety precautions and, moreover, do not travel along Skull Valley Road. Small, laboratory quantities of material could potentially be shipped by common carrier along Skull Valley Road, but the safe packaging of those shipments is strictly regulated by the Department of Transportation so as to prevent a release even in the event of an accident. Hazardous wastes shipped from Dugway Proving Ground do not include chemical agent but rather only chemically neutralized agent, which is far less hazardous and would not threaten the PFSF even if spilled on Skull Valley Road.

Unexploded ordnance would not pose a significant hazard to the PFSF in that 1) it is extremely unlikely that such ordnance would explode spontaneously or accidentally and 2) even if it did, the PFSF is far enough away that the material in the round would not pose a significant hazard. Unexploded ordnance is not likely to be found off Dugway Proving Ground close enough to pose a risk to the PFSF, in that the firing ranges at Dugway are all at least 15 miles away and Army records of where munitions were fired at Dugway give no indication that munitions were fired elsewhere.

The Dugway Proving Ground receives and ships conventional Army weapons approximately 95 times a year. Some of these shipments could travel the Skull Valley Road, which present the only credible potential for an explosion near the PFSF. An accident associated with the transportation of explosives along the Skull Valley Road would be a minimum of 1.9 miles from the Canister Transfer Building and 2 miles from the nearest cask storage pad. Based on the methodology of Regulatory Guide 1.91, the Skull Valley Road is located much further from the PFSF than the distances required to exceed 1 psi overpressure for detonation of explosives transported by highway, as discussed in Section 8.2.4.

The Tooele Army Depot facilities, where toxic gas munitions are stored and incinerated, are located west and south, respectively, of Tooele City. The North Tooele Army Depot is 17 miles east-northeast of the PFSF and the South Tooele Army Depot is 21 miles east-southeast of the PFSF. The Stansbury Mountains, with an elevation of approximately 8,000 feet, lie between the PFSF and the Tooele Army Depots. The activities and materials at the Tooele Army Depots will therefore present no credible hazard to the PFSF, because of their relative distance and the intervening Stansbury Mountains.

2.2.2 Hazards from Air Crashes

Aircraft flights in the vicinity of the PFSF take place to and from Michael Army Airfield on Dugway Proving Ground, on and around the Utah Test and Training Range (UTTR), and on federal airways J-56 and V-257. While there are no civilian airports within 25 miles of the PFSF, general aviation aircraft, while not reported, may also transit the region. The average annual probability of an aircraft crashing into the PFSF has been calculated to be less than 1 E-6 per year and qualitative factors indicate that the true probability of an aircraft impacting the PFSF is less than 1 E-7 per year. (PFS Feb. 2000) This is an extremely low probability, well below the 1 E-6 regulatory standard the NRC has promulgated for above ground facilities at geologic repositories (which are similar to ISFSIs) (61 Fed. Reg. 64,257, 64,261-62, 64,265-66 (1996)) and below the 1E-7 guideline of NUREG-0800 established for nuclear power plants. Therefore, aircraft crashes do not present a credible hazard to the PFSF and the facility does not need to be designed to withstand the impact of an aircraft crash.

2.2.2.1 Michael Army Airfield and Airway IR-420

Michael Army Air Field is located on the Dugway Proving Ground, 17 miles south-southwest of the PFSF. This military airfield has a 13,125 foot runway, and can accommodate all operative aircraft in the Department of Defense inventory, although the majority of the aircraft flying to and from Michael AAF are large cargo aircraft such as the C-5, C-17, and C-141. The airspace over the Dugway Proving Ground is restricted. Military airway IR-420 passes over the PFSF site area. The methods of NUREG-0800 Section 3.5.1.6 were used to estimate the probability of an aircraft impacting the PFSF from this airway, using the equation:

$P = C \times N \times A / w$, where

P = probability per year of an aircraft crashing into the PFSF
C = in-flight crash rate per mile
N = number of flights per year along the airway
A = effective area of the PFSF in square miles
w = width of airway in miles

NUREG-0800 states the in-flight crash rate as 4 E-10 per mile, which is appropriate to apply to the types of aircraft flying to and from Michael AAF. (PFS Feb. 2000) Information provided by the Dugway Proving Ground states that there are approximately 414 flights annually at this airfield. The effective area of the PFSF is 0.2116 mi^2, calculated using Department of Energy (DOE) formulas. (DOE 1996) The width of the airway is 10 nautical miles (nm), or 10nm x 1.15 mile/nm = 11.5 miles. The probability of an aircraft impacting the PFSF is therefore 3.0 E-9 per year. Because of the distance from the PFSF to Michael Army Airfield, takeoff and landing operations at Michael pose a negligible hazard to the PFSF.

Consideration was given to the plans for landing the X-33 aircraft at Michael Army Airfield. The X-33 is an unmanned half-scale demonstrator launch vehicle planned to test critical components for the next generation space transport system. The X-33 will not pose a hazard to the PFSF because, first, tests for the X-33 at Michael Army Airfield are scheduled to be completed by mid-2000, before the PFSF would be operational, and second, the X-33's flight plan does not take it over Skull Valley, let alone the PFSF.

2.2.2.2 Utah Test and Training Range

The UTTR is an Air Force training and testing range over which the airspace is restricted to military operations. It is divided into a North Area, located on the western shore of the Great Salt Lake, north of Interstate 80, and a South Area, located to the west of the Cedar Mountains, south of Interstate 80 and northwest of Dugway Proving Ground. (PFS Feb. 2000) The airspace over the UTTR extends somewhat beyond the range's land boundaries and is divided into military operating areas (MOAs) and restricted areas. The MOAs on the UTTR are located on the edges of the range, adjacent to the restricted areas. The PFSF site is located over 18 statute miles east of the eastern land boundary of the UTTR South Area and 8.5 statute miles northeast of the northeastern boundary of Dugway Proving Ground. The site lies within the Sevier B MOA, two statute miles to the east of the edge of restricted airspace. (PFS Feb. 2000)

Military aircraft flying in or around the UTTR South Area comprise three groups: 1) F-16 fighter aircraft flying from Hill Air Force Base (AFB), near Ogden, Utah, down Skull Valley en route to the range (Section 2.2.2.2.1); 2) aircraft conducting training in the restricted airspace on the range (Section 2.2.2.2.2); and 3) aircraft departing the range via the Moser Recovery to return to Hill AFB (Section 2.2.2.2.3). Aircraft flying in or around the UTTR North Area pose no credible hazard to the PFSF because of the distance from the facility.

2.2.2.2.1 F-16s Transiting Skull Valley

F-16 fighter aircraft fly north to south down Skull Valley, within Sevier B MOA, en route from Hill AFB to the UTTR South Area. The F-16s use the eastern side of Skull Valley as their predominant route of travel and typically pass approximately five miles to the east of the PFSF site. The U.S. Air Force has indicated that the F-16s typically fly between 3,000 and 4,000 ft. above ground level (AGL), with a minimum altitude of 1,000 ft AGL. In 1998, 3,871 such flights passed through Skull Valley.

Because the predominant route of travel for the F-16s is down the eastern side of Skull Valley, away from the PFSF; because the likely nature of an F-16 crash in Skull Valley would be such that a crashing aircraft would not pose a hazard to the PFSF unless it was pointed directly at the site at the time of the event leading to the crash; and because Air Force pilots are instructed to avoid ground facilities in the event of a mishap in which the pilot retained control of the direction of the aircraft, it is not credible that a crashing F-16 would impact the PFSF. Nevertheless, an impact probability was calculated, using the methodology of NUREG-0800, in which it was conservatively assumed that the F-16 flights are uniformly distributed within the Sevier B MOA airspace in the vicinity of the PFSF. (PFS Feb. 2000)

To calculate the F-16 impact probability using the NUREG-0800 method, the Sevier B MOA airspace in the vicinity of the PFSF was treated as an airway with a width of 10 miles. Given the flight characteristics of the F-16, the PFSF has an effective area of 0.1337 mi^2, assuming a facility at full capacity with 4,000 spent fuel storage casks on site. The number of flights through the valley was taken to be 3,871 per year. The crash rate for the F-16 was calculated from Air Force data to be 2.736 E-8 per mile. It was also determined from Air Force data that over 95 percent of the F-16 crashes in the normal phase of flight (the phase of flight in which the F-16s transit Skull Valley) are attributable to engine failure. Furthermore, because of the training Air Force pilots receive in responding to engine failures, the flight characteristics of the F-16, the absence of other built up areas in Skull Valley, and the small effort required for the pilot to avoid the PFSF site in the event of a crash caused by an engine failure, the pilot would be able to direct the aircraft away from the PFSF at least 95 percent of the time in which an engine failure caused a crash in Skull Valley. Accordingly, 90.25 percent (95% x 95%) of the crashing F-16s would be able to avoid the PFSF and hence the calculated crash impact hazard to the PFSF would be reduced by this fraction. Thus, the annual crash impact probability for the F-16s in Skull Valley (assuming a fully loaded facility) was calculated to be 1.38 E-7. (PFS Feb. 2000)

PFS also calculated the probability that ordnance jettisoned from a crashing F-16 in Skull Valley would impact the PFSF. (PFS Feb. 2000) Some of the F-16 flights through Skull Valley carry ordnance (live or inert) and in the event of an incident leading to a crash in which the pilot would have time to respond before ejecting from the aircraft (e.g., an engine failure), one of the pilot's first actions would be to jettison any ordnance carried by the aircraft. PFS used an approach similar to that of NUREG-0800 to calculate the probability that such ordnance would impact the PFSF. The fraction of the 3,871 F-16s transiting Skull Valley per year that would be carrying ordnance was determined from Air Force data to be 11.8 percent. Thus the number of aircraft carrying ordnance through Skull Valley per year, N, would be 457. The crash rate for the F-16s, C, was taken to be 2.736 E-8 per mile, as above. Nonetheless, the pilot was assumed to jettison ordnance in only 95 percent of all crashes, the fraction of the crashes, e, assumed to be attributable to engine failure (in crashes attributable to other causes it was assumed that the pilot would eject quickly and would not jettison ordnance). Skull Valley was treated as an airway with a width, w, of 10 miles. As with the calculation for F-16s transiting Skull Valley, PFS conservatively assumed that the F-16s are uniformly distributed across the 10 miles, despite the fact that their predominant route of flight is down the eastern side of the valley and that, according to the Air Force, aircraft carrying live ordnance avoid flying over populated areas to the maximum extent possible. The area of the PFSF, from the perspective of a piece of ordnance jettisoned from an aircraft flying from north to south over the site, A, was taken to be the product of the width and the depth of the cask storage area (assuming a full facility with 4,000 casks) plus the product of the width and depth of the canister transfer building, in that the pieces of ordnance are small relative to an aircraft and impact the ground at a steep angle. Thus, the area of the PFSF was calculated to be 0.08763 mi^2. The probability that the ordnance would impact the PFSF is given by P = N x C x e x A/w, or:

P = 457 x 2.736 E-8 x 0.95 x 0.08763 / 10 = 1.04 E-7

2.2.2.2.2 Aircraft Training on the UTTR

According to the Air Force, 8,284 sorties were flown over the UTTR South Area in 1998. (PFS Feb. 2000) Those aircraft conducted a variety of activities, including air-to-air combat training, air-to-ground attack training, air-refueling training, and transportation to and from Michael Army Airfield (which is located beneath UTTR airspace). Hazards posed by aircraft flying to and from Michael Army Airfield are addressed in Section 2.2.2.1 above. Of the remaining aircraft, only fighter aircraft conducting air-to-air training represent a potential hazard to the PFSF, in that aircraft conducting air-to-ground attack training do so over targets that are located more than 20 miles from the PFSF site and aircraft conducting air refueling training do so on the far western side of the UTTR, over 50 miles from the site. The Air Force indicated 6,360 fighter sorties were flown on the UTTR South Area in 1998 and one-third, or approximately 2,120, involved fighter aircraft conducting air-to-air training.

The crash impact probability for fighter aircraft conducting air-to-air training on the UTTR was calculated as follows:

$P = C_a \times A_c \times A/A_p \times R$, where

P = annual crash impact probability

C_a = total air-to-air training crash rate per square mile on the UTTR

A_c = the area of the UTTR from which aircraft could credibly impact the PFSF in the event of a crash

A = effective area of the PFSF in square miles

A_p = the footprint area, in which a disabled aircraft could possibly hit the ground in the event of a crash

R = the probability that the pilot of a crashing aircraft would be able to take action to avoid hitting the PFSF

The total air-to-air training crash rate per square mile on the UTTR, C_a, was calculated from the total number of hours flown in air-to-air training on the UTTR South Area (2,468), the crash rate per hour for fighter aircraft (the F-16) in combat training (3.96 E-5), the distribution of air operations over the sectors of the UTTR nearest the PFSF, and the ground areas of those sectors. (PFS Feb. 2000) As with the F-16s transiting Skull Valley, 95 percent of the crashes on the UTTR attributable to engine failure were determined not to pose a hazard to the PFSF, in that the pilot would retain control of the aircraft and would be able to avoid the site. Based on Air Force data, 44 percent of all F-16 crashes occurring during combat training are attributable to engine failure; thus the factor R in the equation above was set equal to 0.582 (1-(44% x 95%)). The area from which an aircraft could credibly impact the PFSF in the event of a crash, A_c, was taken to be the portion of the UTTR within 10 miles of the PFSF, in that a crashing aircraft more than 10 miles from the site would have to be under control of the pilot in order to glide and reach the site, and the pilot would guide any such aircraft away from the site, which is outside the land boundaries and the restricted airspace of the UTTR. The site effective area, A, was determined as in Section 2.2.2.2.1 above for a facility at a full capacity of 4,000 storage casks. The footprint area, A_p, was calculated by assuming that a crashing aircraft could glide in any direction up to a distance equal to the product of its starting altitude above ground and its glide ratio. Accordingly, the aircraft conducting air-to-air training over the UTTR were divided into altitude bands and an impact probability calculated for each band. Aircraft too low to glide to the PFSF in the event of a mishap were calculated not to contribute to the crash impact hazard, in that they would have no chance of reaching the site. The maximum annual air crash impact probability for aircraft conducting air-to-air training on the UTTR South Area was calculated from the sum of impact probabilities of the altitude bands to be 2.02 E-7.

2.2.2.2.3 Aircraft Using the Moser Recovery

Most of the F-16s returning to Hill AFB from the UTTR South Area exit the northern edge of the range (away from the PFSF) in coordination with air traffic control. However, some aircraft returning to Hill from the UTTR South Area may use the Moser recovery route, which runs from the southwest to the northeast, approximately two miles from the PFSF site. (PFS Feb. 2000) The Moser route is only used during marginal weather conditions or at night under specific wind conditions which require the use of Runway 32 at Hill AFB. Based on information from local air traffic controllers, conservatively estimated, the Moser recovery is used by less than five percent of the aircraft returning to Hill. According to the Air Force, 5,726 F-16 sorties were flown on the UTTR South Area, almost all of which flew from Hill AFB (not all aircraft transit Skull Valley en route to the South Area); thus fewer than 286 aircraft per year (5% x 5,726) would use the Moser recovery on their return flights.

The average annual crash impact probability for aircraft flying the Moser recovery was calculated using the NUREG-0800 method. The Moser recovery is defined as an airway with a width, w, of 10 nautical miles (11.5 statute miles) (equal to the width of military airway IR-420). The number of aircraft, N, is conservatively taken to be 286; the crash probability, C, is equal to 2.736 E-8 per mile; the effective area of the site is 0.1337 mi^2; and it is calculated that 90.25 percent of all crashes would be those attributable to engine failure in which the pilot could direct the aircraft away from the PFSF (see Section 2.2.2.2.1). Thus, the annual crash impact probability is conservatively estimated to be 8.87 E-9.

2.2.2.3 Aircraft Flying Federal Airways

Federal airway J-56 runs east-northeast to west-southwest at a distance (from the airway centerline) of 11.5 miles north of the PFSF. (PFS Feb. 2000) Local air traffic controllers have indicated that fewer than 12 aircraft per day use the airway. The crash impact probability for aircraft on the airway was calculated for the PFSF using the method of NUREG-0800. Using the standard width for federal airways, J-56 is 8 nautical miles (9.2 statute miles) wide and the closest edge of J-56 is 6.9 miles from the PFSF. For facilities outside an airway, the effective width of the airway, w, is equal to the actual width plus twice the distance from the facility to the closest edge. Thus, J-56 has an effective width of 23 miles. The number of aircraft, N, is conservatively taken to be 12 per day, the crash rate, C, from NUREG-0800 is 4 E-10 per mile, and the effective area of the PFSF for commercial airliners (the most common aircraft on the airway) is 0.2615 mi^2, assuming a full facility with 4,000 casks. Accordingly, the maximum annual crash impact probability is 1.9 E-8. (PFS Feb. 2000)

Federal airway V-257 runs north and south at a distance (from the airway centerline) of 19.5 miles east of the PFSF. (PFS Feb. 2000) Local air traffic controllers have indicated that fewer than 12 aircraft per day use the airway. The crash impact probability for aircraft on the airway was calculated for the PFSF using the method of NUREG-0800. V-257 is 12 nautical miles (13.2 statute miles) wide and its closest edge is 12.6 miles from the PFSF. Thus, V-257 has an effective width of 39 miles. The number of aircraft, N, is conservatively taken to be 12 per day, the crash rate, C, is 4 E-10 per mile, and the effective area of the PFSF is 0.2615 mi^2. Accordingly, the annual crash impact probability is 1.2 E-8. (PFS Feb. 2000)

2.2.2.4 General Aviation

There are no civilian airports within 25 miles of the PFSF, the PFSF is located in a sparsely populated area, and the PFSF is located inside a military operating area (MOA) in which flight by civilian aircraft is restricted while the MOA is being used by the Air Force (and which is avoided by general aviation pilots because of the difficulty of getting clearance through it). Thus, the general aviation traffic over Skull Valley is negligible; in fact F-16 pilots who have flown from Hill AFB through Skull Valley indicate never having seen general aviation traffic there. Therefore, it is highly unlikely that a general aviation aircraft would crash into the PFSF. (PFS Feb. 2000) Nevertheless, a conservative upper bound on the crash impact probability for general aviation aircraft was calculated using National Transportation Safety Board (NTSB) crash data and the population of general aviation aircraft in the state of Utah. (PFS Feb. 2000) The crash impact probability is equal to C_a x A, where C_a is the crash rate per square mile and A is the effective area of the PFSF. In 1995, the 182,600 general aviation aircraft in the United States suffered 412 fatal accidents. There are 1,218 general aviation aircraft in the state of Utah, which covers an area of 84,094 mi^2. FAA crash data indicate, however, that only 15 percent of all general aviation crashes occur during the cruise mode of flight, which, because there are no airports nearby, is the mode in which general aviation aircraft would be flying near the PFSF. Furthermore, business jets experience 7.85 percent of all general aviation fatal crashes and they can be excluded from this calculation, in that they fly mostly on federal airways. The effective area of the PFSF with respect to general aviation aircraft crashes is 0.1173 mi^2 (assuming a fully loaded facility with 4,000 casks). Accordingly, the average annual crash impact probability for general aviation aircraft is 5.25 E-7. (PFS Feb. 2000)

The crash impact hazard to the PFSF, however, would be reduced below the calculated impact probability, in that the spent fuel storage casks would be able to withstand the crash impact of most general aviation aircraft. Fifty-five percent of all general aviation aircraft are single-engine piston types weighing less than 3,500 lbs. (PFS Feb. 2000) Such aircraft typically fly at speeds under 100 knots (114 mph). Therefore, the impact of such aircraft at the PFSF would be bounded by the design basis tornado missile impact for the PFSF, an automobile weighing 1800 kg (3,968 lbs.) moving at a speed of 126 mph. (p. 8.2-17) Thus, the impact of such light general aviation aircraft would not cause a radioactive release from a storage cask. Therefore, the calculated general aviation crash impact hazard to the PFSF can be reduced by 55 percent to 2.36 E-7.

2.2.2.5 Cumulative Air Crash Impact Probability

The cumulative maximum air crash impact probability is given in the table below.

Aircraft Crash Impact Probabilities	
Aircraft	**Maximum Annual Probability**
Skull Valley F-16s	1.38 E-7
Aircraft Using the Moser Recovery	8.9 E-9
UTTR Aircraft	2.02 E-7
Aircraft on Airway J-56	1.9 E-8
Aircraft on Airway V-257	1.2 E-8
General Aviation Aircraft	2.36 E-7
Aircraft on Airway IR-420	3.0 E-9
Cumulative Crash Probability	6.17 E-7
Jettisoned Military Ordnance	1.04 E-7
Cumulative Hazard	7.23 E-7

The table shows that the cumulative air crash impact probability is less than 1 E-6 for the PFSF. Qualitative factors discussed below show further that the true impact probability for both facilities is less than 1 E-7. Thus, air crash impact does not pose a credible hazard to the PFSF and the PFSF does not need to be designed to withstand the effects of air crash impacts.

2.2.2.6 Projected Growth in Air Traffic

The Federal Aviation Administration projects that the number of commercial aviation flights in the United States will increase by approximately 66 percent between 1998 and 2025, that the number of general aviation flights will increase by approximately 14 percent over the same period, and that the number of military flights will not increase during this period. (FAA 1999) Because most of the air traffic near the PFSF site is military, the growth in commercial and general aviation projected by the FAA will have no material effect on the air crash impact probability calculated for the facility.

2.2.2.7 Conservatism in the PFSF Air Crash Impact Probabilities

While the calculated cumulative hazard for the PFSF is 7.23 E-7, qualitative factors indicate that the true probability of an aircraft or jettisoned ordnance impacting the site is significantly lower, less than 1 E-7 per year. With respect to the F-16s transiting down Skull Valley en route to the UTTR South Area (and jettisoned military ordnance), these factors include the fact that, according to the U.S. Air force, the predominant route of choice for the F-16s is the east side of the Valley, approximately five miles from the site. Thus, the uniform distribution assumed in calculations in Section 2.2.2.2.1 is highly conservative, especially considering the fact that the only aircraft that pose a real hazard to the site are those that are pointed directly toward it at the time of the incident leading to a crash. In addition. the Skull Valley F-16 calculations assume that F-16s will crash at the 10-year average rate rather than the more recent and lower 5-year average rate.

The calculations of the crash impact hazard posed by other aircraft are conservative as well. The calculations assume that the density of flight operations involving air-to-air training near the edge of the UTTR (near the PFSF) is the same as it is near the center of the range, when in fact it is much lower. They also assume that an aircraft could glide up to 10 miles to impact the PFSF in a crash in which the pilot could not retain control over the aircraft, when in fact such aircraft would most likely fall to the ground without flying a significant distance. The calculation of the general aviation aircraft crash hazard assumes that the density of aircraft over Skull Valley is equal to the average density over the State of Utah, when in fact it is much lower. Therefore, the true crash hazard from those aircraft is significantly lower than the calculated value and in fact is insignificant.

Furthermore, the cumulative hazard calculated above is also conservative in two other major respects. First, the calculated probability is for a fully loaded, 4,000 cask facility, which would be the case for only a short period in the life of the PFSF. The average area of the PFSF site, and hence the average annual probability that an aircraft or jettisoned ordnance would impact the site, is 55 percent of that of the full facility. Thus, the average annual impact probability is roughly 4 E-7. Second, no credit was taken for the resistance to the effects of an air crash impact provided by the concrete storage casks in which the spent fuel canisters will be located (other than resistance to impacts of light general aviation aircraft). The cask construction is robust enough that a significant fraction of the potential air crash impacts at the PFSF would not cause a release of radioactivity. (Davis et al. 1998) The casks could withstand the direct impact of a jet fighter or commercial airliner at a speed of over 370 knots, which is significantly greater than typical air crash impact velocities, and the casks could withstand the impact of the great majority of general aviation aircraft altogether. (PFS Feb. 2000) This resistance of the casks to penetration further reduces significantly the calculated risk to the PFSF from aircraft crashes or jettisoned ordnance.

2.2.3 The Use of Ordnance on the UTTR

As discussed in Section 2.2.2.2, military aircraft conduct air-to-ground attack training using air-delivered ordnance on the UTTR South Area. Military aircraft also conduct weapons testing, including the testing of cruise missiles. (PFS Feb. 2000) As shown in the following paragraphs, the use of air-delivered ordnance on the UTTR does not pose a significant hazard to the PFSF (the hazard posed by jettisoned ordnance in Skull Valley was calculated in Section 2.2.2.2.1 above). The PFSF site is located 18 miles to the east of the easternmost land boundary of the range. Based on the following paragraphs it is concluded that weapons use on the UTTR does not pose a credible hazard to the PFSF and the facility does not need to be designed to withstand a weapon impact.

Weapons use on the UTTR is strictly controlled and the UTTR has never experienced an unanticipated munitions release outside of designated launch/release areas. Aircraft flying over Skull Valley are not permitted to have their armament switches in a release capable mode, and all switches are "safe" until the aircraft are inside DOD land boundaries. Master Arm switches are not actually armed until the aircraft are on the ranges within the UTTR where the bombs are to be dropped. Furthermore, the targets on the UTTR are all over 20 miles from the PFSF site and there are no run-in headings for weapons delivery over the Skull Valley area.

Hung Ordnance

The probability of "hung ordnance" (i.e., the failure of ordnance to release from an aircraft when delivery is attempted) and an unintentional release of the ordnance in Skull Valley are exceedingly low. First, most aircraft do not even carry live ordnance but instead carry training ordnance such as Bomb Dummy Units (BDU) or inert filled or empty MK82 500 lb bombs. According to the U.S. Air Force, only approximately 15% of the 8,711 UTTR sorties flown in Fiscal Year 1998 actually carried live ordnance. Training bombs, by contrast, pose no explosive hazard to the PFSF and the dead weight of the BDUs pose no risk to the facility as well. BDU-33's have ballistic characteristics similar to MK 82 bombs but carry only a small smoke charge for marking purposes. They weigh only 25 pounds and are often the weapon of choice for training missions. Second the probability that any ordnance will "hang" is very low. Michael AAF is the designated primary airfield for aircraft landing with live hung ordnance that has failed to release. There were only five hung ordnance aircraft diversions/recoveries into Michael AAF during 1998. Since only approximately 15% of the aircraft sorties carry live ordnance, a total of only five hung ordnance recoveries in 1998 for a total of about 2,000 sorties (approximately 15% of the 13,367 over the UTTR) produces a probability for failing to release of approximately one in 400. Moreover, a failure to release does not mean there will be an inadvertent release or an inadvertent release and explosion. As indicated above, the Air Force has never had an unintentional release of ordnance outside the launch/drop/shoot boxes on the UTTR. All of these are obviously within the UTTR and in fact are over 20 statute miles from the PFSF site.

Finally, the probability of "hung ordnance" striking the PFSF is not credible because aircraft carrying hung ordnance do not fly over Skull Valley. In the event of hung ordnance, the first priority is to maintain aircraft control and then assess the situation and take appropriate action. Pilots contact Clover Control Air Traffic Control Facility and advise them of the situation. When hung ordnance is encountered, the pilot has the option of either jettisoning the rack and munitions on the range, if able, or recovering to base. Michael AAF is the designated primary recovery base for hung ordnance, although Hill AFB is available as well. Pilots request clearance to Michael AAF for hung ordnance recovery/landing. Pilots maintain a stable flight path and remain in Visual Meteorological Conditions by avoiding clouds. Clover Control provides assistance as required and ensures Michael AAF is prepared to receive the aircraft to include fire fighting equipment and medical personnel standing by. The pilot maneuvers the aircraft to the northwest, approximately 20 statute miles from the proposed PFSF site, and proceeds to Michael AAF, avoiding rapid or steep turns and abrupt climbs or descents. Test facilities or any populated areas are avoided. A long straight-in approach with a shallow rate of descent is established to a full stop landing on runway 12 (to the southeast). Runway 12 is 13,125 ft long and 200 ft wide with a barrier cable at the end. After landing, Dugway Proving Ground Explosive Ordnance Disposal personnel inspect and safe the ordnance.

The UTTR record of no unintended release of live ordnance outside of designated launch/release areas and the procedure for landing aircraft with hung ordnance, which avoids populated areas and approaches Michael Army Airfield from the northwest, away from the PFSF, assures that hung ordnance will not impact the PFSF. Consequently, hung ordnance striking the PFSF is not a credible event.

Cruise Missiles

Missile launches are generally confined to the northern and western portions of the UTTR and are at least 30 statute miles away from the PFSF site. Run-ins, drops, and launches are normally done from north to south or east to west and are thus directed away from the PFSF site. Cruise missile targets on the UTTR are located at least 18 miles from the PFSF. Cruise missiles and other weapon systems that have a capability of exceeding range boundaries are required to have a Flight Termination System (FTS) installed prior to testing on the UTTR. The FTSs are designed to promptly destruct the weapons and terminate the weapons' flight paths in the event of an anomaly. Before a bomber launches a test cruise missile, the Mission Control Center verifies that the missile's remote control systems are working properly. At all times throughout the flight the cruise missile FTS must detect a signal that in effect permits the missile to keep flying (FTS discussed in USAF Accident Investigation Board Report, 12/10/97). If the missile does not detect the signal for a preset time, the FTS activates. Safety Officers can also activate the FTS, if required, at any time. The Range Safety Officer at Mission Control and the Airborne Range Instrumentation Aircraft are also both capable of terminating missile flight almost immediately. The UTTR has never experienced an FTS failure. Consequently, a cruise missile striking the PFSF is not a credible event.

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2.6.1.7 Excavations and Backfill

The proposed detention basin, which will be excavated approximately 5 ft deep over an area that is approximately 200 ft x 800 ft at the north side of the proposed storage facility, is the only major excavation proposed for the PFSF. Shallow excavations will be required to construct the cask storage pads and the strip and spread footings supporting the other structures at least 30 inches below finished grade (to provide protection against frost heave), as well as to provide drainage ditches along the proposed access road.

Excavations for footings deeper than 3 ft shall be completed to the design grades, maintaining stable slopes of not steeper than 2 horizontal to 1 vertical. After construction of the foundations, the excavations will be backfilled with structural fill or soil cement to minimize potential problems in the future.

It was originally intended that the cask storage pads would be founded on the silty clay/clayey silt layer shown underlying the near-surface layer of eolian silt in Figures 2.6-5, Sheets 1 through 14. However, instead of excavating the eolian silt from the pad emplacement area and replacing it with suitable structural fill, it will be mixed with sufficient portland cement and water and compacted to form a strong soil-cement subgrade to support the cask storage pads. Refer to Section 2.6.4.11, Techniques to Improve Subsurface Conditions, for addition details.

The in situ materials generally are not adequate for use as structural backfill; therefore, it is expected that structural fill materials will be obtained from an offsite source. Structural fill material shall be granular material consisting of well graded sand and gravel, containing no more than 10% of material passing the #200 sieve and a maximum particle size not greater than 6 inches. Samples of the structural fill material shall be tested for gradation in accordance with ASTM D-422 and for moisture-density relationship in accordance with ASTM D-1557. New gradation and moisture-density tests shall be required whenever a change in material is observed.

Structural fill material shall be placed in thin lifts, not exceeding 8-inch loose thickness, spread evenly, and compacted to 95% of the maximum dry density as determined in accordance with ASTM D-1557. Compacted surfaces shall be protected from freezing and, if found frozen, shall be excavated, wasted, and replaced with new compacted fill. Compacted surfaces shall be pitched to freely drain to eliminate puddling of storm water. Compacted material shall be tested frequently by performing in-place density and moisture tests, as specified in the construction specifications.

2.6.1.8 Engineering-Geology Features Affecting ISFSI Structures

Engineering Geology is discussed in Section 2.6.4.

2.6.1.9 Site Groundwater Conditions

The groundwater table at the site was encountered in Observation Well CTB-5(OW) at a depth of 125 ft (elevation 4,350 ft) in the vicinity of the Canister Transfer Building. Seismic refraction velocities along Seismic Lines 1, 2, & 3 (Appendix 2B) are indicative of saturated conditions at depths ranging from 90 ft to 136 ft below ground surface across the site area (elevation 4,334 ft to 4385 ft), which corroborates the depth to the water table measured in Boring CTB-5(OW). Local groundwater conditions, based on limited water well data in the area, are somewhat variable and dependent upon the subsurface extent of alluvial fan materials. Stock-watering wells four and five miles westerly from the site have water depths of 280 and 295 ft, (elevations 4,350 ft and 4,325 ft, respectively). About 2.5 miles northeast of the site the water table is at 188 ft depth (elevation 4,350 ft), and 6 miles southeast several wells flow at the surface (elevation 4,605 ft). A well at the Tekoi Rocket Engine Test Facility about 3 miles south of the site was drilled to 400 ft and has static water at 80 ft below ground surface (elevation about 4,480 ft). All the above-mentioned wells were completed in unconsolidated materials without drilling into the bedrock. The locations of all wells within 5 miles of the PFSF are identified in Figure 2.5-1. These data suggest that the main aquifer in the central part of Skull Valley is confined or semi-confined and occurs mainly within the fine-grained Tertiary Salt Lake Group deposits. These sediments interfinger with coarse-grained alluvial fan material along the toe of the fan and may create confined conditions where they overlap the fan deposits. The fan deposits are the main recharge zone for the valley aquifers and the main source for domestic water wells in the valley. The aquifer in the fans is unconfined for the most part, but becomes confined and under artesian conditions downslope where the lake and basinal deposits onlap the fan at depth. Water wells drilled near the lower edge of the fan, such as at the Rocket Engine Test Facility, may penetrate several hundred feet of sediments before encountering a coarse alluvial fan layer. Since the coarse layer is under artesian pressure, the level of water in the well will rise upward to the static condition or may flow at the surface, such as occurs just south of the Reservation.

Groundwater levels at the site appear to closely correlate with levels in the main valley aquifer. They do not appear to be affected by proximity to the alluvial fan. At this time it is believed an adequate quantity of suitable quality water can be developed within the site area for the PFSF needs. Specific properties of aquifer materials are unknown at this time. Based on preliminary testing of the site monitoring well, it is believed that groundwater withdrawals at the PFSF site would have no measurable impact on off-site wells, either up-gradient or down-gradient (SWEC, 1999b). Surface soil at the site has a permeability of 0.2 to 0.6 inch/hr, whereas the soil on the alluvial fan has a permeability of 6 to 20 inches/hr (USDA, unpub. data). It is estimated that 3850 gallons per day (2.7 gpm) would meet facility average daily requirements.

Groundwater quality in the area is variable, with the best quality associated with wells developed in the alluvial fans near the Stansbury Mountains. In general, water quality is lower in the valley bottom, but it is suitable for irrigation or stock watering without treatment. The main dissolved ions are sodium and chloride (Hood and Waddell, 1968). There is also a tendency for the quality to be lower farther north, down-valley, towards the Great Salt Lake, although there are exceptions to this trend. Total dissolved solids range from 1,600 to 7,900 mg/l at the northern end of the valley (Arabasz et al., 1987, App. F). Most sources of water in the valley are high in calcium and would be classified as very hard. Aquifer transmissivities range from 500 to 30,000 sq ft/day with an average for Skull Valley estimated at 5,000 sq ft/day (Arabasz et al., 1987, App. F).

2.6.1.10 Geophysical Surveys

Results of seismic refraction and reflection surveys performed at the site in 1996 are found in Appendix 2B. Engineering properties of site materials based on the geophysical investigations are discussed in Section 2.6.1.11. The results of 1998 geophysical surveys (seismic reflection, gravity, and magnetic) are discussed in Geomatrix Consultants, Inc. (1999a) and Bay Geophysical Associates (1999). Seismic cone penetration tests were performed at the locations designated as "SEIS CPT" on Figure 2.6-19. The purpose of these tests was to measure down-hole P and S-wave velocities. The results of these tests are presented in Appendix C of ConeTec, 1999), and the average velocities vs depth are shown in Figure 2.6-28.

Shear wave velocities of soils are dependent on the effective stress, void ratio, and for clays, the plasticity index and overconsolidation ratio of the soils. If all of these parameters were the same, it would be expected that the shear wave velocities would increase with increasing depth in the profile. The apparent leveling off of the shear wave velocities at a depth of about 10 to 15 ft in the results of the seismic CPTs that were performed at the site (Appendix C of ConeTec, 1999) is an indication that one or more of these parameters have changed. A review of the Q_t plots, which are included on the left-hand side of the same pages that present the shear wave velocities vs depth, indicates that the tip resistance increases greatly in this zone. This increase in tip resistance is most likely associated with a change in soil type, as indicated by the SBT plots on the right-hand side of these same pages, as well as by a decrease in the void ratio of these soils. Therefore, it is not unexpected that the shear wave velocities would change within this zone.

A review of the shear wave velocities vs depth presented in Appendix C of ConeTec (1999) indicates that they do not level off with depth. The general trend in the data is to increase with respect to depth; however this trend is masked by the presence of the marked increase in the shear wave velocities in the "harder" zone that exists generally within the depth range of about 13 feet to about 20 feet. If the shear wave velocities associated with this harder zone are excluded, all of the plots of shear wave velocities show a general increase with respect to depth. This general increase in velocity with increasing depth is more readily observed in Figure 2.6-28.

2.6.1.11 Static and Dynamic Soil and Rock Properties at the Site

Geotechnical laboratory tests were performed on samples obtained from the borings. The results of these tests are included in Appendix 2A and are summarized below. Figure 2.6-20 presents plots of the SPT blow counts vs depth in the pad emplacement area, on a row-by-row basis. This figure also presents the index properties that were measured for these soils, along with the results of triaxial testing. Comparison of these plots indicates that the soil properties are fairly consistent across the site.

Pad Emplacement Area

The results of the tests of the silty clay/clayey silts obtained from the upper 25 to 30 ft layer in the pad emplacement area are as follows:

Index Property:	Minimum	Maximum	Average
Water Content, %	8	58	32
Liquid Limit	25	77	44
Plastic Limit	20	46	30
Plasticity Index	0.5	38	14
Moist Unit Weight, pcf	64	91	78
Dry Unit Weight, pcf	40	71	56
Void Ratio	1.4	3.2	2.1
Saturation, %	28	64	53
Specific Gravity = 2.72			

Consolidation parameters:	Low	High	Average
Maximum past pressure, ksf:	5.6	7.2	6.2
Virgin compression ratio, CR:	0.25	0.34	0.29
Recompression ratio, RR:	0.008	0.017	0.012

Rate of secondary compression is shown by the dashed curve in Figure 2.6-6.

Total-stress strength parameters for analyses of sliding stability due to loads from the design basis ground motion are $\phi = 24.9°$ and c = 1.22 ksf based on direct shear tests that are included in Attachment 7 of Appendix 2A. Total-stress parameters for analyses of stability against a bearing capacity failure due to loads from the design basis ground motion are $\phi = 21.3°$ and c = 1.4 ksf based on consolidated-undrained triaxial tests that are included in Attachments 5 and 8 of Appendix 2A. These strengths were measured in the laboratory by consolidating the samples to the confining pressure that will exist prior to the seismic loading and then shearing them rapidly to simulate conditions that will exist during the earthquake loading.

The original triaxial tests (results reported in Appendix 2A, Attachments 2, 4, and 5) were performed at confining stresses that represent the static conditions that will exist under the fully loaded pads. To demonstrate the cohesive nature of these soils, an additional consolidated-undrained triaxial compression test was performed at a confining stress of 1 ksf, which is representative of the minimum confining stresses expected to exist under the fully loaded pads when the maximum uplift forces due to the design basis ground motion occurs, and one test was performed at a confining stress of 0, which is essentially an unconfined compression test. The results of these tests are included in the total-stress strength parameters reported above, and details of these tests are included in Attachment 8 of Appendix 2A.

The dotted line shown in the plot of Mohr's circles of the results of CU tests performed on samples from Boring B-1 that is included in Attachment 8 of Appendix 2A is tangent to the Mohr's circle for Sample U-2B of Boring B-1. It indicates that the cohesion of this specimen is slightly less than that of the other specimens tested. This strength was lower because its natural water content (w_n=52.9%) was higher than that of the other specimens. As indicated by the plots of water content vs depth presented in Figure 2.6-20, most of the *in situ* soils in the upper ~25-ft layer at the site have w_n < 50%, which is more like Samples U-2C and U-2D; hence the recommendation that c = 1.4 ksf for these soils.

For the partially saturated cohesive soils at the site, the strength of the soil is dependent on its apparent cohesion, friction angle, as well as the consolidation pressure. The strain rates are very high during the seismic event; therefore, the partially saturated cohesive soils will be stressed essentially under undrained conditions. The effect of the pore pressure response, if any, during such tests, either positive or negative, will be manifested directly in the shear strength value measured. Because the strain rate of the laboratory tests is at least one order of magnitude slower than the rate associated with the design basis ground motion, the strength measured in the laboratory is a lower-bound estimate of the strength that will be available to resist the dynamic loadings during the seismic event. See the section titled "Dynamic Strength of Cohesive Soils," presented below, for additional details.

Canister Transfer Building Area

The results of the tests of the silty clay/clayey silts obtained from the upper 25 to 30 ft layer in the Canister Transfer Building area are as follows:

Index Property:	**Minimum**	**Maximum**	**Average**
Water Content, %	7	86	40
Liquid Limit	28	83	51
Plastic Limit	18	48	30
Plasticity Index	4	38	20
Moist Unit Weight, pcf	73	118	92
Dry Unit Weight, pcf	40	98	65
Void Ratio	0.7	3.3	1.8
Saturation, %	40	88	71
Specific Gravity	2.71	2.73	2.72

Consolidation parameters:	**Low**	**High**	**Average**
Maximum past pressure, ksf:	6	26	13
Virgin compression ratio, CR:	0.13	0.37	0.31
Recompression ratio, RR:	0.014	0.020	0.018

Total-stress strength parameters for analyses of sliding stability due to loads from the design basis ground motion are $\phi = 21.1°$ and c = 1.13 ksf. These are based on the average values of the direct shear test results for samples from Borings CTB-6 & CTB-S, presented in Attachments 7 and 8 of Appendix 2A. Total-stress parameters for analyses of stability against a bearing capacity failure due to loads from the design basis ground motion are assumed to be the same ; i.e., $\phi = 21.1°$ and c = 1.13 ksf.

The results of performing consolidated-undrained triaxial tests on samples obtained from beneath the Canister Transfer Building are presented in Table 1. These CU tests were performed at confining stresses of 1.7 ksf, which is equivalent to the final effective stresses expected under the Canister Transfer Building. Comparison of these results with those of similar testing performed at confining stresses of 1.3 ksf and 2.1 ksf on samples obtained from the pad emplacement area indicate that these are conservative values. In addition, comparison of the index properties of samples obtained from both of these areas, presented in the tables above, indicate that these soils are similar, although those in the Canister Transfer Building area have slightly higher water contents, liquid limits, plasticity indices, and unit weights. Because the water contents of the clayey soils obtained from beneath the Canister Transfer building are slightly higher (average w_n = 40% vs 32% in the pad emplacement area), it is reasonable to expect the strength of these soils to be slightly lower than those in the pad emplacement area.

The shear strengths (s_u) measured in the CU triaxial tests of samples obtained from beneath the Canister Transfer Building are reported in Table 1 of Attachment 6 of Appendix 2A and ranged from 1.66 to 3.15 ksf. The average value of these shear strengths was 2.64 ksf and the mean value was 2.73 ksf, as shown at the bottom of the last page of Table 3 in Calculation 05996.02-G(B)-5, (SWEC, 2000a). These average and mean values are nearly equal to the results of averaging the s_u values measured on samples obtained in the pad emplacement area.

Table 2 of Calculation 05996.02-G(B)-5, (SWEC, 2000a) presents a summary of the results of testing that was performed on samples obtained from the pad emplacement area, including the average of the s_u values from the CU triaxial tests. These tests were performed at confining stresses of 1.3 ksf and 2.1 ksf. Averaging these results, to permit comparison with the results of the CU tests that were performed at confining stresses of 1.7 ksf, indicates the average and mean values of s_u corresponding to a confining stress of 1.7 ksf, as used for testing the Canister Transfer Building samples, equal 2.8 ksf for the samples obtained in the pad emplacement area. These values compare favorably with the average and mean values of 2.64 ksf and 2.73 ksf, respectively, reported above for samples obtained from beneath the Canister Transfer Building.

As indicated above, the total-stress strength parameters for analyses of stability against a bearing capacity failure of the cask storage pads due to loads from the design basis ground motion are $\phi = 21.3°$ and c = 1.4 ksf. Since the soils in these two areas are similar, based on their index properties and the results of CU tests, and since the recommended values of $\phi = 21.1°$ and c = 1.13 ksf ° for use in analyses of stability against a bearing capacity failure of the Canister Transfer Building due to loads from the design basis ground motion are slightly lower than these values, they are conservative.

For the sand or sandy soils layer in the Canister Transfer Building area found in some of the borings located at a depth of 8 to 20 ft:

Index Property	Minimum	Maximum	Average
Water Content, %	3	15	6
Moist Unit Weight, pcf	85	105	98
Dry Unit Weight, pcf	77	102	93
Void Ratio	0.64	1.2	0.83
Saturation, %	11	32	19
% Fines	9	38	23
Specific Gravity = 2.69			

Pad Emplacement & Canister Transfer Building Areas

Effective-stress strength parameters are estimated to be ϕ = 30° and c = 0, based on the plasticity index of the silts and clays. These values are very conservative for the sandy soils, which are characterized as dense based on their SPT N-values and the CPT Q_t data. Note, Appendix D of ConeTec (1999) indicates that ϕ based on the CPTs generally exceeds 35 to 40°.

The recommended coefficients of earth pressure for the silts and clays are as follows:

- At-rest, K_o, is 0.5
- Active, K_a, is 0.33
- Passive, K_p, is 3.0.

The recommended value of the coefficient of vertical subgrade reaction of the silt, silty clay, clayey silt for a 1 ft x 1 ft square is 100 kips/ft^3 for the clayey soils. Where the near-surface soils are cohesionless silts, this value should be 120 kips/ft^3. This value should be reduced for footing widths greater than 1 ft by applying a reduction factor, RF, calculated as follows:

For clayey soils: $RF = 1/B$

For cohesionless soils, $RF = [(B+1) / 2B]^2$

where B is the effective width of the footing.

This value should also be reduced for rectangular footings by (1 + 0.5 x B / L) / 1.5, where L is the effective length of the footing.

The recommended value of the coefficient of vertical subgrade reaction of the in situ clayey soils for use in design of the storage pads is 2.75 kips/ft^3, and for the cohesionless soils is 26 kips/ft^3.

The recommended value of the coefficient of horizontal subgrade reaction of the in situ clayey soils for use in the design of drilled caissons is 67 / B kips/ft^3. For cohesionless soils, the recommended value of the coefficient of horizontal subgrade reaction is 20 • z / B kips/ft^3.

Soil compressibility parameters and values of total-stress shear strength for the partially saturated silty clay/clayey silt were obtained from a number of tests to provide conservative results that were applicable for the upper 25 to 30 feet over the storage pad area because of the consistency of the subsurface conditions encountered in these borings. In addition, these results are considered to be conservative for the soils in the upper layer because they were obtained from testing specimens from the upper 25 to 30 feet where the Standard Penetration Test (SPT) blow count was less than or equal to the average value of all samples obtained in this layer, as indicated in Figure 2.6-27. Note, the SPT blow count is directly related to the density and strength of soils and inversely related to compressibility of soils.

Figure 54.4 of Terzaghi and Peck (1967) illustrates this for cohesionless soils. This figure presents the relationship between SPT blow counts (values of "N" in the figure), density, and compressibility of sands. It indicates that the density increases as the N-value increases. It also illustrates that a footing of a given width has a higher allowable soil pressure for a given settlement (1" in this chart) as the SPT blow count increases. Therefore, as the blow count increases, the strength of cohesionless soil increases and its compressibility decreases.

Table 45.2 of Terzaghi and Peck (1967) presents the relationship between consistency, SPT blow count, and strength of clay. This table indicates that the consistency increases from very soft to hard for blow counts ranging from less than 2 blows/ft to greater than 30 blows/ft, respectively. Table 2 of Terzaghi (1955) indicates that the coefficient of subgrade reaction, defined as the ratio between the pressure at a given point of the surface of contact and the settlement produced by that load, increases as the consistency of clay increases. Therefore, as the SPT blow count increases, the consistency of clay increases, and the compressibility (and, hence, settlement) decreases.

This has been demonstrated by the laboratory testing that was performed on samples obtained at greater depths in the Canister Transfer Building area. Additional laboratory tests were performed on samples of the soils from deeper within the profile than those that were tested (from depths of about 10 to 11 ft) in 1996. These tests, reported in Attachments 4 and 6 of Appendix 2A, indicate that the strengths of these deeper soils are higher than those tested in 1996 and their compressibilities are lower.

The depths of the specimens tested for strength and compressibility in Attachment 2 of Appendix 2A were selected to investigate conditions at a depth of about 10 feet below grade, which represents a depth of approximately ½ the width of the loaded area below the foundation due to the loading from the storage cask. It is generally acknowledged in geotechnical engineering that the zone of influence of loads on foundations spread out below the footing (e.g., Section 8.3 of Lambe and Whitman, 1969). The stress increase is greatest at the base of the footing, and it dissipates to an insignificant value at a depth of twice the width of the foundation. It is common practice to place a greater emphasis on the depth below the foundation equal to the width of the load. Testing the soils at ½ of this depth provides parameters that reflect the average performance of the soils within the depth equal to the width of the loaded area.

As indicated in Figure 2.6-27, the average and median values of the SPT blow counts, plotted for each 5-ft elevation interval versus elevation, illustrate that the blow counts increase with depth from grade. This figure also indicates that the locations of the specimens tested for strength and compressibility fall within the zone where the average and median blow counts for each 5-ft elevation interval were less than or equal to the average value for the entire layer (15 blows/ft). Since the strength of these soils is directly related to the blow count, testing soils whose blow count is less than the average provides a conservative estimate of the strength of the soil. In addition, since the compressibility of these soils is inversely related to their blow count, testing soils whose blow count is less than the average provides a conservative estimate of their compressibility and, hence, result in conservative (i.e., higher) estimates of settlements that the cask storage pads will experience.

Figure 2.6-20 plots all of the N-values vs depth for each of the borings drilled in the proposed emplacement area. The borings are plotted by row in the four sets of plots according to their locations in the field, as shown in Figure 2.6-2. That is, the top row of plots on Sheet 1 of Figure 2.6-20 includes the data from the northernmost row of borings, the next row down the sheet represents the next row of borings, moving south on the site, etc. The N-value plots in Figure 2.6-20 illustrate that the soils in the upper 25 to 30-ft thick layer of the profile do not vary significantly across the site.

Additional field work, performed at the site in 1998, is described in Geomatrix Consultants, Inc, (1999a) and included detailed lithostratigraphic soils mapping in test pits and trenches, as well as logging of continuous split-barrel samples in closely spaced boreholes. The results of these studies reaffirm the consistency of the upper layer of the subsurface profile across the site.

Subsurface profiles and stratigraphic descriptions are presented in Plates 3 and 4 in Geomatrix Consultants, Inc (1999a), and they illustrate convincingly that the subsurface conditions are very uniform. They identify a thin (<2.5 ft) surface layer of eolian silt and playa deposits with a poorly developed soil structure. This layer corresponds to the first SPT sample in the borings that were drilled in late 1996 (Attachment 1 of Appendix 2A) in the proposed pad emplacement area. This layer is underlain by a sequence of typical lacustrine sediments associated with several stages of Lake Bonneville, an inland sea that covered the area from about 30,000 to 10,000 years before present (B.P.). These sediments are, by and large, the fine-grained end members of a ternary diagram consisting of silt, clay, and sand. Samples are consistently described as silt, silty clay, clayey silt, or sandy silt. Geomatrix used the term marl or marly as an additional component of these descriptions, which refers to a high calcium carbonate content clay or silt deposited in a fresh-water environment (deep-water facies of Bonneville alloformation).

Geomatrix was able to subdivide the lacustrine sequence into several lake stages based on sedimentary relationships and physical characteristics exposed in continuous wall exposures in trenches and test pits. Their subdivisions of the Bonneville alloformation, presented in their Plate 3, "Map of North Wall Trench T-2", are as follows:

- Bonneville Deep-Water Blocky,
- Bonneville Deep-Water Laminated,
- Post-Stansbury Transgressive, and
- Stansbury Regressive.

This sequence extends to a depth of about 25 to 30 ft, where a continuous, nearly horizontal layer of dense, fine sand is encountered. This layer is the "Stansbury Transgressive", and it represents the oldest deposit of the Bonneville Cycle. The base of this unit occurs at a depth of about 45 to 50 ft and is believed to be an unconformity represented by the Promontory soil. This boundary is an apparent seismic velocity contrast that is recognizable on the recent seismic reflection profiles as a continuous, nearly horizontal layer, the Qp reflector (Geomatrix Consultants, Inc, 1999a).

Dynamic Strength of Cohesive Soils

It has been recognized in the past that the strength of cohesive soil increases as the rate of loading increases. For example, Casagrande and Shannon (1948) conducted soil dynamics investigations in 1948 with research efforts directed at finding the effects of rate of loading on soils common to the Panama Canal zone, i.e., clays, muck, shales, and dense dry sand. A "strain-rate" effect, defined as the ratio of maximum dynamic strength to the maximum static strength, was observed in all soils tested, except for the dry sand. Tests performed on Cambridge Clay (Cambridge, MA), showed that, tested at a rapid rate of loading (0.02 sec), the strength of the clay was approximately 1.9 times greater than that measured at a slow rate of loading (465 sec). This is illustrated in Figure 2.6-24.

Schimming et al (1966) studied the effects of loading rate on the strength of various soil types and defined the "apparent cohesion (c_a)" ratio to compare the dynamic and static failure envelopes of soil. Two different strain-rate strength tests were used in the study. For "dynamic" tests, the maximum shear force in soil specimens was attained within a period of 1 to 5 milliseconds after imposition of the initial force. Conversely, for "rapid static" tests, times to failure ranged from 30 seconds to nearly 50 seconds.

The c_a ratio is defined as: $c_a = c\,(\text{dynamic}) \div c\,(\text{rapid static})$.

Strength and index properties of the silty clay at the PFSF site are very similar to a soil studied by Schimming et al (i.e., Jordan Buff Clay). Average values of the index properties for both soils are as follows:

	PFSF Silty Clay	Jordan Buff Clay
Dry density (pcf)	65 (35)	86 (2)
Water content (%)	39 (117)	32 (2)
Liquid limit (LL)	51 (42)	54 (2)
Plastic limit (PL)	29 (42)	26 (2)
Plasticity index (PI)	21 (42)	28 (2)
Cohesion (psf)	1,100 (2)	1,124 (2)

Note: numbers in parentheses above indicate number of tests.

They report that the c_a ratio for this clay ranged from approximately 1.8 to 2.0, as shown in Figure 2.6-25.

Direct shear tests were performed on samples of the silty clay obtained from Elevation 4,468.4 to 4,469.4 in the Canister Transfer Building area, which is near the bottom of the foundation mat (Elevation 4,470). The results of these tests are included in Attachments 7 and 8 of Appendix 2A and they indicate that the average cohesion of these soils is ~1.1 ksf. The rate of loading used in these tests is slower than the "rapid static" tests performed by Schimming et al (1966). The rate of loading due to the design basis ground motion approximates those used for the "dynamic" tests performed by Schimming et al. To estimate the cohesion that will be available to resist these dynamic forces, the cohesion measured in the direct shear tests are multiplied by an estimated c_a ratio, which Schimming et al indicated varied between 1.8 and 2.1 for similar soils. Therefore, the cohesion available to resist forces caused by the design basis ground motion is estimated to be at least 1.5 to 2 times those measured in the direct shear tests. Stone & Webster (1995) used a similar approach for determining the dynamic strength of clays available to resist uplift loads on H-piles for Category I structures at the TVA's Sequoyah Nuclear Power Plant, Units 1 and 2.

Analyses of resistance to sliding of the Canister Transfer Building due to dynamic forces from the design basis ground motion, discussed in Section 2.6.1.12.2, are performed using a value of cohesion that is conservatively specified using c_a based on the lower bound of this range; i.e., only a 50% increase:

$$c_{dynamic} = 1.1 \text{ ksf} \times 1.5\,(c_a) = 1.65 \text{ ksf}.$$

The dynamic foundation parameters in support of the soil-structure interaction analyses are discussed in Section 2.6.2.1.

2.6.1.12 Stability of Foundations for Structures and Embankments

All exterior footings will be founded at a depth of no less than 30 inches below finished grade to provide protection against frost, in accordance with local code requirements. Interior footings in heated areas may be founded at shallower depths, if desired.

The minimum factor of safety against a bearing capacity failure due to static loads (dead load plus maximum live loads) is 3.0.

In accordance with the requirements of NUREG-75/087, Section 3.8.5, "Foundations," Section II.5, "Structural Acceptance Criteria," the recommended minimum factor of safety against overturning or sliding failure from static loads (dead load plus maximum live loads) is 1.5 and due to static loads plus loads from extreme environmental conditions, such as the design basis ground motion, is 1.1. In addition, it is recommended that a factor of safety of 1.1 be used to design footings against a bearing capacity failure from static loads plus loads due to the design basis ground motion.

If the factor of safety against sliding is less than 1 due to the design basis ground motion, additional analyses of the displacements the structure may experience are performed using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes to demonstrate that such displacements, if they did occur, would not have an adverse impact on the performance of the Important to Safety structures.

Recommended design earth pressure distributions are presented in Figure 2.6-7. Lateral earth pressures for determining driving forces shall be based on K_o, the at-rest earth pressure coefficient. These can be reduced to "active" earth pressures if the yield ratio exceeds 0.1%, where yield ratio, S/H, is defined as shown for the active case in Figure 2.6-8. In determining "passive" pressures resisting lateral movement, assume the lateral earth pressure coefficient varies from K_o at a yield ratio of 0% to a maximum of K_p at a yield ratio of 2%, where yield ratio, S/H, is defined as shown for the passive case in Figure 2.6-8. Compaction-induced lateral stresses are determined as shown in Figure 2.6-9.

2.6.1.12.1 Stability and Settlement Analyses—Cask Storage Pads

Bearing Capacity

The bearing capacity of the cask storage pads was determined using the general bearing capacity equation and associated shape, depth, and inclination factors, as presented in Das (1994). Refer to Calculation 05996.02-G(B)-04 (SWEC, 2000b) for details. These analyses are based on the strength parameters for the silty clay/clayey silts underlying the pads. Conservatively ignoring the presence of the soil cement that will be constructed adjacent to and beneath the cask storage pads (Figure 4.2-7) and the dense sand layer at a depth of ~25 to 30 ft, they demonstrate that there is an adequate factor of safety against a bearing capacity failure for both static and dynamic loadings.

As indicated below, the soil cement can be designed to have sufficient strength to provide, in passive resistance alone, all of the horizontal resistance required to obtain a factor of safety against sliding that exceeds the criterion (1.1) for dynamic loadings. The soil cement will be capable of resisting most of the horizontal forces due to the earthquake, which will greatly reduce the angle of inclination of the vertical load. The allowable bearing capacity is inversely related to the angle of inclination of the load, and it is markedly reduced for the inclination angles applicable for the dynamic horizontal loads from the design basis ground motion. The presence of the soil cement, therefore, will greatly improve the bearing capacity of these foundations.

These analyses included determination of factors of safety against a bearing capacity failure of the foundation due to static loads and due to static plus dynamic loads from the design basis ground motion (PSHA 2,000-yr return period earthquake). The dynamic bearing capacity analyses are discussed in detail in the section below titled *"Dynamic Bearing Capacity of the Cask Storage Pads."*

Static Bearing Capacity of the Cask Storage Pads

Table 2.6-6 presents the results of the bearing capacity analyses for the following static load cases. As indicated above, the minimum factor of safety for these static load cases is 3.

Case IA Static using undrained strength parameters (ϕ = 0° & c = 2.2 ksf).
Case IB Static using effective-stress strength parameters (ϕ = 30° & c = 0).
Case IC Static using total-stress strength parameters (ϕ = 21.3° & c = 1.4 ksf).

As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 4 ksf. However, loading the storage pads to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume ϕ = 0° and c = 2.2 ksf, as measured in the UU tests that are reported in Attachment 2 of Appendix 2A, to model the end of construction. Using the estimated effective-stress strength of ϕ = 30° and c = 0 or the total-stress strength of ϕ = 21.3° and c = 1.4 ksf, as measured in the consolidated undrained triaxial shear tests performed on samples obtained from the pad emplacement area (Attachment 8 Appendix 2A), results in higher allowable bearing pressures. As shown in Table 2.6-6, the gross allowable bearing capacities of the cask storage pads for static loads for these soil strengths are greater than 9 ksf and 12 ksf, respectively.

Effect of Cohesionless Soils Underlying the Cask Storage Pads on Bearing Capacity

Consolidated undrained triaxial test results included in Attachments 4 and 6 of Appendix 2A indicate that 2.2 ksf is a reasonable lower-bound value to use for bearing capacity analyses of the undrained conditions to represent the end-of-construction case. These triaxial tests were performed at confining pressures comparable to the estimated final stresses under the fully loaded pads and, thus, provide a realistic estimation of the minimum strength that will be available for resisting a bearing capacity failure at the end of construction.

As indicated in Section 2.6.1.6, based on the CPT program, most of the soils underlying the pad emplacement area are mischaracterized as soils that behave as "sandy" soils, rather than as cohesive soils. These soils were found to be mostly cohesive soils in the borings that were drilled in 1996, as indicated in Attachment 1 of Appendix 2A. The soil behavior types reported in ConeTec (1999) were determined based on correlations developed from testing saturated, uncemented soils. The soils at the site are partially saturated and cemented; thus, the soil behavior types determined from the cone penetration test data must be recalibrated to agree with the soil classifications determined based on samples obtained in the borings and tested in the laboratory.

Figure 2.6-30, Sheets 1 through 6, present comparisons of the boring and laboratory soil classifications plotted vs elevation alongside the soil behavior type data from nearby cone penetration tests. The differences shown under the column labeled ΔSBT represent the SBT zoning shift required to more correctly characterize these soils as a result of the effects of partial saturation and cementation, as discussed above. Evaluation of these ΔSBT values leads to the conclusion that the soil behavior type values reported in ConeTec (1999) that are greater than 5 (i.e., sandier soils), as well as some of those equal to 5 (i.e., clayey silts), typically should be adjusted downward one or two zones to more accurately reflect the soil classifications that were determined based on the borings and confirmed by the laboratory tests performed specifically for the purpose of classifying the soil types. Conservatively adjusting these data by subtracting 1 from the SBTs that were reported to be greater than 5 (i.e., "sandy" soils), as discussed in Section 2.6.1.6, results in the SBTs presented on the pad emplacement area foundation profiles, Sheets 2 through 14 of Figure 2.6-5. As shown on these figures, the subsurface soils that were reported in ConeTec (1999) as being silty sand/sand and sands are more correctly described as silts with some sandy silts. The following discussion is included to demonstrate that even if these soils are cohesionless soils, the factor of safety against a bearing capacity failure is much greater than that reported above for the clayey soils identified in the borings.

Whereas the bearing capacity of cohesive soils is a function of the strength of the soil, that of cohesionless soils is also a function of the width of the foundation. The foundations in question for this project have widths that are 30 ft or greater. Such large foundations, supported by soils having Standard Penetration Test blow counts that were measured for these soils, have much greater bearing capacities if they are founded on cohesionless soils than if supported by undrained cohesive soils. Therefore, characterizing the soils in the upper layer as cohesive even though some of these may be cohesionless provides a conservative estimate of the bearing capacity.

Analyses of bearing capacity were made in Calculation 05996.02-G(B)-4, (SWEC, 2000b), based on the assumption that the entire upper layer, approximately 25 to 30-ft thick, was comprised of cohesive soils similar to those tested at depths of 10 to 12 ft. In these analyses, the strength of the soils in the entire upper layer (~25 to 30-ft thick) was set equal to that measured in the UU tests (s_u > 2.2 ksf) that were performed at depths of approximately 10 to 12 feet. As indicated on Table 2.6-6 for Case IA, the factor of safety of the cask storage pad foundation is 6.3 using this undrained strength for the cohesive soils. The results for Cases IB and IC in Table 2.6-6 illustrate that the factor of safety against a bearing capacity failure increases to greater than 14 when the effective-stress strength of $\phi = 30°$ is used and greater than 19 when the total-stress strength of $\phi = 21.3°$ and c = 1.4 ksf is used. Therefore, all of these cases result in factors of safety against a bearing capacity that exceed the minimum allowable value of 3 for static loads.

The friction angle used in the effective-stress strength analyses discussed above is less than the friction angle shown for the soils that behave as sandy soils (SBT>5) based on the CPT data presented in Appendix D of ConeTec (1999). These plots illustrate that most of the "Phi" values are between 35° and 40° for these soils, with very few values that are slightly less than 35°. Therefore, assuming that all of the soils underlying the cask storage pads are cohesionless, as represented by the preponderance of soils that behave as "sandy" soils based on the uncorrected CPT SBT data, the factor of safety against a bearing capacity failure will be much greater than 14.

Static Settlements of the Cask Storage Pads

Analyses were performed to estimate the settlement of the cask storage pads as a result of the weight of the pad and the weight of eight, fully loaded, Holtec HI-STORM casks (356.5 K vs. 310 K for the SNC cask) in Calculation 05996.02-G(B)-3 (SWEC, 1999e). The actual bearing pressure for this case was about 1.9 ksf, and the estimated total settlement of the pad was determined to be about 3.3 inches. The total settlement consists of the following three components:

• Elastic settlement	0.5 inch
• Primary consolidation settlement	1.7 inches
• Secondary compression	1.1 inches
• Total estimated settlement	3.3 inches

In order to accommodate the total estimated settlement, the storage pads will be constructed 3.5 inches above adjacent finished grade. Exposed edges of the pads will be chamfered and the compacted aggregate surface material will be feathered to meet the edges of the raised pads for transporter access, as shown in Figure 4.2-7.

This settlement represents an upper-bound estimate of the total compression, because it was developed assuming that the consolidation characteristics that were measured for the clayey soils at a depth of about 10 ft are applicable for the entire upper layer. The SPT data from the borings and the CPT results indicate that the soils become stiffer within the 10 to 20 ft depth zone. Additional consolidation tests performed on samples obtained from depths of about 25 ft in the Canister Transfer Building area, reported in Attachment 6 of Appendix 2A, indicate that the soils at that depth are less compressible than those used to estimate the settlements presented above. Further, based on the CPT program, most of the soils underlying the pad emplacement area are characterized as soils that *behave* as "sandy" soils, rather than as cohesive soils. Such soils are much less compressible than the clayey soils described above. Therefore, assuming that the entire upper layer at the site was comprised of soils whose compressibilities are similar to those measured at a depth of 10 to 12 ft conservatively overestimates the expected settlements.

Effect of Cohesionless Soils Underlying the Cask Storage Pads on Settlements

As discussed above, the soil behavior types determined from the cone penetration test data and reported in ConeTec (1999) must be recalibrated to agree with the soil classifications determined based on samples obtained in the borings and tested in the laboratory. Figure 2.6-30, Sheets 1 through 6, present comparisons of the boring and laboratory soil classifications plotted vs elevation alongside the soil behavior type data from nearby cone penetration tests. These figures illustrate that the soil behavior type values reported in ConeTec (1999) that are greater than 5 (i.e., sandier soils), as well as some of those equal to 5 (i.e., clayey silts), typically should be adjusted downward one or two zones to more accurately reflect the soil classifications that were determined based on the borings and confirmed by the laboratory tests performed specifically for the purpose of classifying the soil types. The following discussion is included to demonstrate that even if these soils are cohesionless, the estimated settlements will be much less than those reported above assuming that the entire upper layer at the site was comprised of soils whose compressibilities are similar to those measured in consolidation tests performed on samples obtained at a depth of 10 to 12 ft.

A review of the CPT data (ConeTec, 1999) indicates that most of the soil behavior type (SBT) values represent soils whose behavior is similar to that of "sandy" soils. As indicated in Figure 5 of ConeTec (1999), these include SBT values that are greater than 5. A map was produced to show the thickness of those soils for which the soil behavior type values are greater than 5. The purpose of this map is to readily identify those areas where the subsurface profile differs from the assumption that the soils in the upper layer (~25 to 30 ft) are predominantly cohesive soils.

This map, titled "Contour Map Showing Thickness of Soils with CPT Soil Behavior Type > 5 (Sandy)", is included as Figure 2.6-29. The thickness of the soils beneath the cask storage pads that behave as "sandy" soils based on the CPT data are posted under the CPT identifiers shown on this plan view of the site. These values were calculated by subtracting the top three feet, to account for the proposed depth of the pads, as well as the total thickness of all zones where the SBT values were found to be less than 6, from the total depth of the CPT. The thicknesses were contoured to facilitate interpretation of the SBT > 5 data obtained in the CPT program. As indicated in the figure, the thickness of the soils that behave as sandy soils (SBT>5) based on the CPT data ranges from 13.8 feet at CPT-15, near the center of the pad emplacement area, to a high of 26.4 feet at CPT-33 near the center of the western edge of the pad emplacement area. The thicknesses are generally about 20 to 25 feet.

Calculation 05996.02-G(B)-3 (SWEC, 1999e) incorporated the calculation of settlements for the soils whose behavior is similar to that of "sandy" soils based on the CPT data. In this analysis, settlements are calculated based on Equation 6-17 of Lunne, Robertson, and Powell (1997), which was developed by Schmertmann (1970, 1978). This method is applicable for estimating settlements of foundations over sand using CPT data. The Schmertmann method takes into account the depth of footing, time of loading (40 years was used in the analysis), shape of the footing, and strain influence factor, which varies with depth. The equivalent Young's modulus, which appears in the equation, is related to the cone penetration resistance by a factor, α. This factor is related to the degree of loading, soil density, stress history, cementation, age, grain shape, and mineralogy of the deposit. In this analysis, α was assumed to be 5, which is in the middle of the range recommended in ConeTec (1998) for aged (>1,000 years) normally consolidated sands.

Two sets of estimated settlements were calculated and are summarized in the table presented on Page 44 of the calculation. Because of the preponderance of soils whose behavior is similar to that of "sandy" soils based on the uncorrected CPT soil behavior type data, settlements were calculated assuming that the Schmertmann method is applicable to the entire upper layer. As indicated by the left-hand column of settlements reported on Page 44 of the calculation, the estimated settlements for this case varied from 0.34 inches at CPT-26 to 0.56 inches at CPT-38.

The analyses were repeated, excluding those soils whose behavior is not similar to "sandy" soils, since the Schmertmann method is applicable only for cohesionless soils. In this analysis, cohesionless soils were defined as those with SBT values greater than

5, which includes silts, sandy silts, silty sands, and sands. The estimated settlements for this case are presented in the right-hand column on Page 44 of the calculation and range from 0.24 inches at CPT-31 to 0.50 inches at CPT-10.

These results are posted on the map showing the locations of the CPTs on Page 46 of the calculation. As indicated, the differential settlements between CPT locations average less than 0.1 inches. The maximum difference between two adjacent (diagonally) CPTs is 0.19 inches, CPT-34 to CPT-29. Total and differential settlements of this magnitude are not significant in the design or performance of the cask storage pads. These results confirm that if the soils are actually "sandy" soils, as indicated by the uncorrected SBTs from the cone penetration testing (ConeTec, 1999), then the estimated settlements will be much less than those reported above assuming that the entire upper layer at the site was comprised of soils whose compressibilities are similar to those measured in consolidation tests performed on samples obtained at a depth of 10 to 12 ft.

Dynamic Bearing Capacity of the Cask Storage Pads

The dynamic bearing capacity of the cask storage pads was analyzed in Calculation 05996.02-G(B)-4 (SWEC, 2000b) using two different sets of dynamic forces. The dynamic forces used in the first set of analyses were the inertial forces applicable for the peak ground accelerations from the design basis ground motion. The second set of analyses used the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 1999) for the pad supporting 2 casks, 4 casks, and 8 casks. These latter dynamic forces represent the maximum force occurring at any time during the earthquake at each node in the model used to represent the cask storage pads. These forces, therefore, represent an upper bound of the dynamic forces that could act at the base of the pad. As in the structural analyses discussed in Section 4.7.1.5.3., "Structural Analysis," the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions.

Table 2.6-7 presents the results of the bearing capacity analyses for the following cases, which include static loads plus inertial forces due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the total-stress strength parameters that were measured in consolidated-undrained triaxial tests (ϕ = 21.3° and c = 1.4 ksf). As indicated above, for these cases including dynamic loads from the design basis ground motion, the minimum acceptable factor of safety is 1.1.

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction.

As indicated in Table 2.6-7, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the inertial loads due to the design basis ground motion exceeds 10 ksf for all loading cases identified above. The minimum allowable value was obtained for Load Case IIIB, wherein 40% of the earthquake loads act in the N-S and Vertical directions and 100% acts in the E-W direction, tending to rotate the cask storage pad about the N-S axis. The actual factor of safety for this condition was 3.8, which is well above the criterion for dynamic bearing capacity (FS ≥ 1.1).

In these dynamic bearing capacity analyses, the dynamic forces were based on the inertial forces due to the earthquake. The total vertical force shown in Table 2.6-7 includes the static weight of the pad and 8 fully loaded casks ± the vertical inertial forces due to the earthquake. The vertical inertial force is calculated as a_V x [pad + cask dead loads], multiplied by the appropriate factor (±40% or ±100%) for the load case. In these analyses, the minus sign for the percent loading in the vertical direction signifies uplift forces, which tend to unload the pad. Similarly, the horizontal inertial forces are calculated as a_H x [pad + cask dead loads], multiplied by the appropriate factor (40% or 100%) for the load case. The horizontal inertial force from the casks was confirmed to be less than the maximum force that can be transmitted from the cask to the pad through friction for each of these load cases. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad considered in the HI-STORM cask stability analysis (μ = 0.8, as shown in Section 8.2.1.2, Accident Analysis) x the normal force acting between the casks and the pad.

The lower-bound friction case (discussed in Section 4.2.3.5.1B), wherein μ between the steel bottom of the cask and the top of the concrete storage pad = 0.2, results in lower horizontal forces being applied at the top of the pad. This decreases the inclination of the load applied to the pad, which results in increased bearing capacity. Therefore, bearing capacity analyses are not performed for μ = 0.2 in Calculation 05996.02-G(B)-04 (SWEC, 2000b).

Table 2.6-8 presents a summary of the bearing capacity analyses that were performed using the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 1999) for the pad supporting 2 casks, 4 casks, and 8 casks. As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the very conservative maximum dynamic cask driving forces due to the design basis ground motion is approximately 20 ksf for the 2-cask, 4-cask, and 8-cask loading cases. The minimum allowable value was obtained for the 8-cask loading. The actual factor of safety for this case was 3.2, which is well above the criterion for dynamic bearing capacity (FS ≥ 1.1).

As indicated above, these maximum dynamic cask driving forces represent the upper bound of the dynamic forces that could act at the base of the pad. The horizontal forces from the casks were confirmed to be less than the maximum force that can be transmitted from the cask to the pad through friction acting at the base of the cask for each of these load cases. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad (μ = 0.8, as shown in Section 8.2.1.2) x the normal force acting between the casks and the pad. These maximum dynamic cask driving forces can be transmitted to the pad through friction only when the inertial vertical forces act downward; therefore, these analyses are performed only for Load Case IV. The analyses conservatively assume that 100% of the horizontal forces act in the E-W and vertical directions at the same time. The width (30 ft) is less in the E-W direction than the length N-S (64 ft); therefore, the E-W direction is the critical direction with respect to a bearing capacity failure.

Because of the nature of the subsurface materials, dynamic settlements due to the design basis ground motion are not expected to occur. See Section 2.6.4.7 for more details.

<u>Overturning Stability of the Cask Storage Pads</u>

The factor of safety against overturning is defined as:

$$FS_{OT} = \Sigma M_{Resisting} \div \Sigma M_{Driving}$$

The resisting moment is calculated as the weight of the pad and casks x the distance from one edge of the pad to the center of the pad in the direction of the minimum width. The weight of the pad is calculated as 3 ft x 64 ft x 30 ft x 0.15 kips/ft^3 = 864 K, and the weight of 8 casks is 8 x 356.5 K/cask = 2,852 K. The moment arm for the resisting moment equals ½ of 30 ft, or 15 ft. Therefore,

$$\Sigma M_{Resisting} = [864\text{ K} + 2{,}852\text{K}] \times 15\text{ ft} = 55{,}740\text{ ft-K.}$$

The driving moment includes the moments due to the horizontal inertial force of the pad x ½ the height of the pad, the vertical inertial force of the pad plus casks x ½ the minimum width of the pad, and the horizontal force from the casks acting at the top of the pad x the height of the pad. The casks are simply resting on the top of the pads; therefore, this force cannot exceed the friction force acting between the steel bottom of the cask and the top of the concrete storage pad. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad (μ = 0.8, as shown in Section 8.2.1.2) x the normal force acting between the casks and the pad. This force is maximum when the vertical inertial force due to the earthquake acts downward. However, when the vertical force from the earthquake acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the dynamic vertical force acts in the upward direction, tending to unload the pad.

When the vertical inertial force due to the earthquake acts upward, the friction force = 0.8 x (2,852K - 0.533 x 2,852K) = 1,066 K. This is less than the maximum dynamic cask horizontal driving force of 1,855 K (Table D-1(c) in CEC, 1999). Therefore, the worst-case horizontal force that can occur when the vertical earthquake force acts upward is limited by the upper-bound value of the coefficient of friction between the bottom of the casks and the top of the storage pad, and it equals 1,066K.

$$\Sigma M_{Driving} = 1.5 \text{ ft} \times 0.528 \text{ X } 864 \text{ K} + 0.533 \times [864 \text{ K} + 2{,}852 \text{ K}] \times 15 \text{ ft} + 3 \text{ ft} \times 1{,}066 \text{ K} = 33{,}592 \text{ ft-K}.$$

Therefore, $FS_{OT} = 55{,}740 \div 33{,}592 = 1.66$

This is greater than the criterion of 1.1; therefore, the cask storage pads have an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion.

Sliding Stability of the Cask Storage Pads

The sliding stability analyses of the cask storage pads are presented in Calculation 05996.02-G(B)-4 (SWEC, 2000b). These pads will be constructed on and within soil cement, as illustrated in Figure 4.2-7 and described in Sections 2.6.1.7 and 2.6.4.11. The following section discusses the sliding stability of these pads embedded in soil cement and demonstrates that embedding them in soil cement will greatly enhance their resistance to sliding due to dynamic loads from the design basis ground motion.

Subsequent sections demonstrate that sliding will not occur along deeper surfaces within the profile underlying the cask storage pads. First, the sliding resistance of the *in situ* silty clay/clayey silt layer is addressed to demonstrate that sliding will not occur along the interface between the bottom of the soil cement and those soils. As shown in the pad emplacement area foundation profiles (Figures 2.6-5), a layer, composed in part of sandy silt, underlies the clayey layer at a depth of about 10 ft below the cask storage pads. Sandy silts oftentimes are cohesionless; therefore, the subsequent section addresses the possibility that sliding may occur along a deep slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

These analyses demonstrate there is an adequate factor of safety against sliding of the cask storage pads and along deeper surfaces beneath the storage pads due to the maximum loadings of design basis ground motion.

Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement

The analysis of sliding stability of the cask storage pads embedded in soil cement is included in Calculation 05996.02-G(B)-4 (SWEC, 2000b). This analysis demonstrates that the soil cement can be designed to provide sufficient resistance, considering only the passive resistance of the soil cement, to provide a factor of safety against sliding that exceeds the minimum required value of 1.1 for the maximum loadings due to the PSHA 2,000-yr return period earthquake. This analysis conservatively ignores the resistance along the base of the pads due to friction and cohesion, which will also be available to resist sliding during the earthquake. Therefore, embedding the pads within soil cement will greatly improve their stability against sliding.

The passive resistance required to obtain a factor of safety against sliding of 1.1 was calculated based on $K_p = 1.0$, which is applicable for $\phi = 0$. This is conservative, because the soil cement likely will have $\phi > 40°$, based on Tables 5 and 6 of Nussbaum and Colley (1971). Based on these conservative assumptions, the soil cement would need to have an unconfined compressive strength, f_c, of ~250 psi to provide sufficient thrust from passive resistance alone to obtain a factor of safety against sliding that is greater than the minimum required value of 1.1.

Soil cement with strengths higher than this are readily achievable. As illustrated by the lowest curve in Figure 4.2 of ACI (1998), which applies for fine-grained soils similar to the eolian silts in the pad emplacement area, $f_c = 40C$, where C = percentage of cement required in the soil-cement mix. To obtain f_c of 250 psi, ~6.25% cement would be required. This is even less cement than would typically be used in constructing soil cement for use as road base. Further, f_c required would be even less if the passive resistance was calculated using K_p applicable for $\phi > 0$ or if the shear resistance acting on the base of the cask storage pads were included.

If the passive resistance acting on the sides of the pads by the soil cement is ignored, the minimum shear strength required at the base of the pad foundation and within the underlying soils to provide a factor of safety against sliding that is greater than 1.1 is approximately 9.5 psi (1.36 ksf). This value is based on the maximum dynamic cask driving forces presented in Table D-1(c) of Calculation 05996.02-G(PO17)-2 (CEC, 1999) and is calculated as follows:

$F_A =$ 69.1 K (Combined static and dynamic active earth pressure from Calculation 05996.02-G(B)-4, SWEC, 2000b).

$EQh_{pad} =$ 456 K (= 0.528 x 3' x 64' x 30' x 0.15 K/ft³)

$EQh_{casks} =$ 1,855 K

$$\tau_{reqd} = \frac{(69.1\text{ K} + 456\text{ K} + 1{,}855\text{ K}) \times 1.1 \times 1{,}000\text{ lbs/K}}{30' \times 64' \times 144\text{ in.}^2/\text{ft}^2} = 9.5\text{ psi}$$

The minimum normal load acts at the base of the pad when the inertial force of the design basis ground motion acts upward. Calculation 05996.02-G(B)-04 (SWEC, 2000b) indicates that this minimum normal force is 1,735 K. Conservatively assuming that the friction angle of the soil cement is equal to that of the silty clay/clayey silt, ϕ = 24.9°. The lower-bound value of the frictional portion of the sliding resistance is calculated as:

$$T_f = N \times \tan\phi = 1{,}735\ K \times \tan 24.9° = 805\ K$$

Thus, the portion of the shear strength attributed to frictional resistance is 805 K / (30' x 64'), or 0.42 ksf, which equals 2.91 psi. The remaining shear strength, which equals 9.5 – 2.9 = 6.6 psi (0.95 ksf), can be provided easily by the cohesion of the soil cement. This also is less than the cohesion of the silty clay/clayey silt layer; therefore, the shear resistance required at the base of the pads can be provided easily by the bond between the pad foundation and soil-cement contact and the cohesive strength of the soil cement.

As indicated in Figure 4.2-7, the soil cement will extend at least 1 ft below all of the cask storage pads. As shown in Figures 2.6-5, the pad emplacement area foundation profiles, it typically will extend 3 to 5 ft below most of the pads. Shear resistance will be transferred through the approximately 3-ft thick soil-cement layer and into the underlying silty clay/clayey silt subgrade. Additional resistance will be provided by the continuous layer of soil cement under and between the pads; thus, the area available to resist sliding will greatly exceed that of the embedded portion of the pads alone, as was used in the analysis described above. Shear resistance requirements at the soil cement/clayey silt interface, therefore, will be lower than those required to construct the pads directly on the silty clay/clayey silt without the proposed soil-cement layer.

The soil cement will have higher cohesive and frictional strength than the underlying silty clay/clayey silt layer; therefore, resistance to sliding on that interface will be limited by the strength of the silty clay/clayey silt. Direct shear tests on samples of these soils (presented in Attachments 7 and 8 of Appendix 2A) indicate the total-stress strength available to resist sliding during the design basis ground motion is ϕ = 24.9°and c = 1.22 ksf. The following section indicates that there is an adequate factor of safety against sliding of the pads, postulating that they are constructed directly on the silty clay/clayey silt without the soil cement. The factor of safety against sliding along the soil cement/silty clay interface will be much greater than this, because the shearing resistance will be available over the areas between the pads, as well as under the pads, and additional passive resistance will be provided by the continuous soil cement layer existing below the pads. Therefore, the soil cement will greatly improve the sliding stability of the cask storage pads.

Sliding Stability of the Interface Between the Soil Cement and the Silty Clay/Clayey Silt Underlying the Cask Storage Pads

The sliding stability of the interface between the soil cement and the in situ silty clay/clayey silt layer underlying cask storage pads is presented in Calculation 05996.02-G(B)-4 (SWEC, 2000b). The sliding stability of this interface is demonstrated by ignoring the presence of the soil cement and demonstrating that the factor of safety against sliding of the pads supported directly on the in situ clayey soils is ~2, which provides an adequate margin against sliding. As discussed above, the soil cement will distribute the loads from the earthquake deeper into the profile, spreading them out over an area that is much greater than that of the pads. Thus, the shear resistance requirements at the bottom of the soil cement will be less than would be required if the pads were constructed directly on the clayey soils. Therefore, sliding will not occur along the interface between the soil cement and the in situ clayey soils.

In these analyses, the factor of safety (FS) against sliding is defined as:

$$FS = \text{resisting force} \div \text{driving force}$$

The resisting force, or tangential (T) shear force, below the base of the pad is defined as:

$$T = N \tan \phi + c\, B\, L$$

where, N = normal force

ϕ = 24.9° (for Silty Clay/Clayey Silt)

c = 1.22 ksf

B = 30 feet

L = 64 feet

Material around the pad will be soil cement. In this analysis, the passive resistance provided by the soil cement was ignored to demonstrate that there is an acceptable factor of safety against sliding of the pads if they were founded directly on the silty clay/clayey silt. The soil cement is assumed to have the same properties that were used in Rev 4 of this calculation to model the crushed stone (compacted aggregate) that was originally proposed adjacent to the pads. These include:

γ = 125 pcf — Because of the low density of the eolian silts that will be used to construct the soil cement, it is likely that γ will be less than this value. It is conservative to use this higher value, because it is used in this analysis only for determining upper-bound estimates of the active earth pressure acting on the pad due to the design basis ground motion.

ϕ = 40° — Tables 5 & 6 of Nussbaum & Colley (1971) indicate that ϕ exceeds 40° for all A-4 soils (CL & ML, similar to the eolian silts at the site) treated with cement; therefore, it is likely that ϕ will be higher than this value. This value is not used, however, in this analysis for calculating sliding resistance. It also is used in this analysis only for determining upper-bound estimates of the active earth pressure acting on the pad due to the design basis ground motion.

H = 3 ft As shown in SAR Figure 4.2-7, the pad is 3 ft thick, but it is constructed such that the top is 3.5" above grade to accommodate potential settlement. The depth of the pad is used in this analysis only for calculating the maximum dynamic lateral earth pressure; therefore, it is conservative to ignore the 3.5" that the pad sticks out of the ground.

The values of ϕ and c are based on the results of the direct shear tests that were performed on specimens obtained from a depth of 5 to 6.3 ft from Sample U-1 in Boring C-2, which was drilled in the pad emplacement area. These test results are consistent with the results obtained from the direct shear tests that were performed on samples obtained from within the Canister Transfer Building area (Borings CTB-6 and CTB-S). All of these direct shear test results are reported in Attachments 7 and 8 of Appendix 2A. Minimum sliding resistance exists when the dynamic forces due to the vertical component of the earthquake act in an upward direction. In determining the resisting forces in these analyses, no credit is taken for passive resistance acting on the embedded pad.

The sliding stability was checked for Load Cases III and IV, conservatively assuming that 100% of the dynamic forces due to the earthquake act in both the N-S and Vertical directions at the same time.

The resistance to sliding is minimum when the forces due to the earthquake act upward; i.e., Load Case III. However, the maximum horizontal force that can be applied to the top of the pad by the casks is limited to the maximum value of the coefficient of friction between the cask and the top of the pad, which equals 0.8, multiplied by the cask normal force. For Load Case III, the maximum frictional force was much less than the maximum cask driving force. Therefore, the sliding stability also was checked for Load Case IV, which has the dynamic forces due to the earthquake acting downward. With the earthquake force acting downward, the frictional force that can be transmitted from the casks to the top of the pad is large enough to transmit the maximum cask driving force.

The horizontal driving force in these analyses includes the inertial forces of the pad and the maximum cask driving forces reported in Calculation 05996.02-G(PO17)-2 (CEC, 1999) that are due to the PSHA 2,000-yr return period earthquake. The dynamic loads due to soil pressures acting on the embedded pad were also included, calculated based on the Mononobe-Okabe method, as described in Seed and Whitman (1970). The driving forces were calculated based on the peak vertical and peak horizontal accelerations; i.e., no credit was taken for the fact that these peaks are not expected to occur at different times.

The driving force due to dynamic active earth pressures acting on the pad in the E-W direction are greater than those acting in the N-S direction, because the dimension of the pad in the N-S direction (64 ft) is greater than twice the width in the E-W direction (30 ft). Therefore, ignoring passive resistance, sliding will be more critical in the E-W direction. The maximum dynamic cask driving force, however, acts in the N-S direction. To be conservative, this analysis assumed that driving force due to dynamic active earth pressures calculated for the E-W direction acts in the N-S direction. However, the maximum horizontal force that can be applied to the top of the pad by the casks is limited to the maximum value of the coefficient of friction between the cask and the top of the pad, which equals 0.8, multiplied by the cask normal force.

As indicated above, these analyses are very conservative for a number of reasons. They combine the maximum horizontal and vertical forces of the earthquake, rather than using reduced values to account for the fact that the peaks in these motions are not expected to occur at the same time. They also conservatively use the shear strength parameters as measured in the static direct shear tests; i.e., no credit is taken for the increase in this strength that is applicable for dynamic loadings, as discussed in Section 2.6.1.11 under "Dynamic Strength of Cohesive Soils." Therefore, these analyses yield lower-bound factors of safety against sliding where the pads are supported on clayey soils. The resulting factor of safety for both load cases is ~2, which provides an adequate margin against sliding.

Sliding Stability of the Cask Storage Pads on Cohesionless Soils

Adequate factors of safety against sliding due to maximum forces from the design basis ground motion have been obtained for the storage pads founded directly on the silty clay/clayey silt layer, conservatively ignoring the passive resistance of the soil cement that will be placed under and adjacent to the pads. Much of the shearing resistance is provided by the cohesive portion of the shear strength of the silty clay/clayey silt layer, which is not affected by upward earthquake loads. As shown in SAR Figures 2.6-5, Pad Emplacement Area – Foundation Profiles, a layer, composed in part of sandy silt, underlies the clayey layer at a depth of about 10 ft below the cask storage pads. Sandy silts oftentimes are cohesionless; therefore, to be conservative, the sliding stability of the cask storage pads was analyzed assuming that the soils in this layer are cohesionless, ignoring the effects of cementation that were observed on many of the split-spoon and thin-walled tube samples obtained in the drilling programs.

The CPT results (ConeTec, 1999) indicate the presence of a layer of soils that behave like silty sands and sands under the clayey layer at a depth of about 10 ft. Note, however, that recalibrating the SBTs as discussed in Section 2.6.1.6 results in most of these silty sands and sands being more correctly identified as clayey silt/silt with some sandy silt, as shown in Sheets 2 through 14 of Figure 2.6-5. The plots included in Appendix D of ConeTec, 1999) indicate that s_u, the undrained shear strength, or the cohesion, drops to 0 and that ϕ is generally greater than 35 to 40° for these soils. If the cohesion available to resist sliding drops to 0 and cementation effects are ignored, the shearing resistance of this layer is directly related to the normal stress.

Analyses were performed to address the possibility that sliding may occur along a deep slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces. To simplify the analysis, it was assumed that cohesionless soils extend above the 10 ft depth and, thus, the pads are founded directly on cohesionless materials. In this analysis, a friction angle of 30° was used to define the strength of the soils to conservatively model a loose cohesionless layer, even though the values measured in the CPTs generally were greater than of 35 to 40°. Without cohesion and ignoring passive resistance acting against the side of the pad, the resistance to sliding is calculated as N tan 30°, or 0.58 N, where N is the normal force. Because of the magnitude of the peak ground accelerations (0.53g) due to the design basis ground motion at this site, the frictional resistance available when N is reduced due to the uplift from the inertial forces applicable for the vertical component of the design basis ground motion is not sufficient to resist sliding. However, analyses were performed to estimate the amount of displacement that might occur due to the design basis ground motion for this case. These analyses, based on the method of estimating displacements of dams and embankments during earthquakes developed by Newmark (1965), indicate that even if these soils are cohesionless and even if they are conservatively located directly at the base of the pads, the estimated displacements would be less than ½ inch. Whereas there are no connections between the ground and these pads or between the pads and other structures, this minor amount of displacement will not adversely affect the performance of these structures.

The resistance to sliding on cohesionless materials is lowest when the dynamic forces due to the design basis ground motion act in the upward direction. The dynamic vertical force acting upward reduces the normal force and, hence, the shearing resistance, at the base of the foundations. Thus, the analyses were performed for Load Cases IIIA, IIIB, and IIIC, in which the pads are unloaded due to uplift from the earthquake forces. These load cases included:

Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.

Newmark's Method of Estimating Displacements Due to Earthquakes

Newmark (1965) defines N W as the steady force applied at the center of gravity of the sliding mass in the direction in which the force can have its lowest value to just overcome the stabilizing forces and keep the mass moving. For a block sliding on a horizontal surface, N W = T, where T is the shearing resistance between the base of the block on the sliding surface.

Shearing resistance, $T = \tau \times \text{Area}$

where:

$\tau = \sigma_n \tan \phi$

σ_n = Normal Stress

ϕ = Friction angle of cohesionless layer

$$\sigma_n = (\text{Net Vertical Force}) / \text{Area} = (F_v - F_{v(Eqk)}) / \text{Area}$$

$$T = (F_v - F_{v(Eqk)}) \tan \phi$$

$$N W = T$$

$$N = [(F_v - F_{v(Eqk)}) \tan \phi] / W$$

The maximum relative displacement of the pad relative to the ground, u_m, is calculated as

$$u_m = [V^2 (1 - N/A)] / (2gN)$$

The maximum horizontal ground velocities required as input in Newmark's method of analysis of displacements due to earthquakes were estimated for the cask storage pads assuming that the ratio of the maximum ground velocity to the maximum ground acceleration equaled 48 (i.e., 48 in./sec per g). The accelerations used for the cask storage pads were those due to the PSHA 2,000-yr return period earthquake; i.e., a_H = 0.528g.

The above expression for the relative displacement is an upper bound for all the data points for N/A less than 0.15 and greater than 0.5, as shown in Figure 2.6-26, which is a copy of Figure 21 of Newmark (1965). Within the range of 0.5 to 0.15 the following expression gives an upper bound for all data.

$$u_m = V^2 / (2gN)$$

The following table presents the results (from Calculation 05996.02-G(B)-4, SWEC, 2000b) of estimating the horizontal displacements that the cask storage pads might experience due to the PSHA 2,000-yr return period earthquake if they were founded directly on cohesionless soils with ϕ = 30°.

LOAD COMBINATION				DISPLACEMENT
Case IIIA	40% N-S,	-100% Vertical,	40% E-W.	0.4 inches
Case IIIB	40% N-S,	-40% Vertical,	100% E-W.	0.1 inches
Case IIIC	100% N-S,	-40% Vertical,	40% E-W.	0.1 inches

The estimated relative displacement of the cask storage pads ranges from ~0.1 inches to 0.4 inches. Because there are no connections between the pads or between the pads and other structures, displacements of this magnitude, were they to occur, would not adversely impact the performance of the cask storage pads. There are several conservative assumptions that were made in determining these values and, therefore, the estimated displacements represent upper-bound values.

The soils in the layer that are assumed to be cohesionless, the one ~10 ft below the pads that is labeled "Clayey Silt/Silt & Some Sandy Silt" in the foundation profiles in the pad emplacement area (SAR Figures 2.6-5, Sheets 1 through 14), are clayey silts and silts, with some sandy silt. To be conservative in this analysis, these soils are assumed to have a friction angle of 30°. However, the results of the cone penetration testing (ConeTec, 1999) indicate that these soils have ϕ values that generally exceed 35 to 40°, as shown in Appendices D & F of ConeTec (1999). These high friction angles likely are the manifestation of cementation that was observed in many of the specimens obtained in split-barrel sampling and in the undisturbed tubes that were obtained for testing in the laboratory. Possible cementation of these soils is also ignored in this analysis, adding to the conservatism.

In addition, this analysis postulates that cohesionless soils exist directly at the base of the pads. In reality, the surface of these soils is 10 ft or more below the pads, and it is not likely to be continuous, as the soils in this layer are intermixed. For the pads to slide, a surface of sliding must be established between the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft below the pads, through the overlying clayey layer, and daylighting at grade. As shown in the analysis preceding this section, the overlying clayey layer is strong enough to resist sliding due to the earthquake forces. The contribution of the shear strength of the soils along this failure plane rising from the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft to the resistance to sliding is ignored in the simplified model used to estimate the relative displacement, further adding to the conservatism.

These analyses also conservatively ignore the presence of the soil cement under and adjacent to the cask storage pads. As shown below, this soil cement can easily be designed to provide all of the sliding resistance necessary to provide an adequate factor of safety, considering only the passive resistance acting on the sides of the pads, without relying on friction or cohesion along the base of the pads. Adding friction and cohesion along the base of the pads will increase the factor of safety against sliding.

2.6.1.12.2 Stability and Settlement Analyses—Canister Transfer Building

In addition to the finite element, soil-structure interaction analysis described in Chapter 4, conventional static and dynamic stability analyses of the building mat foundation were performed. These included bearing capacity, overturning, and sliding stability analyses. These analyses, performed in Calculation 05996.02-G(B)-13 (SWEC, 2000c), are discussed below. These analyses indicate that the building is stable and the performance of the structure will not be adversely affected by the estimated settlements or seismic displacements.

The Canister Transfer Building is a large and massive building consisting of exterior reinforced concrete walls 2'-0" thick, a reinforced concrete roof 1'-0" thick, and a solid reinforced concrete mat foundation 5'-0" thick. The interior partitions that make up the low level waste holding area will be constructed of concrete or concrete masonry. The equipment and office areas on the east side of the building will utilize steel-framed partition walls covered with gypsum board. The total weight (static load) of the building and foundation is approximately 73,000 kips (Calculation 05996.02-SC-5, SWEC, 1999f) or 36,500 tons.

Bearing Capacity of the Canister Transfer Building

The bearing capacity of the Canister Transfer Building foundation was determined using the general bearing capacity equation and associated shape, depth, and inclination factors, as presented in Das (1994). Refer to Calculation 05996.02-G(B)-13 (SWEC, 2000c) for details. These analyses are based on the strength parameters for the silty clay/clayey silt layer directly underlying the mat. Conservatively ignoring the presence of the denser layers, which start at a at a depth of ~25 to 30 ft, these analyses demonstrate that there is an adequate factor of safety against a bearing capacity failure for both static and dynamic loadings. They included determination of factors of safety against a bearing capacity failure of the foundation due to static loads and due to static plus dynamic loads from the design basis ground motion (PSHA 2,000-yr return period earthquake). The dynamic bearing capacity analyses are discussed in detail in the section below titled *"Dynamic Bearing Capacity of the Canister Transfer Building."*

Static Bearing Capacity of the Canister Transfer Building

Table 2.6-9 presents the results of the bearing capacity analyses for the following static load cases. As indicated above, the minimum factor of safety required for static load cases is 3.

Case IA Static using undrained strength parameters ($\phi = 0°$ & $c = 2.2$ ksf).

Case IB Static using effective-stress strength parameters ($\phi = 30°$ & $c = 0$).

Case IC Static using total-stress strength parameters (ϕ = 21.1° & c = 1.1 ksf).

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 4 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume ϕ = 0° and c = 2.2 ksf, as measured in the UU tests that are reported in Attachment 2 of Appendix 2A, to model the end of construction. Using the estimated effective-stress strength of ϕ = 30° and c = 0 or the total-stress strength of ϕ = 21.1° and c = 1.1 ksf, as measured in the consolidated undrained triaxial shear tests performed on samples obtained from the Canister Transfer Building area (Attachment 6 Appendix 2A), results in higher allowable bearing pressures. As shown in Table 2.6-9, the gross allowable bearing capacities of the cask storage pads for static loads for these soil strengths are greater than 20 ksf.

Settlement of the Canister Transfer Building

Analyses were performed to estimate the settlement of the Canister Transfer Building for the static dead and live loads in Calculation 05996.02-G(C)-14 (SWEC, 1998). A total building settlement of approximately 3 inches is estimated over the life of the building. The settlement will be generally uniform. Of the total building settlement, approximately 1.9 inches will occur within a few years after construction and an additional 1.1 inches will occur during the life of the building. These analyses were performed using the results of the consolidation tests that are included in Attachment 2 of Appendix 2A.

As indicated in Section 2.6.1.12.1 regarding the settlement analyses of the storage pads, this settlement represents an upper-bound estimate of the settlement, because it was developed assuming that the consolidation characteristics that were measured for the clayey soils at a depth of about 10 ft are applicable for the entire upper layer. The SPT data from the borings and the CPT results indicate that the soils become stiffer within the 10 to 20 ft depth zone. Additional consolidation tests performed on samples obtained from depths of about 25 ft in the Canister Transfer Building area, reported in Attachment 6 of Appendix 2A, indicate that the soils at that depth are less compressible than those used to estimate these settlements.

Dynamic Bearing Capacity of the Canister Transfer Building

The dynamic bearing capacity was analyzed in Calculation 05996.02-G(B)-13 (SWEC, 2000c) using the dynamic loads for the building that were developed in Calculation 05996.02-SC-5, (SWEC, 1999f). The development of these dynamic loads is described in Section 4.7.1.5.3. As in the structural analyses discussed in Section 4.7.1.5.3., "Structural Analysis," the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions.

Table 2.6-10 presents the results of the bearing capacity analyses for the following cases, which include static loads plus dynamic loads due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the total-stress strength parameters that were measured in consolidated-undrained triaxial tests (ϕ = 21.1° and c = 1.1 ksf). As indicated above, for these cases including dynamic loads from the design basis ground motion, the minimum acceptable factor of safety is 1.1.

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.

Case IVB 40% N-S direction, 40% Vertical direction, 100% E-W direction.

Case IVC 100% N-S direction, 40% Vertical direction, 40% E-W direction.

Table 2.6-10 indicates the minimum factor of safety against a dynamic bearing capacity failure was obtained for Load Case IIIA, the load combination of full static, 100% seismic uplift, and 40% of the seismic forces in both horizontal directions. This load case resulted in an actual soil bearing pressure of 2.5 kips per square foot (ksf), compared with an ultimate bearing capacity of 2.7 ksf. The resulting factor of safety against a bearing capacity failure for this load case is ~1.1, the minimum allowable factor of safety for seismic loading cases.

In these analyses, no credit was taken for the fact that strength of cohesive soil increases as the rate of loading increases. As indicated in the section titled "Dynamic Strength of Cohesive Soils" in Section 2.6.1.11, above, $c_{dynamic}$ is conservatively estimated to be 1.65 ksf for the clayey soils underlying the Canister Transfer Building. For $\phi = 21.1°$ and c = 1.65 ksf, the ultimate bearing capacity for the loads in Case IIIA increases to 3.9 ksf, resulting in a factor of safety against a bearing capacity failure for this load case of 1.5. Therefore, the Canister Transfer Building has an adequate factor of safety against a dynamic bearing capacity failure.

<u>Overturning Stability of the Canister Transfer Building</u>

The overturning stability of the Canister Transfer Building was analyzed in Calculation 05996.02-G(B)-13 (SWEC, 2000c) using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake, which were developed in Calculation 05996.02-SC-5 (SWEC, 1999f). The development of these dynamic loads is described in Section 4.7.1.5.3. The masses and accelerations of the joints used in the model of the Canister Transfer Building in Calculation 05996.02-SC-5 are listed on the left side of Table 2.6-11, and the resulting inertial forces and associated moments are listed on the right. Based on building geometry and the forces and moments shown in Table 2.6-11, overturning is more critical about the N-S axis (~265 ft) than about the E-W axis (~165 ft).

The factor of safety against overturning is defined as:

$$FS_{OT} = \Sigma M_{Resisting} \div \Sigma M_{Driving}$$

The resisting moment is calculated as the weight of the building x the distance from one edge of the mat to the center of the mat. The weight of the building is 72,988 K, as shown in Table 2.6-11. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of ~165 ft, or 82.5 ft. Therefore,

$$\Sigma M_{Resisting} = 72{,}988 \text{ K} \times 82.5 \text{ ft} = 6{,}021{,}510 \text{ ft-K}.$$

The driving moments include the $\Sigma M_{N\text{-}S}$, which is 2,513,041 ft-K, and the moment due to the uplift force ($\Sigma F_{V\,dyn}$ = 57,139 K) x ½ the width of the mat. The vertical force due to the earthquake can act upward or downward. However, when it acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the dynamic vertical force acts in the upward direction, tending to unload the mat.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions at the same time. The moments acting about the E-W axis do not contribute to overturning about the N-S axis; therefore,

$$\sum M_{Driving} = \sqrt{2{,}513{,}041^2 + (57{,}139 \text{ K} \times 82.5 \text{ ft})^2} = 5{,}341{,}991 \quad \text{ft} - \text{K}$$

and $FS_{OT} = 6{,}021{,}510 \div 5{,}341{,}991 = 1.13$

This is greater than the criterion of 1.1; therefore, the Canister Transfer Building has an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion.

Sliding Stability of the Canister Transfer Building

The Canister Transfer Building will be founded on clayey soils, as indicated in Figures 2.6-21 through 2.6-23. The sliding stability was evaluated in Calculation 05996.02-G(B)-13 (SWEC, 2000c) using the same method that was used for storage pads. Refer to Section 2.6.1.12.1 for details. The loads used in this analysis were developed in the soil-structure interaction analyses (Calculation 05996.02-SC-5, SWEC, 1999f). In this case, the strength of the clayey soils at the bottom of the CTB mat was based on the average of the two sets of direct shear tests performed on samples of soils obtained from beneath the CTB at the elevation proposed for founding the mat. The results of these tests are included in Attachments 7 and 8 of Appendix 2A. As indicated in Section 2.6.1.11, $\phi = 21.1°$ and a dynamic cohesion of 1.65 ksf were used in determining resisting forces for the earthquake loading combinations described below.

The results of the sliding stability analysis of the Canister Transfer Building are presented in Table 2 of Calculation 05996.02-G(B)-13 (SWEC, 2000c) and indicate that the factors of safety were >1.1 for all load combinations examined. The lowest factor of safety was 1.27, which applies for the case where 100% of the dynamic earthquake forces acts in the east-west direction and 40% acts in the other two directions. Table 3 of that calculation indicates that if credit is not taken for the increase in strength applicable for the "dynamic" rates of shearing applicable for earthquakes, the factor of safety for this case drops to 0.94. This case is less critical, however, than the case described below, which postulates that the soils at the base of the foundation are cohesionless.

Sliding Stability of the Canister Transfer Building on Cohesionless Soils

The Canister Transfer Building will be founded on clayey soils that have an adequate amount of cohesive strength to resist sliding due to the dynamic forces from the design basis ground motion. As shown in Figures 2.6-21 through 2.6-23, however, some of the soils underlying the building may be cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. Analyses were performed to address the possibility that sliding may occur along a deeper slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

These analyses are presented in Calculation 05996.02-G(B)-13 (SWEC, 2000c). They were performed only for Load Cases IIIA, IIIB, and IIIC, because the resistance to sliding is greatly reduced for frictional materials when the dynamic forces due to the earthquake act upward. As described above, these load cases were defined as follows:

Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.

As shown in Figures 2.6-21 through 2.6-23, the top of the cohesionless layer varies from about 5 ft below the mat to about 9 ft, and it generally is at a depth of about 6 ft below the mat. These analyses included the passive resistance acting on a plane extending from grade down to the top of the cohesionless layer, plus the frictional resistance available along the top of the cohesionless layer. The weight of the clayey soils existing between the top of the cohesionless soils and the bottom of the mat was included in the normal force used to calculate the frictional resistance acting along the top of the cohesionless layer. A review of the cone penetration test results (ConeTec, 1999) obtained within the top 2 ft of the layer of nonplastic silt/silty sand/sandy silt underlying the Canister Transfer Building indicated that ϕ = 38° was a reasonable minimum value for these soils. The factor of safety against sliding along the top of this layer was found to be ≥ 1.1 for all three of the load cases; therefore, there is an adequate factor of safety against sliding along the surface of the cohesionless soils underlying the Canister Transfer Building.

An additional analysis of sliding on cohesionless soils, similar to that described above for sliding of the cask storage pads, was performed to define the upper bound of potential movement that might occur due to the earthquake if the mat was founded directly on cohesionless soils. In this analysis it was postulated that the cohesionless soils extend above the depth of 10 ft and the structure is founded directly on the cohesionless materials. These analyses conservatively assumed that ϕ = 35° and c = 0 for these soils.

The higher value of ϕ used here, compared to that used in the cask storage pad sliding analysis, is based on the fact that the cohesionless soils underlying the Canister Transfer Building area are sandier than those in the pad emplacement area. Further, this higher value is justified by the results of the cone penetration testing, which indicates that the average and median ϕ range from 40 to 44° for the cohesionless soils underlying the Canister Transfer Building. The high values reported in the CPT likely are the manifestation of cementation that was observed in many of the specimens obtained in split-barrel sampling. Possible cementation of these soils is also ignored in this analysis, adding to the conservatism.

Because of the magnitude of the dynamic forces resulting from the soil-structure interaction analyses, the factor of safety against sliding of this building would be less than 1 if it were founded directly on cohesionless soils. For this case, the displacements the building may experience were calculated using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes. Refer to the discussion presented above in Section 2.6.1.12.1 for the storage pads for more information about Newmark's method of analyzing displacements due to earthquakes.

The maximum ground accelerations and velocities of the Canister Transfer Building due to the design basis ground motion, which were developed in Calculation 05996.02-SC-5 (SWEC, 1999f, p. 37), were used in this analysis of displacements. The displacements were calculated, combining the maximum earthquake ground motions in the vertical, north-south (N-S), and east-west (E-W) directions. Because the peak motions of the three components are not expected to occur at the same time, their effects are accounted for by combining 100% of the maximum motion in one direction with 40% of the maximum motions in the other two directions. The minimum resistance to sliding for frictional materials occurs when the vertical forces due to the earthquake act upward. Therefore, the following load cases were used in this analysis:

Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.

The following table presents a summary of the evaluation of sliding of the Canister Transfer Building, assuming it is founded directly on cohesionless soils.

LOAD COMBINATION				DISPLACEMENT
Case IIIA	40% N-S,	-100% Vertical,	40% E-W.	0.8 to 1.2 inches
Case IIIB	40% N-S,	-40% Vertical,	100% E-W.	0.5 inches
Case IIIC	100% N-S,	-40% Vertical,	40% E-W.	0.6 inches

These analyses indicate that there is an adequate factor of safety against sliding along the surface of the soils underlying the building that may be cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. The analysis that postulated that these cohesionless soils exist higher in the profile, such that the building was constructed directly on them, includes several conservative assumptions. Even with this high level of conservatism, the estimated relative displacement of the building ranged from 0.5 inches to 1.2 inches.

Motions of this magnitude, occurring at the depth of the silty sand/sandy silt layer, would likely not even be evident at the ground surface. For the building to slide, a surface of sliding must be established between the horizontal sliding surface in the silty sand/sandy silt layer and through the overlying clayey layer. In the simplified model used to estimate these displacements, the contribution of this surface of sliding through the overlying clayey layer to the dynamic resistance to sliding motion is ignored, as is the passive resistance that would act on the embedded portion of the building foundation and the block of soil that is postulated to be moving with it. It is likely, that should such slippage occur within the cohesionless soils underlying the building, it would minimize the level of the accelerations that would be transmitted through the soil and into the structure. In this manner, the cohesionless soils would act as a built-in base-shear isolation system. Any decrease in these accelerations as a result of this would increase the factor of safety against sliding, which would decrease the estimated displacements as well. Further, since there are no Important to Safety systems that would be severed or otherwise impacted by movements of this small amount as a result of the earthquake, such movements do not adversely affect the performance of the Canister Transfer Building.

2.6.1.12.3 Allowable Bearing Capacity—Other Structures

Other structures at the PFSF include the Administration Building, Operating and Maintenance Building, and Security and Health Physics Building. These structures will be founded on strip and spread footings. The allowable bearing capacity of these footings is limited by shear failure of the soil underlying the footing and by footing settlement.

Bearing capacity analyses were performed for a variety of footing widths and depths for both strip footings and square footings, for vertical loads, and for loads inclined 10 and 20 degrees from the vertical. These analyses were performed using effective-stress strength parameters to investigate long-term conditions, which are applicable for static loads. For these analyses, the allowable bearing pressure was determined using a factor of safety of 3. Bearing capacity analyses were also performed using total-stress strength parameters, which are applicable for earthquake loads. The static analyses yielded the minimum allowable bearing pressures, primarily due to the higher factor of safety required for static loadings.

To limit the expected differential settlements to tolerable values, wall footings of all structures should be designed such that the maximum estimated settlement at the center of the wall along the minimum width of the building is less than or equal to 2 inches. Spread footings supporting column loads spaced approximately 16 ft to 24 ft should be designed such that the maximum estimated settlement at the center of the footing is less than or equal to 1.5 inches. These criteria are based on Table 14.1, "Allowable Settlement," of Lambe & Whitman (1969).

The gross allowable bearing pressure of these footings is presented as a function of the minimum effective footing width and depth in Figure 2.6-10 for strip footings and Figure 2.6-11 for square footings. In these figures, the straight lines represent the allowable bearing pressure that will provide the required factor of safety against a shear failure and the curves represent the bearing pressure that will result in a given amount of settlement. As indicated, the bearing pressure based on shear failure increases with increasing depth (and, typically, increasing width) of footing. Footing settlement increases as the load increases; therefore, for a given bearing pressure, as the width of the footing increases, there comes a point at which the amount of settlement exceeds the allowable settlement. Thus, as the footing width increases beyond this point, the allowable bearing pressure must decrease as shown by the curves in Figures 2.6-10 and 2.6-11, in order to limit the settlement to a tolerable value.

The design curves in these figures are for vertical loads applied at the center of the footings. For inclined or eccentrically applied loads, the allowable bearing pressures must be reduced. For loadings inclined at 10 degrees from the vertical, these allowables must be reduced by 25%, and for loadings inclined at 20 degrees from the vertical, these allowables must be reduced by 50%. Eccentric loads are addressed using the concept of "effective footing width", where the effective width (and length, if appropriate) of the footing is determined as shown in Figures 2.6-10 and 2.6-11.

2.6.2 Vibratory Ground Motion

The PFSF site is situated near the eastern margin of the Basin and Range province in an area known as the Great Basin. It has long been recognized that the pattern of north-south trending ranges and valleys in the Basin and Range is the result of periodic movement on normal faults that border the ranges on one or both sides. This activity is believed to be related to east-west horizontal extension starting in the late Cenozoic (Zoback and Zoback, 1989) and continues today, as evidenced by historic seismicity patterns, ground surface ruptures associated with infrequent, large magnitude, historic seismic events (6.5 M to 7.5 M), and deformation of late Quaternary and Holocene sediments across range-bounding faults.

The eastern boundary of the Basin and Range with the Middle Rocky Mountains province is commonly placed along the Wasatch Front, the north-south trending and west-facing escarpment that follows the Wasatch fault zone. This boundary is much less distinct than it appears physiographically, however. A transition zone up to 60 miles wide occurs east of the fault zone, in which block faulting overprints compressional features of the Sevier orogeny. Historic seismicity is actually higher east of the Wasatch fault than along it and geophysical data indicate the crustal boundary between the provinces occurs here as well (Smith, 1978). When examined on a regional scale, this belt of seismicity can be seen to be part of a larger zone that extends in a curvilinear pattern from northern Arizona and southern Nevada to northwestern Montana (Figure 2.6-12). This zone was first recognized in 1970 and is known as the Intermountain Seismic Belt (ISB) (Smith and Sbar, 1970; Sbar and Barazangi, 1970). Since that time, numerous investigators have discussed the origin and history of the ISB and have attempted to define the seismicity in a plate tectonic setting. Notable among these are the following: Smith and Sbar (1974), Anderson (1989), Stickney and Bartholomew (1987), Smith (1978), Smith et al. (1989), and Smith and Arabasz (1991).

The Skull Valley PFSF is interpreted to lie within the ISB near its western boundary (Arabasz et al., 1987) although it should be noted the boundary is somewhat arbitrary because of the diffuse, low level of seismic activity in this area. At least 16 earthquakes of magnitude 6.0 or greater have occurred in the ISB since settlement of the area began in the late 1840s (Figure 2.6-12). Ground surface faulting has been documented for three of these events: 1959 Hebgen Lake, MT (M_s 7.5); 1983 Borah Peak, ID (M_s 7.3); and 1934 Hansel Valley, UT (M_s 6.6). Surface faulting has also occurred elsewhere in the Basin and Range, in central and western Nevada and eastern California (Slemmons, 1980). The largest of these were the 1915 Pleasant Valley, NV (7.75 magnitude) and the 1872 Owens Valley, CA (8.0 magnitude) events. Arabasz et al. (1987) discuss these events in relation to determining a maximum size for Wasatch Front earthquakes. They concur with studies by Youngs et al. (1987) that the maximum probable event is M_s 7.5 and could have up to 6 meters of vertical displacement. (For an explanation of the various magnitude designations, see Stover and Coffman, 1993, page 2-3.)

Other studies, summarized by Arabasz et al. (1987), indicate there is a threshold magnitude value below which surface faulting is not likely in the Basin and Range. This value is approximately magnitude 6.0 to 6.5. More recent studies also suggest an estimated maximum magnitude of M_L about 6.5 (Arabasz et al, 1992; dePolo, 1994). This value represents the hypothetical maximum "background" or "random" earthquake for this area, one of several seismic sources evaluated to determine peak ground accelerations at the PFSF site. Geomatrix Consultants, Inc. (1999a) consider the maximum magnitude for the "random" event to be between M 5.5 and 6.5, with a mean value of 6.0.

Probabilistic analysis of capable faults and seismic zones in the region is summarized in Section 2.6.2.3 and detailed in Geomatrix Consultants, Inc. (1999a). Peak acceleration levels of 0.53g for horizontal ground motion and 0.53g for the vertical ground motion were determined as the design bases of the PFSF for a 2,000-yr return period (Geomatrix Consultants, Inc, 1999b).

2.6.2.1 Engineering Properties of Materials for Seismic Wave Propagation and Soil-Structure Interaction Analyses

Dynamic soil properties were developed for the subsurface soils at the site in Geomatrix Consultants, Inc (1999d), based on the geotechnical and geophysical investigations that were performed in 1996 and 1998. Refer to Section 2.6.1.5 for additional details about these investigations and to Section 2.6.2.1 for a description of the general stratigraphy. The dynamic soil properties include profile layering, low-strain shear and compression wave velocities, Poisson's ratios, and unit weights. In accordance with US NRC Standard Review Plan, Chapter 3.7, which stipulates that SSI analyses be performed using a range of soil properties, three different sets of shear and compression wave velocity profiles were developed. The best-estimate velocity profile and the high and low velocity profiles are tabulated in Table 2.6-1.

One-dimensional site response analyses were performed using the three different velocity profiles presented in Table 2.6-1 to determine the response based on the best-estimate velocities and the high and low velocities. Figures 2.6-13 and 2.6-14 present the strain-compatible shear-wave velocity and damping ratio profiles for these three cases.

Based on the strain-compatible profiles obtained from the one-dimensional site response analyses, idealized horizontally layered soil profiles were developed for use in the soil-structure interaction analyses based on the SASSI continuum model. The dynamic properties for these idealized layers are presented in Table 2.6-2, and the details of this idealization are presented in Geomatrix Consultants, Inc (1999c).

The equivalent, single-layer shear modulus, Young's modulus, damping ratio, and unit weight of the soil were computed as a weighted average of the values within 30 ft below the surface (the minimum width of the cask storage pads). The weighting factors were assumed to decrease linearly with increasing depth. These equivalent dynamic soil parameters were computed for a rectangular foundation of 30 ft by 64 ft in accordance with Table 3.1 of Newmark and Rosenblueth (1971) for vertical, horizontal, and rocking modes. The resulting parameters are presented in Table 2.6-3.

Refer to Section 2.6.1.11 for discussion of the static and dynamic engineering properties of the soils underlying the site.

2.6.2.2 Earthquake History

The historic record of earthquakes in Utah began in 1850 with the publication of the region's first newspapers in Salt Lake City. Prior to mid-1962 when a scattered, state-wide network of seismographic stations became operational, most records were based upon felt reports. A few larger events were recorded instrumentally at regional stations beginning in the 1950's, including seismograph stations at Salt Lake City and Logan since 1955. Since 1974, a network of modern stations (presently > 85 stations) has provided data to the University of Utah's Seismograph Station (Arabasz et al., 1980). Coverage in the PFSF site area has been provided since 1968 by a station at Dugway, about 14 miles to the south; at Fish Springs, about 50 miles southwest; and on Stansbury Island, about 30 miles north-northeast. Arabasz et al. (1980) estimated the historical catalog for the Wasatch Front region to be complete for Modified Mercalli (MM) intensity greater than VIII since 1850; greater than VII since 1880; greater than VI since 1940; and greater than V since 1950. They judged that instrumental monitoring has provided a complete record down to magnitude (M_L) 2.3 since mid-1962.

Arabasz et al. (1987) provide a comprehensive evaluation of the University of Utah earthquake data base with particular application to an area at the north end of the Cedar Mountains, west of Skull Valley. They conclude that the threshold of earthquake detection is M_L approximately 2.0 or less in an area that includes the PFSF site.

Figure 2.6-15 is a map of all earthquakes within 160 km (100 miles) of the PFSF site of magnitude 3.0 or greater from the University of Utah Seismograph Station catalog. Table 2.6-4 is a chronological listing and description of those events. Only one earthquake greater than magnitude 3.0 has been reported within 50 km of the PFSF site. This event occurred on August 11, 1915 at an assumed location north of Deseret Peak in the Stansbury Mountains. It was reported at Iosepa, a settlement on the western foothill of the Stansbury Mountains. The University of Utah catalog indicates a magnitude 4.3, based on conversion of MM intensity V from the felt report (Arabasz et al., 1987). Stover et al. (1986) list an intensity VI for this event. However, Stover and Coffman (1993) do not list this event in their catalog, which has a threshold magnitude of 4.5. The earthquake was not reported in Tooele, less than 20 miles from Iosepa (Everitt and Kaliser, 1980), nor in Salt Lake City, about 43 miles to the east (Arabasz et al., 1987).

The largest historic earthquakes to occur within 160 km (100 mi.) of the PFSF site occurred in the Hansel Valley at the northern end of Great Salt Lake. A magnitude 6.6 earthquake occurred on March 12, 1934 and produced the only surface offset associated with an historic earthquake in Utah. The event occurred beneath an alluvium-filled valley and resulted in 50 cm of vertical ground surface displacement in a zone 12 km long. Some lateral displacement may also have occurred. Liquefaction and land subsidence occurred locally (Smith, 1978). Slight damage was reported in Grantsville and Tooele with MM intensity V experienced at Tooele (Everitt and Kaliser, 1980). Oaks (1987) reports MM intensity VIII in Salt Lake City caused buildings to sway and a 2-ton clock mechanism fell from the tower of the Salt Lake County Building. Chimneys were toppled and structures were shifted on their foundations. The location of the earthquake is about 90 miles north of the PFSF site and appears to be associated with northerly-trending faults along the base of the Hansel Mountains (dePolo et al., 1989). Four aftershocks occurred within the following 2 months, ranging in size from magnitude 4.8 to 6.1. It is not known what effects, if any, these events had in the PFSF site area. An isoseismal map indicates the PFSF site would have been subject to MM intensity V effects from the original event (Stover and Coffman, 1993).

The Hansel Valley was the site of a prior moderate event magnitude 6.3 on October 6, 1909. Everitt and Kaliser (1980) indicate an MM intensity VII in the epicentral area; the event received no mention in the Tooele paper. The Salt Lake City paper indicated some buildings at the Saltair Resort on the southern shore of the Great Salt Lake were knocked out of plumb. Waves reportedly rolled over the boathouse pier and windows were cracked in Salt Lake City.

The closest magnitude 5.0 or greater earthquakes to the PFSF site occurred near Magna, UT, about 42 miles to the northeast. A magnitude 5.0 event on February 22, 1943 and a magnitude 5.2 event on September 5, 1962 were felt locally in Tooele but no damage was reported (Everitt and Kaliser, 1980). Other sources (Coffman and von Hake, 1973; Stover and Coffman, 1993) report cracked plaster and windows in Salt Lake City and damage to chimneys at Magna from both of these events. Wong et al. (1995) speculate this activity is occurring on the "Saltair structure" and estimate a maximum magnitude 6 for this feature.

Another historic earthquake worthy of mention occurred on August 1, 1900 near the towns of Eureka and Goshen. This magnitude 5.7 event damaged chimneys and plaster in the epicentral area and caused a mine shaft nearby to be thrown out of alignment (Stover and Coffman, 1993). The epicenter is about 48 miles southeast of the PFSF site.

There is no evidence of any effects from any historic earthquake in the PFSF site vicinity.

2.6.2.3 Determining the Design Basis Ground Motion

Federal regulations governing the requirements for siting an ISFSI are contained in 10 CFR 72. These regulations require that seismicity at an ISFSI located west of the Rocky Mountain Front, such as the PFSF, be evaluated using the criteria for determining the safe shutdown earthquake at a nuclear power plant (10 CFR 100 Appendix A) in the same area. Vibratory ground motion design bases were determined by using a "deterministic" approach based upon a single set of earthquake sources. The regulations for siting nuclear power plants (10 CFR 100.23) were amended in 1997 in order to recognize the inherent uncertainties in geologic and seismologic parameters that must be addressed in determining the seismic hazard at a nuclear power plant site. One of the ways to address these uncertainties is through a probabilistic seismic hazard analysis (PSHA). In response to the Part 100 changes and anticipated changes to Part 72 (SECY-98-126), a probabilistic seismic hazard assessment has been performed for the PFSF for vibratory ground motions and surface fault displacement. Methodologies used and the results thereof are detailed in Sections 6 and 7 and Appendix F of Geomatrix Consultants, Inc. (1999a). The hazards results are presented as mean hazard curves that incorporate the uncertainty in input data and interpretations. The seismic source model used 16 capable fault sources and 4 seismic source zones within 100 km. Clarification of the PSHA formulation is provided in SAR Appendix 2F.

The NRC staff has recommended a risk-informed graded approach in their proposed changes to 10 CFR 72 when determining the appropriate hazard frequency or return period. It was determined that an appropriate design probability level for the PFSF is 5 x 10^{-4} per year or a 2,000-yr return period (PFS letters of April and August 1999).

2.6.2.3.1 Capable Faults

The historical record of earthquakes does not provide a complete assessment of seismic potential in the Basin and Range province. There is considerable evidence of late Quaternary and Holocene surface faulting throughout the Basin and Range of Utah. Hecker (1993) has compiled all known or suspected Quaternary fault locations in Utah and provides a description and summary of the evidence for each feature. Goter (1990) provides a 1:500,000 scale map of Hecker's faults with historic seismicity plotted as well. A portion of Goter's map is reproduced as Figure 2.6-16. Figure 2.6-15 also includes Quaternary faults from Hecker (1993). Geomatrix Consultants, Inc. (1999a) provides a detailed discussion of capable faults and seismic source zones within 100 km, as shown on their Plate 7 and listed in Table 6-1. As can be seen on these maps, it is evident there are numerous Quaternary age faults within 100 miles (160 km) of the PFSF site.

Seismic sources include all structures that have some potential for causing strong ground motion at the PFSF (≥ magnitude 5). Seismic sources modeled in the probabilistic seismic hazard analysis are of two types: fault-specific sources and seismic source zones. Fault-specific sources include mapped late Quaternary faults. Seismic source zones are areas that have similar geological or seismologic characteristics that are assumed to have uniform earthquake potential. Seismic source zones are used to model the occurrence of seismicity that cannot be attributed to mapped late Quaternary faults.

A total of sixteen fault-specific sources were analyzed and included in the PSHA as well as four separate seismic source zones. Fault sources are listed in Table 6-1, Geomatrix Consultants, Inc. (1999a). The key parameters used to characterize these sources are as follows:

- Total fault length and plan-view geometry
- Probability of activity
- Maximum earthquake magnitude
- Slip rate
- Recurrence

The values for these key parameters and the weighting factors assigned to each parameter for all seismic sources used in the PSHA are given in Table 6-2, Geomatrix Consultants, Inc. (1999a).

Figure 6-12 in Geomatrix Consultants, Inc. (1999a) shows the contributions of the various fault sources to the total hazard for horizontal motion at the Canister Transfer Building (CTB) location. The largest contributors to the hazard are the Stansbury and East-Springline faults. For long period ground motions the contribution due to the Stansbury fault increases due to the potential for larger earthquakes on the Stansbury than on the mid-Valley faults. The contribution of various earthquake magnitude intervals to the mean hazard for horizontal motion at the CTB location is shown on Figure 6-13 (Geomatrix Consultants, Inc., 1999a). It is evident the hazard is dominated by ground motions from nearby M 6 to 7 events, consistent with the proximity of the Stansbury and East-Springline faults to the CTB. Figure 6-20 (Geomatrix Consultants, Inc.,1999a) shows the contributions of the various fault sources to the total hazard for vertical motions. Again, the Stansbury and East-Springline faults are the dominant sources. The effects of using various models of attenuation, fault segmentation, and fault independence are documented in the report.

2.6.2.3.2 Maximum Earthquake

Several estimates have been made of maximum earthquake magnitude on the Stansbury fault. Arabasz et al. (1987), in their evaluation of seismic parameters for the Superconducting Supercollider facility proposed for a location just west of Skull Valley, calculate a maximum magnitude of M_S 7.3. This value is based on a measured maximum displacement on the fault of 12.6 ft (3.86 m) for a single event and regression relationships derived by Youngs et al. (1987).

Helm (1994, 1995) recently studied the Stansbury fault and identified evidence for segmentation of the fault, as mentioned above. Helm calculated a maximum magnitude of M = 7.0 ± 0.28, based on Wells and Coppersmith's (1994) regression and a surface rupture length of 45 km. This length is for the entire Stansbury fault as if both segments ruptured together. If the north segment (20 km) ruptures next, as Helm (1995) suggests is more likely, a moment magnitude 6.6 ± 0.28 event would be generated.

Pechmann and Arabasz (1995) accept Helm's (1995) subdivision of the Stansbury fault and calculate a maximum magnitude (M_w) of 6½ for each segment. They also utilize the empirical relations of Wells and Coppersmith (1994) but their segment lengths are 17 km and 21 km (straightline length).

Wong et al. (1995) estimate a maximum earthquake of M_w = 6¾ for the Stansbury fault, again based on Wells and Coppersmith (1994), but their possible rupture length is 34 km.

Geomatrix Consultants, Inc. (1999a) divided the Stansbury fault into four segments and analyzed five rupture combination scenarios. Based on empirical relationships between magnitude and rupture length, magnitude and rupture area, magnitude and single event displacement, and a relationship between magnitude, rupture length, and slip rate , Geomatrix Consultants, Inc. determined the maximum magnitude distribution for the Stansbury fault is M 6.5 to 7.5 with a mean of 7.0.

Similarly, they also determined mean maximum magnitudes for the recently identified East fault (M 6.5) and the West fault (M 6.4). These values for the individual faults were utilized in the probabilistic seismic hazard assessment of the PFSF site.

2.6.3 Surface Faulting

The site investigations document the presence of capable faults in the immediate PFSF vicinity. In order to determine the potential hazard of coseismic displacement on these faults, a probabilistic fault displacement hazard analysis was also performed and is described in Geomatrix Consultants, Inc.,1999a, Section 7. Fault displacement hazard analysis is based on methodology developed for the Yucca Mountain repository. Three separate categories of faults that appear to underlie the site were evaluated for displacement hazard: faults that appear to displace the Promontory/Bonneville unconformity (Faults D and F), faults that appear to displace the Tertiary/Quaternary unconformity but not the Promontory/Bonneville (Fault C), and, the zone of distributive faulting between the East and West faults.

Two separate approaches were utilized, an "earthquake approach" and a "displacement approach". Figure 7-8 in Geomatrix Consultants, Inc. (1999a) shows the contribution of the various seismic sources to the displacement hazard using the earthquake approach. The East fault dominates the hazard due to the potential for distributive faulting from a large event near the site. Figure 7-9 compares the mean hazard results for both approaches at the three fault locations beneath the site. The earthquake approach produces similar hazard as the displacement approach at Fault C and lower hazards at the other two locations.

As the consequences of failure of the cask storage system due to fault displacement are comparable to those due to ground motions, the probability level of interest for displacement is also judged to be 5 x 10^{-4} per year, or a 2,000-yr return period. At these probability levels, the displacements associated with faulting on Faults C, D, and F were determined to be less than 0.1 cm (Geomatrix Consultants, Inc. 1999a, Figure 7-7).

2.6.4 Stability of Subsurface Materials

2.6.4.1 Geologic Features that Could Affect Foundations

Dolomite or limestone bedrock is believed to underlie the site at depths between 520 to 880 ft. Examination of outcrops in the area indicates no evidence of cavernous or karst conditions in these rocks and there is no history of karst development in the region. The near-desert conditions make the development of karst very unlikely and the great depth to bedrock precludes effects at the ground surface. There is no evidence of any significant soluble mineral deposits in the unconsolidated materials beneath the site to at least a depth of 225 ft, and no record from water wells in the valley indicates the presence of similar material at greater depths. Evaporites associated with the waning stages of Lake Bonneville and the Great Salt Lake were not deposited here as the area remained above the extent of saline stages of these lakes.

There is no history of oil or gas development or subsurface mining in the Skull Valley and little potential for development in the future. There are no injection wells in the area and no evidence of past activities affecting the ground surface. Groundwater is withdrawn at a few scattered locations in the valley bottom for irrigation and stock watering but not to such an extent to cause surface subsidence or ground cracking. The nearest wells of this type are located 2.5 miles northeast of the PFSF and 3 miles southeast. (See Figure 2.5-1).

Bedrock is not exposed at the PFSF site and will not be encountered by excavation or foundations. As a result, problems associated with alteration, deformation, or weathering of bedrock or anomalous in situ stresses are not a consideration for the foundations.

2.6.4.2 Properties of Underlying Materials

Static and dynamic engineering properties of the soils underlying the site are discussed in Sections 2.6.1.6, 2.6.1.11, and 2.6.2.1.

2.6.4.3 Plot Plan

The plot plan is shown in Figure 2.6-2 and discussed in Section 2.6.1.5. Refer to Section 2.6.1.6 for a description of the subsurface profile.

2.6.4.4 Soil and Rock Characteristics

Soil characteristics are described in detail in Sections 2.6.1.6 and 2.6.2.1. No rock will be encountered by excavations or foundations.

2.6.4.5 Excavations and Backfill

Refer to Section 2.6.1.7 for a discussion of excavations and backfill.

2.6.4.6 Groundwater Conditions

Groundwater conditions at and near the PFSF are discussed in Sections 2.5 and 2.6.1.9.

2.6.4.7 Response of Soil and Rock to Dynamic Loading

The dynamic engineering properties of the soils underlying the site are discussed in Section 2.6.2.1.

Dynamic settlements due to the design basis ground motion are not expected to occur at the PFSF site because of the nature of the subsurface materials. Dynamic settlements, as reported in the geotechnical literature, are based on two different mechanisms, depending on whether the soils are above the groundwater table or below the groundwater table. Silver and Seed (1971) developed a technique for estimating dynamic settlements of dry cohesionless sands above the groundwater table. For such soils, the dynamic settlement mechanism is compaction due to soil grain slip, and it is a function of the magnitude of the cyclic shear strain developed due to the earthquake, the applied number of cycles of this shear strain, and the relative density of the soils.

As indicated in Section 2.6.1.9, the groundwater table is about 125 ft deep at the site. The top 30 ft of the profile consists of silt, silty clay, and clayey silt. The median blow count for this material is 14 blows per ft, indicating that it is "stiff". It appears to be weakly cemented, and unconsolidated-undrained triaxial tests on this material indicate that it has an apparent cohesion that is greater than 2,000 psf. Therefore, the technique for estimating dynamic settlements of soils above the groundwater table is not applicable for these materials, since they are not expected to compact as a result of soil grain slip.

In addition, cyclic triaxial tests were conducted on undisturbed thin-walled tube samples of the soils obtained from the upper ~25-ft thick layer at the site to assess the potential that they might collapse due to shaking caused by the design basis ground motion. These test results are included in Attachment 6 of Appendix 2A.

Five tests were performed on samples from borings in the Canister Transfer Building area. Three of the samples were from the 6 to 10-ft depth range, and the other two were from the 20 to 25-ft depth range. These samples were tested at their natural water content in a partially saturated state. The shallower samples were highly plastic and had void ratios of 1.90, 2.04, and 2.22. The two deeper samples were moderately plastic and had void ratios of 1.26 and 1.55.

Under a confining stress of 2.0 ksf, which approximates the final stresses under the storage pads and the Canister Transfer Building in the upper 25 ft layer, an axial cyclic stress of 1.9 ksf was applied at a rate of 1 Hz for at least 500 cycles. This cyclic stress was determined based on the accelerations associated with the PFSF deterministic design basis ground motion (i.e., 0.67g), not the lower accelerations associated with the PSHA 2,000-yr design basis ground motion (i.e., 0.53g); therefore, these results are very conservative.

The range of double-amplitude strains measured during the test was 0.3% to 1.2%, with an average of 0.7%. All of the samples showed little or no increase of cyclic strain with an increase in the number of stress cycles. The axial cyclic displacement appeared to be elastic in nature. These results demonstrate that these soils will not collapse due to shaking caused by earthquakes with peak ground accelerations that exceed those due to the design basis ground motion.

The upper soil layer is underlain by very dense, fine sands that have uncorrected blow counts that commonly exceed 100 blows per ft. This material is underlain by silts that have even higher blow counts. Because of their very dense nature, these materials are not susceptible to settlement due to the dynamic settlement mechanism applicable for soils above the groundwater table; i.e., compaction due to grain slip.

The underlying soils that are below the groundwater table are greater than 125 ft below grade. The penetration resistances of these soils, as measured down to a depth of 226 ft in Boring CTB-1 and as indicated by the P-wave velocities (5,100 ft/sec to 5,900 ft/sec) reported by Geosphere Midwest, Inc. (Appendix 2B), demonstrate that these soils are also very dense. Because of their very dense nature, these materials are not susceptible to dynamic settlements, even though they may be saturated.

The in situ void ratio of 1.9 reported in Section 2.6.1.11 for the upper layer of soils in the subsurface profile was determined based on data obtained in performing the consolidation tests that are presented in Attachment 2 of Appendix 2A. These tests were performed on samples of the clayey silt. The void ratio of the nonplastic silts was not determined, but based on the standard penetration test (SPT) N-values of the soils, these nonplastic silts would not be characterized as loose.

A review of test results indicated that nonplastic silts were observed in the split-spoon samples obtained above and below Sample U2 in Boring A-2. Therefore, this Shelby tube was opened to see if it contained nonplastic silts that could be tested to determine the void ratio. However, as indicated by the Atterberg limits test results shown on Table 1 of Attachment 3 of Appendix 2A, this tube contained highly plastic clayey silt. Torvane tests performed on these soils demonstrated that the undrained shear strength ranged from 0.65 to 1.8 tons/ft^2, with an average value of 1.25 tons/ft^2, and the void ratio averaged 2.1. These results are consistent with the test results reported in Attachment 2 of Appendix 2A for the clayey silt.

Additional Atterberg limits tests were performed on split-spoon samples obtained in Borings A-2, B-3, C-4, and D-4. These results, shown in Table 1 of Attachment 3 of Appendix 2A, confirmed that Samples S3 in Borings A-2 and C-4, and Sample S3A in Boring D-4 were essentially nonplastic. However, these Atterberg limits indicate that Samples S1 in Borings A-2 and B-3 and Sample S2 in Boring D-4, which were described as nonplastic in the boring logs, are actually slightly or moderately plastic. The descriptions on the boring logs were revised to reflect these laboratory results, as well as those included in Attachments 4 through 7 of Appendix 2A.

A review of the sample descriptions included in the boring logs indicates that only two samples of nonplastic silt are characterized as "loose". These two samples, Samples S-1 in Borings AR-2 and AR-3, were both obtained at the ground surface along the access road. Soils at the ground surface are not of interest since they will be removed during construction. All other nonplastic silt samples for which density is included in the description are characterized as being dense, very dense, or compact.

The following discussion applies to the SPT samples obtained in the upper layer of silt, silty clay, and clayey silt in the areas of the site proposed for the cask storage pads, the Canister Transfer Building, and the Security and Health Physics Building. It excludes the samples obtained at the ground surface, which represent soils that will be excavated for construction of the facilities.

The borings in the vicinity of the proposed locations of the cask storage pads, the Canister Transfer Building, and the Security and Health Physics Building (Borings A-1 through A-4, B-1 through B-4, C-1 through C-4, D-1 through D-4, E-3, and E-4) indicate that the upper layer (~30 ft) consists mostly of soils with some plasticity, especially in the cask storage pad area. The average thickness of nonplastic soils in these borings is ~10 ft. Borings A-2 through A-4, B-1 through B-3, C-1 through C-3, and D-3 have less than or equal to 10 ft of nonplastic soils. Borings A-1 in the northwest, D-1 and D-2 in the northeast, and B-4, C-4, D-4, and E-4 along the south have ~20 ft of nonplastic soils. Note that these nonplastic soils often include occasional thin layers of clay or slightly plastic silt, which will minimize the potential for dynamically induced settlement.

A total of 64 SPT samples of silt (ML) were obtained. Of these, 31 were nonplastic and 33 exhibited some plasticity, ranging from slightly plastic to highly plastic. The N-values for the nonplastic silts in this layer ranged from 11 blows/ft to 40 blows/ft. The median N-value was 18 blows/ft, and the average was 20 blows/ft. This median N-value corresponds to a corrected blow count, N_1, of ~23 blows/ft, based on the relationship between penetration resistance and relative density developed by Gibbs and Holtz (1957) for granular soils.

If the nonplastic silts were cohesionless, they would behave more like fine sands rather than cohesive soils, and based on their N-values, would be classified as very dense rather than loose. Figure 7.5 of Lambe and Whitman (1969) presents the relationship between penetration resistance and relative density developed by Gibbs and Holtz (1957) for granular soils. Using this relationship to estimate the relative density of the non-plastic silts is very conservative, since a decrease in mean grain size tends to cause a decrease in SPT N-value for the same relative density, and the nonplastic silts at the site have a much smaller mean grain-size than the sand and fine sand used by Gibbs and Holtz. Using the 10 psi curve in this figure, or slightly below it, which is the approximate overburden stress for the mid-depth of this layer, fine sands having the median blow count of the nonplastic silts in this layer would be characterized as "very dense", not "loose".

The dynamic settlements of the nonplastic silts in this layer were estimated based on the method presented in Tokimatsu and Seed (1987). As they indicate, for soils above the groundwater table, dynamic settlements are calculated based on procedures originally developed by Silver and Seed (1971), and the effects of multidirectional shaking are estimated based on studies reported by Pyke, Seed, and Chan (1975). The dynamic settlement mechanism is compaction due to grain slip, and it is a function of the magnitude of the cyclic shear strain developed due to the earthquake, the applied number of cycles of this shear strain, and the relative density of the soils.

Figure 13 of Tokimatsu and Seed (1987) presents the relationship between volumetric strain due to compaction, cyclic shear strain, and corrected penetration resistance (N_1) of dry sands for 15 equivalent uniform strain cycles. The cyclic shear strain is estimated based on the average cyclic shear stress due to shaking caused by the design basis ground motion and the shear modulus of the soil. Figure 13 is used to estimate the volumetric strain due to compaction for 15 equivalent uniform strain cycles. Table 4 of Tokimatsu and Seed (1987) is then used to adjust for differences in the number of representative cycles of applied shear stress due to the design basis ground motion (~12 for Magnitude 7) and the 15 cycles used in Tokimatsu and Seed's studies. The dynamic settlement is calculated as the volumetric strain multiplied by the thickness of the nonplastic silts in the layer. Multidirectional effects of the earthquake are addressed by multiplying this result by 2, based on studies reported by Pyke, Seed, and Chan (1975).

The average cyclic shear stress developed in the field due to earthquake shaking is calculated as:

$$\tau_{avg} = 0.65 \bullet a_{max} \bullet \sigma_v \bullet r_d/g = 442 \text{ psf},$$

where: a_{max} = 0.53 g for the design basis ground motion

$\sigma_v = \gamma_{total} \bullet z$ above the groundwater table

γ_{total} = 90 pcf

z = depth below grade

r_d = stress reduction factor, which varies from 1.0 at z=0 to 0.9 at z=30'.

An iterative technique is used to determine the cyclic strain in the field due to the earthquake, γ_{field}. For an assumed value of the cyclic strain, G is calculated as $G_{max} \bullet G / G_{max}$, where G / G_{max} for the nonplastic silt is estimated using the curve for PI=0 presented in Figure 6 of Vucetic and Dobry (1991). G_{max} equals ~1,800 ksf, based on V_s ~ 800 fps and γ_{total} ~ 90 pcf, as indicated in Table 2.6-1 for the upper 25 to 30-ft layer. The following table presents the results of these iterations.

Determination of Cyclic Shear Strain Due to the Design Basis Ground Motion

Iteration No.	$\gamma_{assumed}$ x10^{-4} in./in.	G / G_{max}	G ksf	γ_{field} x10^{-4} in./in.	$\Delta\gamma$ %
1	5.0	0.38	680	6.5	30.
2	7.5	0.30	537	8.2	9.8
3	10.0	0.25	447	9.9	-1.2

The cyclic strain in the field, γ_{field}, is calculated as τ_{avg} / G. Note, it is approximately equal to the assumed cyclic strain for Iteration No. 3; therefore, additional iterations are not required, and γ_{field} is ~10 x 10^{-4} in./in., or 0.10%.

The volumetric strain due to compaction from 15 cycles is estimated as a function of this cyclic shear strain and N_1 of ~23 blows/ft, based on Figure 13 of Tokimatsu & Seed (1987). This results in a volumetric strain, $\varepsilon_{c,N=15}$, of 0.078%.

The design basis ground motion is magnitude 7 (Section 2.6.2.3). Table 4 of Tokimatsu & Seed (1987) indicates this corresponds to ~12 cycles of loading and that the volumetric strain ratio, $\varepsilon_{c,N=12} / \varepsilon_{c,N=15}$, should be ~0.9. Therefore, the volumetric strain corresponding to the design basis ground motion is $\varepsilon_{c,N=12}$, which is 0.9 x 0.078%, or 0.07%.

$$\varepsilon_c = \frac{\Delta\rho_{dyn}}{\Delta H}$$

where $\Delta\rho_{dyn}$ is the dynamic settlement of the layer, and ΔH is the thickness of the layer.

The thickness of the nonplastic silts in the upper layer is conservatively estimated to be 20 ft, based on the discussion presented above. Therefore, for unidirectional shaking,

$\Delta\rho_{dyn,1}$ = 0.17 inches = 20 ft x 12 in./ft x $\varepsilon_{c,N=12}$ / 100%.

The dynamic settlement is multiplied by 2 to account for multidirectional shaking due to the earthquake. This results in an estimated dynamic settlement of the nonplastic silts in the upper layer of 0.34 inches.

Examination of these soils, which are deposits from ancient Lake Bonneville, indicates the presence of numerous tiny shells (Ostracodes). Considerable void space was present under some of these shells, and it is believed that these voids are contributing to the high, in situ void ratio measured for the clayey silt.

Calcium carbonate is present in these soils, as evidenced by a vigorous reaction upon application of hydrochloric acid to these soils. Therefore, these soils are believed to be cemented, the result of carbonate cement bonding of the silt and clay-size particles, imparting cohesion to these soils.

The void ratio of 1.9 reported in Section 2.6.1.11 was determined on samples of the clayey silts from the upper layer, not the nonplastic silts. As evidenced by the SPT data, these nonplastic silts are not loose. The dense nature of these soils, which is most likely the result of carbonate cement bonding of the silt particles, minimizes the potential for dynamically induced settlements due to the design basis ground motion. Ignoring this cementing, the total dynamic settlement is conservatively estimated to be less than ½ of an inch.

This estimated dynamic settlement was determined based on the thickness of nonplastic silts in areas where the nonplastic silts are thickest, not on an average or median thickness. This conservatively overestimates the settlement. In addition, it conservatively neglects the fact that these nonplastic silts are stratified with layers of clay and clayey silt, which will minimize the potential for dynamically induced settlements. Thus, this estimated dynamic settlement is very conservative.

Dynamic settlements will be much less than this over most of the cask storage pad area, since most of the soils in this area are not nonplastic. Rather, these soils are sufficiently stiff and cohesive that they will not experience dynamic compaction due to the shaking caused by the design basis ground motion.

Dynamic settlements of this magnitude are not expected to adversely affect the performance of the facilities.

2.6.4.8 Liquefaction Potential

The soils underlying the proposed PFSF site are not susceptible to liquefaction as a result of the design basis ground motion because they are only partially saturated from grade down to the groundwater level at a depth of 125 ft. The upper ~30-ft thick layer of soils are typically cohesive or cemented and, being essentially dry or only partially saturated, are not subject to liquefaction. The soils from that depth down to the groundwater table at a depth of 125 ft are similarly only partially saturated and they are very dense. The standard penetration test N-values for these soils typically exceed 100 blows per ft, and they increase with depth. The presence of this greater than 90-ft thick, very dense layer overlying the saturated soils is expected to preclude any surface manifestation of liquefaction (e.g., sand boils) of the saturated soils below the groundwater table, if it were possible for them to liquefy. Below the groundwater table, liquefaction is considered unlikely, however, because the density of the soils encountered in the borings increases with depth, as evidenced by the SPT N-values down to a depth of 226 ft in Boring CTB-1 and the high P-wave velocities (5,100 ft/sec to 5,900 ft/sec) measured for the soils below the groundwater table, reported by Geosphere Midwest, Inc. (Appendix 2B).

2.6.4.9 Design Basis Ground Motion

The design basis ground motion was determined by a probabilistic seismic hazard analysis and is defined as having a peak horizontal ground acceleration of 0.53g and a peak vertical ground acceleration of 0.53g. The development of the design basis ground motion is described in Geomatrix Consultants, Inc. (1999a and 1999b). The site specific response spectra are presented in Table 1 and Figure 5 of Geomatrix Consultants, Inc. (1999b).

2.6.4.10 Static Analyses

Refer to Section 2.6.1.12 for a detailed discussion of static analyses in the stability of foundations for structures.

2.6.4.11 Techniques to Improve Subsurface Conditions

Discussions presented in Section 2.6.1.12, above, indicate that the soils underlying the eolian silt layer at the surface of the PFSF site are suitable for support of the proposed structures; therefore, no special construction techniques are required for improving the subsurface conditions below the eolian silt. The eolian silt, in its *in situ* loose state, is not suitable for founding the structures at the site. The Canister Transfer Building will be founded on the silty clay/clayey silt layer beneath the eolian silt. It was originally intended that the cask storage pads also would be founded on the silty clay/clayey silt layer. However, instead of excavating the eolian silt from the pad emplacement area and replacing it with suitable structural fill, it will be mixed with sufficient portland cement and water and compacted to form a strong soil-cement subgrade to support the cask storage pads. The required engineering characteristics of the soil cement can be easily engineered during detailed design to meet the necessary strength requirements.

The surficial layer of eolian silt, existing across the entire site as shown in the pad emplacement area foundation profiles (Figure 2.6-5, Sheets 1 through 14), is a major factor in the earthwork required for construction of the facility. This layer consists of a nonplastic to slightly plastic silt, and it has an average thickness of approximately 3 feet across the pad emplacement area. This layer was expected to be removed prior to construction of the storage pads. However, based on evaluation of the earthwork associated with site grading requirements for flood protection and the environmental impacts of truck trips required to import fill to replace this material, PFS will stabilize this soil with cement and use it as base material beneath the storage pads and adjacent driveways.

Section 2.6.1.12 indicates that there is ample margin in the factor of safety against a bearing capacity failure of the silty clay/clayey silt underlying the site and that the settlements are acceptable for these structures. They indicate that the critical design factor with respect to stability of these structures is the resistance to sliding due to loadings from the design basis ground motion. As discussed in that section, the silty clay/clayey silt layer has sufficient strength to resist these dynamic loadings; therefore, adequate sliding resistance can be provided by constructing the structures directly on the silty clay/clayey silt layer. The soil cement will be designed to provide shear strength that exceeds the strength of the silty clay/clayey silt. Therefore, the resistance to sliding due to loadings from the design basis ground motion will be enhanced by constructing the cask storage pads on a properly designed and installed soil-cement subgrade.

Using soil cement to stabilize the eolian silt will reduce the amount of spoil materials generated, create a stable and level base for pad construction, and substantially improve the sliding resistance of the storage pads. The soil cement will be placed above the *in situ* silty clay/clayey silt layer and will be designed to improve the strength of the eolian silt so that it will be stronger than the clayey soils that were originally intended for use as the founding medium for the pads. The soil cement will also be used to replace the compacted structural fill that the original plan included between the rows of pads. This continuous layer of soil cement, existing under and between the pads, will spread the loads from the pads beyond the footprint of the pads, resulting in decreased total and differential settlements of the pads. The layer of soil cement above the base of the pads and the bond and friction of the pad foundation with the underlying soil-cement layer will greatly increase the sliding resistance of the pad.

Soil cement has been used extensively in the United States and around the world since the 1940's. It was first used in the United States in 1915 for constructing roads. It also has been used at nuclear power plants in the United States and in South Africa. The largest soil-cement project worldwide involved construction of soil-cement slope protection for a 7,000-acre cooling-water reservoir at the South Texas Nuclear Power Plant near Houston, TX. Soil cement also was used to replace an ~18-ft thick layer of potentially liquefiable sandy soils under the foundations of two 900-MW nuclear power plants in Koeberg, South Africa (Dupas and Pecker, 1979).

The strength of soils can be improved markedly by the addition of cement. The eolian silt at the site is similar to the soils identified as Soil A-4 in Nussbaum and Colley (1971), Soils 7 and 8 in Balmer (1958), and Soil 4 in Felt and Abrams (1957). As indicated for Soil A-4 in Table 5 of Nussbaum and Colley (1971), the addition of just 2.5% cement by weight to the silt increased the cohesion from 5 psi (720 psf) to 30 psi (4,320 psf). The cohesion for Soils 7 and 8 also were increased significantly by the addition of low percentages of cement, as shown on Tables VI and VII of Balmer (1958). Figure 10 in Felt and Abrams (1957) illustrates the continued strength increase over time for these soil-cement mixtures. Other examples of soil-cement strength increases over time are presented in Figure 4.3 of ACI (1998), Table 6 of Nussbaum and Colley (1971), and Figures 6 and 7 of Dupas and Pecker (1979). Therefore, the soil cement will be much stronger than the underlying silty clay/clayey silt and the strength will increase with time, providing an improved foundation material. This will provide additional margin against sliding compared to the original plan to construct the pads directly on the silty clay/clayey silt layer..

As shown in the section titled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement" in Section 2.6.1.12.1 above, the shear resistance required at the base of the pads can be provided easily by the passive resistance of the soil cement acting against the vertical side of the foundation and by bond between the pad foundation and soil-cement contact and the cohesive strength of the soil cement. Shear resistance will be transferred through the approximately 3-ft thick soil-cement layer and into the underlying silty clay/clayey silt subgrade. Additional resistance will be provided by the continuous layer of soil cement under and between the pads; therefore, shear resistance requirements within the silty clay/clayey silt layer will be less with the soil-cement layer compared to the original plan to construct the pads directly on the silty clay/clayey silt without the proposed soil-cement layer.

DeGroot (1976) indicates that this bond strength can be easily obtained between layers of soil cement. He performed nearly 300 laboratory direct shear tests to determine the effect of numerous variables on the bond between layers of soil cement. These variables included the length of time between placement of successive layers of soil cement, the frequency of watering while curing soil cement, the surface moisture condition prior to construction of the next lift, the surface texture prior to construction of the next lift, and various surface treatments and additives

His results demonstrate that, with the exception of treating the surface of the lifts with asphalt emulsion, asphalt cutback, and chlorinated rubber compounds, the bond strength always exceeds this required cohesion value. The minimum bond strength he reports, other than for the asphalt and chlorinated rubber surface treatments identified above, is 8.7 psi. This value applied for two tests that were performed on samples that had time delays of 24 hours and did not have a cement surface treatment along the lift line. He reports that nearly all of the specimens that used a cement surface treatment broke along planes other than along the lift lines, indicating that the bond between the layers of soil cement was stronger than the remainder of the specimens. Excluding the specimens that had 24-hr delays between lift placements and which did not use the cement surface treatment, the minimum bond strength was 10.7 psi and there were only two others that had bond strengths that were less than 20 psi. Even these minimum values for the group of specimens that did not use a cement surface treatment exceed the required cohesive strength, and all of the rest were much greater, generally more than an order of magnitude greater than the 6.6 psi required to obtain an adequate factor of safety against sliding.

DeGroot reached the following conclusions:

1. Increasing the time delay between lifts decreases bond.
2. High frequency of watering the lift line decreases the bond.

3. Moist curing conditions between lift placements increases the bond.
4. Removing the smooth compaction plane increases the bond.
5. Set retardants decreased the bond at 4-hr time delay.
6. Asphalt and chlorinated rubber curing compounds decreased the bond.
7. Small amounts of cement placed on the lift line bonded the layers together.

DeGroot (1976) noted that increasing the time delay between placement of subsequent lifts decreases the bond strength. The nature of construction of soil cement is such that there will be occasions when the time delay will be greater than the time required for the soil cement to set. This will clearly be the case for construction of the concrete storage pads on top of the soil-cement surface, because it will take some period of time to form the pad, build the steel reinforcement, and pour the concrete. He noted that several techniques can be used to enhance the bond between these lifts to overcome this decrease in bond due to time delay. In these cases, more than sufficient bond can be obtained between layers of soil cement and between the set soil-cement surface and the underside of the cask storage pads by simply using a cement surface treatment.

DeGroot's direct shear test results demonstrate that the specimens having a cement surface treatment all had bond strengths that ranged from 47.7 psi to 198.5 psi, with the average bond strength of 132.5 psi. Even the minimum value of this range is nearly an order of magnitude greater than the cohesion required to obtain a factor of safety against sliding of 1.1. Therefore, when required, due to unavoidable time delays, the techniques DeGroot describes for enhancing bond strength will be used between the top of the soil cement and succeeding lifts or the concrete cask storage pads, to assure that the bond at the interfaces are greater than the minimum required value. These techniques will include roughening and cleaning the surface of the underlying soil cement, proper moisture conditioning, and using a cement surface treatment.

PFS has discussed the change to use of soil cement beneath the storage pads with the project consultants who have analyses in-place that are based on the storage pads resting on the silty clay/clayey silt. The consultants contacted were Geomatrix (development of seismic criteria and soil dynamic properties), Holtec International (cask stability analysis), and International Civil engineering Consultants (pad design). Each has indicated their analyses would not be adversely affected by this proposed change.

The design, placement, testing, and performance of soil cement is a well-established technology. The "State-of-the-Art Report on Soil Cement" (ACI, 1998) provides information about soil cement, including applications, materials, properties, mix proportioning, design, construction, and quality-control inspection and testing techniques. PFS will develop site-specific procedures to implement the recommendations presented in ACI (1998) regarding mix proportioning, testing, construction, and quality control. The following describes the processes that will be used to develop a proper soil-cement mix design and establish adequate sliding resistance at each material interface in the storage pad and soil system:

- Soil-Cement Mix and Procedure Development – The sliding forces due to the design basis ground motion will be resisted by bond between the base and sides of the foundation and the soil cement and by passive resistance of the soil cement acting against the vertical side of the foundation. The soil-cement mix will be designed and constructed to exceed the minimum shear resistance requirements. During the soil-cement design phase, direct shear testing will be conducted along manufactured soil-cement lift contacts and concrete contacts that represent anticipated field conditions. The direct shear testing, along with other standard soil-cement testing, will be used to confirm that adequate shear resistance and other strength requirements will be provided by the final soil-cement mix design. Procedures required for placement and treatment of the soil cement, lift surfaces, and foundation contact will be established in accordance with the recommendations of ACI (1998) during the mix design and testing process. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during this detailed design phase of the project.

- Soil-Cement Lift and Concrete Interface – The soil cement will be constructed in lifts approximately 6-in thick (compacted thickness) as described in ACI (1998). Construction techniques will be used to ensure that the interface between the soil-cement layers will be adequately bonded to transmit shear stresses. Techniques described in Section 6.2.2.5 of ACI (1998) will include, but will not be limited to: minimizing the time between placement of successive layers of soil cement, moisture conditioning required for proper curing of the soil cement, producing a roughened surface on the soil cement prior to placement of additional lifts or concrete foundations, and using a dry cement or cement slurry to enhance the bonding of concrete or new soil cement layers to underlying layers that have already set. In addition to conventional quality control testing performed for soil-cement projects, direct shear testing will be performed on representative samples obtained from placed lift contacts to confirm design requirements are obtained. Sacrificial soil-cement lifts may be used to protect the soil-cement subgrade in the pad foundation areas.

- Soil Cement and *In Situ* Clay Interface – The soil cement and *in situ* clay interface will be constructed such that a good bond will be established between the two materials. Construction techniques will be utilized that will ensure that the integrity of the upper surface of the clay is maintained and that a good interface bond between the two materials is obtained. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during the detailed design phase of the project.

An additional benefit of incorporating the soil cement into the design is that it will minimize the environmental impacts of constructing the facility. Using on-site materials to construct the soil cement, rather than excavating and spoiling those materials, will reduce environmental impacts of the project. In addition, replacement of some of the structural fill layer between the rows of pads with soil cement, as shown in SAR Figure 4.2-7, will result in reduced trucking requirements associated with transporting those materials to the site.

2.6.4.12 Criteria and Design Methods

The allowable bearing capacity of footings is limited by shear failure of the underlying soil and by footing settlement. The minimum factor of safety against a bearing capacity failure from static loads (dead load plus maximum live loads) is 3.0 and from static loads plus loads due to extreme environmental conditions, such as design basis ground motion, is 1.1. Allowable settlements are determined based on Table 14.1, "Allowable Settlement," of Lambe & Whitman (1969) and assume that the differential settlement will be 3/4 of the maximum settlement. Section 2.6.1.12 provides more details.

In order to comply with the requirements of NUREG-75/087, Section 3.8.5, "Foundations," Section II.5, "Structural Acceptance Criteria," the recommended minimum factor of safety against overturning or sliding failure from static loads (dead load plus maximum live loads) is 1.5 and from static loads plus loads due to extreme environmental conditions, such as design basis ground motion, is 1.1. Where the factor of safety against sliding is less than 1 due to the design basis ground motion, the displacements the structure may experience are calculated using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes. The magnitude of these displacements are evaluated to assess the impact on the performance of the structure. See Section 2.6.1.12 for details about these analyses.

2.6.5 Slope Stability

There are no slopes close enough to the proposed Important to Safety facilities that their failure could adversely affect the operation of these facilities.

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TABLE 2.6-6

SUMMARY – ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Static Loads

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\ E\text{-}W}$	β_L $EQ_{H\ N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
IA - Static Undrained Strength	3,716	0	0	0	0	0.0	0.0	13.05	4.35	0.0	0.0	30.0	64.0	1.94	*6.7*
IB - Static Effective Strength	3,716	0	0	0	0	0.0	0.0	28.34	9.44	0.0	0.0	30.0	64.0	1.94	*14.6*
IC - Static Total Strength	3,716	0	0	0	0	0.0	0.0	37.12	12.37	0.0	0.0	30.0	64.0	1.94	*19.2*

c = **2,200** Undrained strength (psf) & ϕ = 0.

c = **1,400** Total stress cohesion (psf)

ϕ = **21.3** Total stress friction angle (deg)

B = **30** Footing width (ft)

L = **64** Footing length (ft)

D_f = **2.7** Depth of footing (ft)

γ = **80** Unit weight of soil (pcf)

γ_{surch} = **100** Unit weight of surcharge (pcf)

FS = **3** Factor of safety for static loads.

ϕ = **30.0** Effective stress friction angle (deg), c = 0.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E\text{-}W}$ or $EQ_{H\ N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\ E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\ N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2e_B$ $L' = L - 2e_L$

$q_{actual} = F_V / (B' \times L')$

TABLE 2.6-7

Summary – Allowable Bearing Capacity of Cask Storage Pads

Based on Inertial Forces Due to Design Basis Ground Motion: PSHA 2,000-Yr Return Period

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\ E\text{-}W}$	β_L $EQ_{H\ N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
II	3,716	1,962	1,962	20,006	20,006	27.8	27.8	14.41	13.09	5.4	5.4	19.2	53.2	3.63	*4.0*
IIIA	1,735	785	785	8,002	8,002	24.3	24.3	16.12	14.65	4.6	4.6	20.8	54.8	1.52	*10.6*
IIIB	2,924	785	1,962	20,006	8,002	33.9	15.0	11.49	10.44	6.8	2.7	16.3	58.5	3.06	*3.8*
IIIC	2,924	1,962	785	8,002	20,006	15.0	33.9	22.15	20.13	2.7	6.8	24.5	50.3	2.37	*9.3*
IVA	5,697	785	785	8,002	8,002	7.8	7.8	27.85	25.31	1.4	1.4	27.2	61.2	3.42	*8.1*
IVB	4,508	785	1,962	20,006	8,002	23.5	9.9	16.32	14.83	4.4	1.8	21.1	60.5	3.53	*4.6*
IVC	4,508	1,962	785	8,002	20,006	9.9	23.5	26.23	23.84	1.8	4.4	26.5	55.1	3.09	*8.5*

c = **1,400** Total stress cohesion (psf)

ϕ = **21.3** Total stress friction angle (deg)

B = **30** Footing width (ft)

L = **64** Footing length (ft)

D_f = **2.7** Depth of footing (ft)

γ = **80** Unit weight of soil (pcf)

γ_{surch} = **100** Unit weight of surcharge (pcf)

FS = **1.1** Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E\text{-}W}$ or $EQ_{H\ N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\ E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\ N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\,e_B$ $L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

TABLE 2.6-8

SUMMARY – ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Maximum Cask Driving Forces Due to Design Basis Ground Motion: PSHA 2,000-Yr Return Period for Loading Case IV: 100% N-S, 100% Vertical (Downward), and 100% E-W

Case IV	F_V	$EQ_{H\,N\text{-}S}$	$EQ_{H\,E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\,E\text{-}W}$	β_L $EQ_{H\,N\text{-}S}$	GROSS		e_B	e_L	EFFECTIVE			FS_{actual}
								q_{ult}	q_{all}			B'	L'	q_{actual}	
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
2 Casks	2,647	768	909	9,874	13,104	19.0	16.2	23.2	21.07	3.73	4.95	22.1	22.5	5.31	*4.4*
4 Casks	4,633	1,265	1,378	13,807	27,290	16.6	15.3	22.4	20.37	2.98	5.89	24.0	36.2	5.32	*4.2*
8 Casks	8,755	2,247	2,311	30,818	34,320	14.8	14.4	21.7	19.76	3.52	3.92	23.0	56.2	6.79	*3.2*

c = **1,400** Total stress cohesion (psf)

ϕ = **21.3** Total stress friction angle (deg)

B = **30** Footing width (ft)

L = **Varies** Footing length (ft)

D_f = **2.7** Depth of footing (ft)

γ = **80** Unit weight of soil (pcf)

γ_{surch} = **100** Unit weight of surcharge (pcf)

FS = **1.1** Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$\Sigma M_{@N\text{-}S} = e_B \times F_V$ $\Sigma M_{@E\text{-}W} = e_L \times F_V$

$B' = B - 2\,e_B$ $L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

TABLE 2.6-9

SUMMARY - ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Based on Static Loads

Case	F_V	$EQ_{H\,N\text{-}S}$	$EQ_{H\,E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\,E\text{-}W}$	β_L $EQ_{H\,N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
IA - Static Undrained Strength	72,988	0	0	0	0	0.0	0.0	13.23	4.41	0.0	0.0	165.0	265.0	1.67	*7.9*
IB - Static Effective Strength	72,988	0	0	0	0	0.0	0.0	135.00	45.00	0.0	0.0	165.0	265.0	1.67	*80.9*
IC - Static Total Strength	72,988	0	0	0	0	0.0	0.0	61.31	20.43	0.0	0.0	165.0	265.0	1.67	*36.7*

c = **2,200** Undrained strength (psf) & ϕ = 0.

ϕ = **30.0** Effective stress friction angle (deg), c = 0.

c = **1,100** Total stress cohesion (psf)

F_V = Vertical load (Static + EQ_V)

ϕ = **21.1** Total stress friction angle (deg)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$

B = **165** Footing width (ft)

$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

L = **265** Footing length (ft)

$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

D_f = **5.0** Depth of footing (ft)

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\quad e_L = \Sigma M_{@E\text{-}W} / F_V$

γ = **90** Unit weight of soil (pcf)

$B' = B - 2\,e_B$ $\quad L' = L - 2\,e_L$

γ_{surch} = **80** Unit weight of surcharge (pcf)

$q_{actual} = F_V / (B' \times L')$

FS = **3** Factor of safety for static loads.

TABLE 2.6-10

SUMMARY - ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Based on Dynamic Loads Due to Design Basis Ground Motion: PSHA 2,000-yr Return Period

Case	F_V	$EQ_{H\,N\text{-}S}$	$EQ_{H\,E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\,E\text{-}W}$	β_L $EQ_{H\,N\text{-}S}$	GROSS		e_B	e_L	EFFECTIVE			FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	q_{ult} ksf	q_{all} ksf	ft	ft	B' ft	L' ft	q_{actual} ksf	
II	72,988	62,040	67,572	2,513,041	1,961,325	42.8	40.4	6.87	6.24	34.4	26.9	96.1	211.3	3.59	*1.9*
IIIA	15,849	24,816	27,029	1,005,216	784,530	59.6	57.4	2.70	2.45	63.4	49.5	38.2	166.0	2.50	*1.1*
IIIB	50,132	24,816	67,572	2,513,041	784,530	53.4	26.3	3.89	3.53	50.1	15.6	64.7	233.7	3.31	*1.2*
IIIC	50,132	62,040	27,029	1,005,216	1,961,325	28.3	51.1	12.58	11.43	20.1	39.1	124.9	186.8	2.15	*5.9*
IVA	130,127	24,816	27,029	1,005,216	784,530	11.7	10.8	26.04	23.67	7.7	6.0	149.6	252.9	3.44	*7.6*
IVB	95,844	24,816	67,572	2,513,041	784,530	35.2	14.5	9.23	8.38	26.2	8.2	112.6	248.6	3.42	*2.7*
IVC	95,844	62,040	27,029	1,005,216	1,961,325	15.7	32.9	19.98	18.16	10.5	20.5	144.0	224.1	2.97	*6.7*

c =	1,100	Total stress cohesion (psf)	F_V = Vertical load (Static + EQ_V)
ϕ =	21.1	Total stress friction angle (deg)	EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$
B =	165	Footing width (ft)	$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).
L =	265	Footing length (ft)	$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).
D_f =	5.0	Depth of footing (ft)	$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\quad e_L = \Sigma M_{@E\text{-}W} / F_V$
γ =	90	Unit weight of soil (pcf)	$B' = B - 2\,e_B$ $\quad L' = L - 2\,e_L$
γ_{surch} =	80	Unit weight of surcharge (pcf)	$q_{actual} = F_V / (B' \times L')$
FS =	1.1	Factor of safety for dynamic loads.	

TABLE 2.6-11

FOUNDATION LOADINGS FOR THE CANISTER TRANSFER BUILDING

Based on Dynamic Loads Due to Design Basis Ground Motion: PSHA 2,000-yr Return Period

								SHEAR X	UPLIFT	SHEAR Z	$\Sigma M_{Base\ @\ El\ 95}$	
JOINT	ELEV ft	MASS X $k\text{-}sec^2/ft$	MASS Y $k\text{-}sec^2/ft$	MASS Z $k\text{-}sec^2/ft$	Ax g	Ay g	Az g	$F_{HN\text{-}S}$ k	$F_{V\ dyn}$ k	$F_{HE\text{-}W}$ k	$M_{@N\text{-}S}$ ft-k	$M_{@E\text{-}W}$ ft-k
1	95	1257.0	1257.0	1257.0	0.805	0.720	0.769	32,583	29,142	31,126	155,628	162,913
2	130	490.7	490.7	490.7	0.864	0.764	0.834	13,652	12,072	13,178	461,218	477,808
3	170	299.2	299.2	157.0	0.939	0.829	0.966	9,047	7,987	4,884	366,264	678,491
4	190	219.8	166.9	219.8	0.955	0.839	1.067	6,759	4,509	7,552	717,417	642,112
5	190	0.0	52.9	0.0	0.000	2.013	0.000	0	3,429	0	0	0
6	170	0.0	0.0	142.2	0.000	0.000	2.366	0	0	10,834	812,515	0
		WEIGHT =	72,988	k			Σ =	62,040	57,139	67,572	2,513,041	1,961,325

Ref: Calc 05996.02-G(B)-13, Rev 2 (SWEC, 2000c)

Based on sliding and uplift forces from p 37 of Calc 05996.02-SC-5, Rev 1 (SWEC, 1999h), which are applicable for "High" Moduli in Calc 05996.02-G(PO18)-2, Rev 0 (Geomatrix, 1999c).

CHAPTER 2

APPENDIX 2A

GEOTECHNICAL DATA

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APPENDIX 2F

Clarification of PSHA Formulation

(6 pages plus Figures C-1, C-2, and C-3)

CLARIFICATION OF PSHA FORMULATION

PSHA Formulation for Ground Motion Hazard

Equation (6-2) of the text, repeated below, is the basic equation used in computing the ground shaking hazard. The hazard is expressed as the frequency of exceeding a specified level of ground motion, $\nu(z)$, where z is the ground motion level. Given a *known* set of models and model parameters for representing the frequency of earthquake occurrence, the randomness of size and location of future earthquakes, and the randomness in the level of ground motion they may produce at the site, $\nu(z)$ is computed by the expression:

$$\nu(z) = \sum_{n} \alpha_n(m^0) \int_{m^0}^{m^u} f(m) \left[\int_{0}^{\infty} f(r|m) \cdot P(Z > z|m,r) \cdot dr \right] \cdot dm \qquad (6\text{-}2)$$

where $\alpha_n(m^0)$ is the frequency of all earthquakes on source n above a minimum magnitude of engineering significance, m^0; $f(m)$ is the probability density of earthquake size between m^0 and a maximum earthquake the source can produce, m^u; $f(r\,|\,m)$ is the probability density function for distance to an earthquake of magnitude m occurring on source n; and $P(Z>z\,|\,m,r)$ is the probability that, given an earthquake of magnitude m at distance r from the site, the peak ground motion will exceed level z.

However, the models and model parameters of Equation (6-2) are not known with certainty. They depend upon the collective set of scientific judgments and data interpretations documented in the PSHA report. These can be represented by a set of parameters Θ. The elements of Θ include all of the parameters of Equation (6-2), together with the specific interpretations that lead to those parameters. The uncertainty in Θ is characterized using the logic trees shown on Figures 6-3 and 6-5 of the PSHA report. Each end branch at the right hand side of the log tree defines a specific set of input parameters, θ_i that can be used to compute the hazard using Equation (6-2). The result is a frequency of exceeding ground motion level z that is conditional on θ_i, $\nu(z|\theta_i)$ and Equation (6-2) can be rewritten as:

$$\nu(z|\theta_i) = \sum_{n} \alpha_n(m^0|\theta_i) \int_{m^0}^{m^u|\theta_i} f(m|\theta_i) \left[\int_{0}^{\infty} f(r|m,\theta_i) \cdot P(Z > z|m,r,\theta_i) \cdot dr \right] \cdot dm \qquad (C\text{-}1)$$

The probability that Θ will take on any particular value θ_i is equal to the joint probability of the set of parameters θ_i being the true parameter values. $P(\Theta = \theta_i)$ is obtained by multiplying the probabilities on all of the branches leading to θ_i:

$$P(\Theta = \theta_i) = \prod_k P(branch_k \mid branch_1 \cdots branch_{k-1}) \tag{C-2}$$

where $P(branch_k \mid branch_1 \ldots branch_{k-1})$ is the probability that a specific branch at node k is the correct branch conditional on all of the branches leading to node k represent the correct path through the logic tree.

As a result of computing the hazard for each end branch of the logic tree, a discrete distribution for $\nu(z \mid \Theta)$ is obtained. The expected or mean value of $\nu(z \mid \Theta)$ is given by:

$$E[\nu(z \mid \Theta)] = \sum_i \nu(z \mid \theta_i) \cdot P(\Theta = \theta_i) \tag{C-3}$$

and the fractiles of the distribution are obtained by ordering the values of $\nu(z|\theta_i)$ and computing the sum of $P(\Theta = \theta_i)$ until the desired fractile levels are reached.

PSHA Formulation for Fault Displacement Hazard

The formulation for probabilistic evaluation of the hazard from fault displacement is analogous to that developed for the hazard from ground shaking. The fault displacement PSHA provides the frequency of exceeding a specified level of displacement, $\nu(d)$, where d is the amount of fault displacement. Equation (7-1) in the PSHA report presents the basic hazard formulation is its simplest terms:

$$\nu(d) = \lambda_{DE} \cdot P(D > d) \tag{7-1}$$

where λ_{DE} is the frequency of displacement events and $P(D>d)$ is the conditional probability that the displacement in a single event will exceed value d. The exact form of Equation (7-1) used in the calculation depends upon whether the *earthquake approach* or the *displacement approach* is being used.

For the earthquake approach, λ_{DE} is given by Equation (7-3) in the PSHA report:

$$\lambda_{DE} = \sum_{j=1}^{n} \lambda_j(\text{Events on source } j) \times P_i(\text{Slip}|\text{Event on source } j) \qquad \textbf{(7-3)}$$

where P_i(Slip|Event on j) is the probability of slip at point i due to an earthquake on source j, given by Equations (7-4) and (7-5) in the PSHA report, and λ_j is the frequency of earthquakes of different sizes and at different locations from Equation (6-2). Thus, using Equations (6-2) and (7-3), Equation (7-1) is recast as:

$$v(d) = \sum_j \alpha_j(m^0) \int_{m^0}^{m^u} f(m) \left[\int_0^{\infty} f(r|m) \cdot P(\text{slip}|m,r,h) \cdot P(D > d|m,r) \cdot dr \right] \cdot dm \qquad \textbf{(C-4)}$$

Because both P_i(Slip|Event on j) and $P(D>d)$ vary with earthquake magnitude and source-to-site distance, they are included within the magnitude and distance integrals. [Note that for ground motion hazard, the analogous probability, P_i(Shaking|Event on j), is equal to 1.0 because it is assumed that every earthquake will produce some level of shaking at a site, though the level may be very small.] As was the case for Equation (6-2), incorporating the uncertainty in the models and parameters leads to the displacement hazard form of Equation (C-1) for the earthquake approach:

$$v(d|\theta_i) = \sum_j \alpha_j(m^0|\theta_i) \int_{m^0}^{m^u|\theta_i} f(m|\theta_i) \left[\int_0^{\infty} f(r|m,\theta_i) \cdot P(\text{slip}|m,r,h,\theta_i) \cdot P(D > d|m,r,\theta_i) \cdot dr \right] \cdot dm \qquad \textbf{(C-5)}$$

where again, θ_i represents a specific set of models and model parameters used to compute the hazard.

For the displacement approach using fault slip rate, the formulation is much simpler, with λ_{DE} given by Equation (7-2) in the PSHA report and $P(D>d)$ dependent on the average displacement per event, $\bar{D}_E$, and the form of the distribution for $D/\bar{D}_E$. Incorporating uncertainty in the models and parameters leads to displacement hazard form of Equation (C-1) for the displacement approach:

$$v(d|\theta_i) = \frac{SR|\theta_i}{\bar{D}_E|\theta_i} + P\left(D > d \middle| \bar{D}_E, \theta_i \right) \qquad \textbf{(C-6)}$$

The mean hazard integrated over the uncertainty in Θ is computed using Equation (C-3).

Probability of Distributed Slip for Earthquake Approach to Fault Displacement Hazard

For the distributed faulting approach, the probability that an earthquake on source j will cause distributed slip on the feature at point i is computed using the logistic regression model of Equation (7-4) in the PSHA report:

$$P_i(\text{Slip} \mid \text{Event on } j) = \frac{e^{f(m,r)}}{1+e^{f(m,r)}} \qquad \textbf{(7-4)}$$

where $f(m,r)$ is given by Equation (7-5) in the PSHA report

$$f(m,r,h,\tau) = 3.27 + (-8.28 + 0.577m + 0.629h) \cdot \ln(r + 4.14) + 0.611\tau \qquad \textbf{(7-5)}$$

in which h is 1.0 if the site lies in the hanging wall of the rupture and 0.0 if the site lies in the foot wall, and τ is a random variate with 0 mean and unit variance that accounts for variability from earthquake to earthquake. When Equation (7-5) is used to compute the probability of distributed slip, the mean value of P_i(Slip|Event on j) is found by integrating over the random effect distribution. Figure C-1 shows the variation in the predicted probability of distributed rupture for a magnitude 6.5 earthquake as the random effect τ is varied from −1.22 to +1.22, corresponding to a $\pm$ 2 standard deviation range for a normal variate which encompasses 95% of the probability mass. Note that the curves shown on Figure C-1 represent a balance between the data with non zero densities of distributed faulting and the larger mass of data with observed zero density of distributed faulting show by the data points at the bottom of the plots.

The general form of Equation (7-5) was developed as part of the seismic hazard assessment for Yucca Mountain (CRWMS M&O, 1998, Appendix H). The relationship preferred by the majority of the experts was:

$$f(m,r,h) = 2.06 + (-4.62 + 0.118m + 0.682h) \cdot \ln(r + 3.32) \qquad \textbf{(C-7)}$$

During application of the displacement hazard methodology in a subsequent project for the Los Alamos National Laboratory (Olig and others, 1998) it was suggested that the distributed faulting data may be more scattered than represented by the form of Equation (C-7) and that a *random effects* model might provide a better fit. Hosmer and Lemeshow (1989, page 141) define a goodness of fit statistic, $\hat{C}$, for logistic regression in the form of a Pearson χ^2 statistic for a table of observed and predicted frequencies. Using this approach, Olig and others (1998) found a goodness of fit statistic, $\hat{C}$, for Equation (C-7) of 317 with a p-value of 0.00, indicating that the data are more scattered than expected for the model.

The suggested improvement in the model was adding a random effect term, $\gamma\tau_i$, to Equation (C-7) to represent variability from earthquake to earthquake resulting from unknown variables (e.g. Brillinger and Preisler, 1983). Parameter τ_i is a normal variate with 0 mean and unit variance representing a random effect for the i^{th} event, and γ is a parameter estimated from the data that defines the magnitude of this variation. Brillinger and Preisler (1983) present a general approach for estimating the coefficients of a random effects model using maximum likelihood combined with Gaussian quadrature. Applying this method, Olig and others obtained Equation (7-5). The resulting goodness of fit statistic, $\hat{C}$, was 8.4 with a p-value of 0.68, indicating a large improvement in the model. Thus, it was judged that the use of Equation (7-5) from Olig and others (1998) rather than Equation (C-7) from the Yucca Mountain study was warranted for computing the displacement hazard at the Skull Valley site.

Figure C-2 compares the predicted probabilities of distributed slip obtained using Equation (C-7) to those obtained using Equation (7-5) with the random effect set to zero. The values obtained using Equation (C-7) are much less sensitive to earthquake magnitude. Figure C-3 shows the effect on the computed displacement hazard of using Equation (C-7) instead of (7-5). At a displacement of 1 cm, there is about a factor of two increase in the frequency of exceedance. The difference between the two results decreases as the displacement level increases. The difference between the two results is primarily due to the lower rate of attenuation of the predicted probabilities of distributed slip from Equation (C-7), which results in a greater contribution from events at larger distances. Because of the attenuation in the amount of slip with distance, these events contribute more to the hazard for small displacements than large displacements. The resulting mean hazard curve using Equation (C-7) in the earthquake approach remains near or below the hazard computed using the preferred displacement approach. Thus, the overall effect on the total hazard is small.

References

Brillinger, D.R., and Preisler, H.K., 1983, Maximum likelihood estimation in a latent variable problem: *in* Studies in Econometrics, Time Series, and Multivariate Statistics: S. Karlin, T. Amemiya, and L.A. Goodman (eds.), Academic Press, New York, p. 31-65.

Hosmer, D.I., and Lemeshow, S. 1989, *Applied Logistic Regression*, John Wiley & Sons, New York, 307 p.

Olig, S., Youngs, R., and Wong, I., 1998, Probabilistic seismic hazard analysis for surface fault displacement at TA-3, Los Alamos National Laboratory: report prepared for Los Alamos National Laboratory, University of California, 7, July.

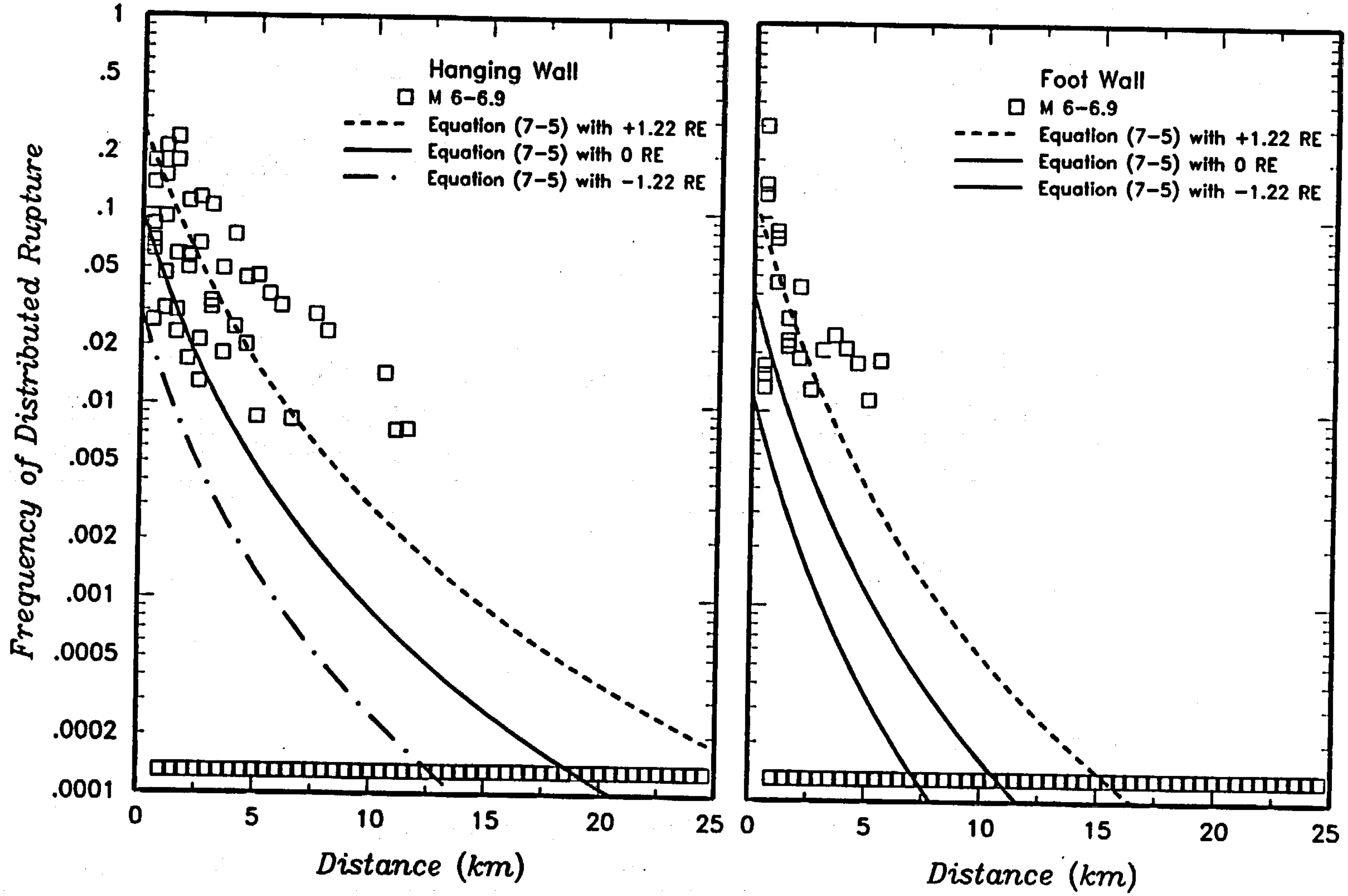

Figure C-1 Range of prediced probability of distributed rupture for magnitude 6.5 earthquakes due to +/- 2 standard deviations in the random effect

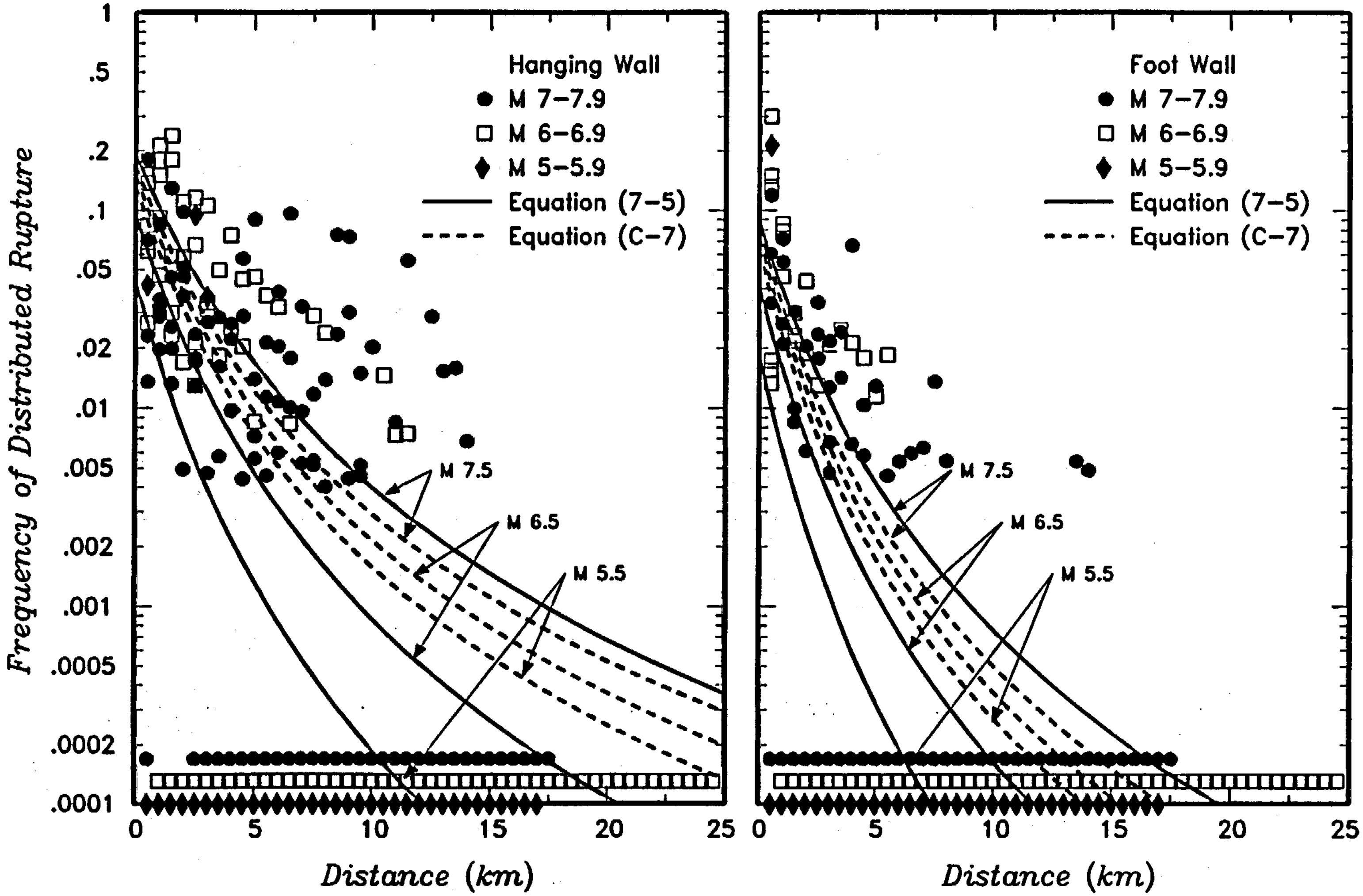

Figure C–2 Comparison of prediced probability of distributed rupture

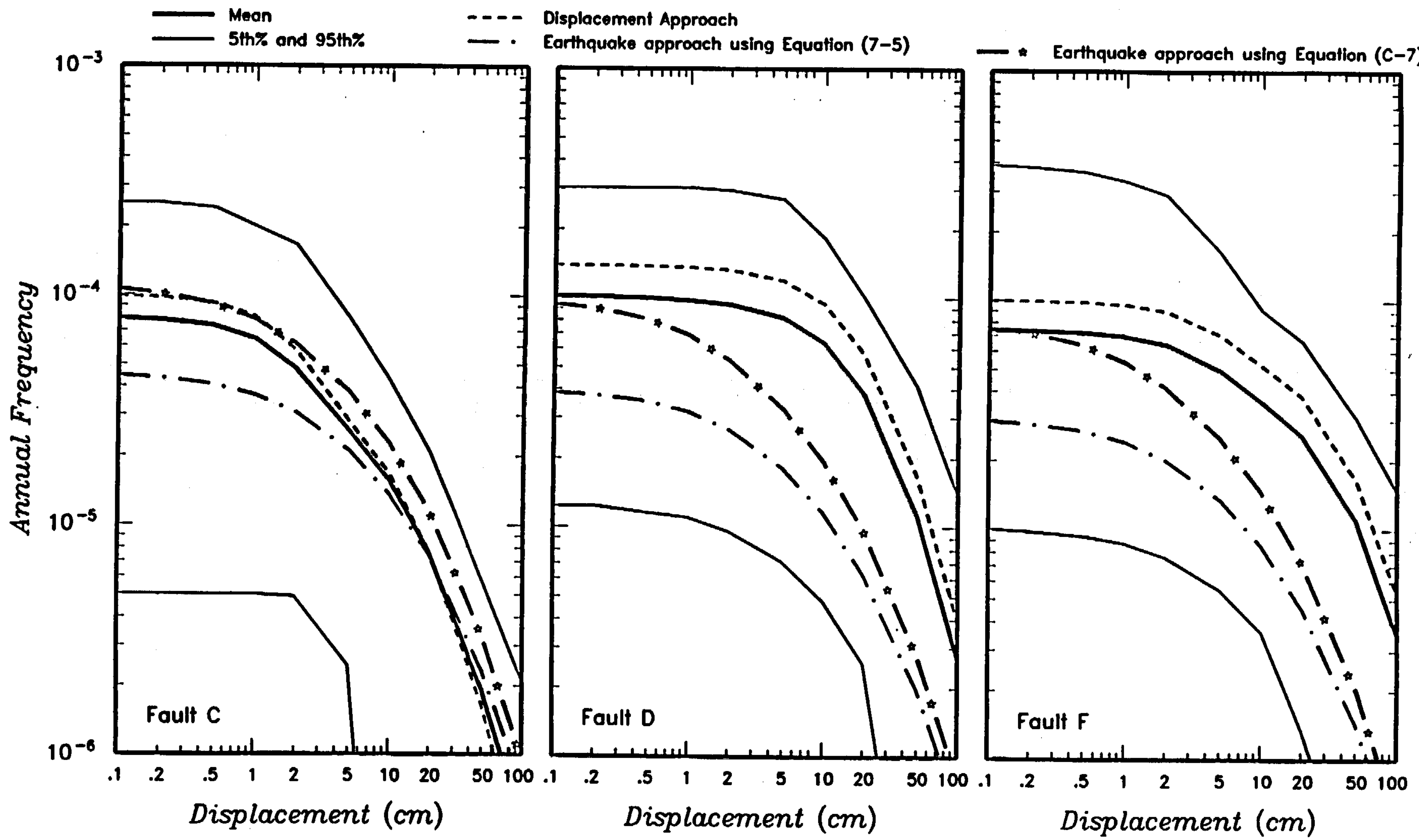

Figure C-3 Comparison of earthquake approach displacement hazard computed using Equation (C-7) with results presented on Figure 7-9 of PSHA report

CHAPTER 3

PRINCIPAL DESIGN CRITERIA

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3.2.4 Internal/External Pressure

Internal and external pressure loads are defined as loads resulting from the differential pressure between the helium fill gas inside the canister and the environmental pressure. The pressure may be positive (internal pressure) or negative (external pressure). The pressure must be considered for both normal and off-normal conditions, except for pressurization from a fuel rod rupture, which is an accident-level condition addressed under accident loads.

3.2.5 Lateral Soil Pressure

Lateral soil loads must be considered where applicable as they would result from normal, off-normal, and accident conditions. Lateral soil pressure includes lateral pressure resulting from soil and hydrostatic loads external to the structure transmitted to the structure by the adjacent soil mass.

3.2.6 Thermal Loads

Thermal loads are defined as loads resulting from normal, off-normal, and accident-level condition temperature distributions and thermal gradients within the structure, expansions and contractions of components, and restraints to expansions and contractions, except for thermal loads that are separately identified and used in the load combination.

The lowest ambient temperature taken near the site is -30° F, recorded at Salt Lake City (Reference 5).

Based on data recorded for areas near the site, Dugway (12 miles south of the site), Iosepa South Ranch (8 miles NW of the site), and the PFS met tower, the following temperatures where recorded. This information is obtained from References 6 and 33.

Temperature, F	Dugway	Iosepa Ranch	PFSF Met Data
Annual Average	51	50	49
Average Daily Maximum	94	95	92.6

Normal-level thermal loads are based on the highest recorded average annual temperature at the site. The annual average takes into account both day and night, summer and winter temperatures throughout the year and is the principle design parameter in the storage system design analysis because it establishes the basis for demonstration of long-term spent nuclear fuel integrity. The long term integrity of the spent fuel cladding is a function of the averaged ambient temperature over the entire storage period, which is assumed to be at the maximum average yearly temperature in every year of storage for conservatism in the cladding service life computations. As shown above, the highest average annual temperature taken near the site is 51° F, recorded at Dugway, 12 miles south of the site (Reference 6).

Off-normal level thermal loads are based on the highest recorded 24-hour average (day-night) temperature at the site, which represents extreme environmental conditions. However, 24-hour average temperatures are not typically recorded. A conservative approach is to use the "average daily maximum temperature," which is an average of the peak temperatures throughout the hottest month, July. Use of this temperature value, which bounds any 24-hour average provides an ample margin from the vendor's off-normal temperature limits. As shown above, the highest average daily maximum temperature taken near the site is 95° F, recorded at Iosepa Ranch, 8 miles NW of the site (Reference 6).[i]

[i] The HI-STORM Storage Cask SAR defines off-normal temperature as a three-day average temperature which shall be limited to 100°F.

Accident-level thermal loads are due to a temperature rise resulting from the loss of cooling air for an extended period of time or loads resulting from the maximum anticipated heat loads such as, a fire or burial under debris.

3.2.7 Accident Loads

Accident loads are defined as loads due to the direct and secondary effects of an off-normal or design basis accident that could result from an explosion, drop, tipover, pressurization, fire, or other human-caused occurrences. The accident events to be addressed in the design of the facility are discussed in Chapter 8.

3.2.8 Tornado and Wind Loadings

The design of SSCs shall consider loading associated with maximum site-specific meteorological conditions, including tornado and extreme wind. The tornado and wind loading used in the design shall be in accordance with ANSI/ANS 57.9 (Reference 4), NUREG-0800 (Reference 7), Regulatory Guide 1.76 (Reference 8), and ASCE-7.

3.2.8.1 Applicable Design Parameters

The normal design basis wind shall have a velocity of 90 mph as shown in Figure 6-1 of ASCE-7. The design basis wind is defined as a 3-second gust speed at 33 ft above ground for Exposure C category and is associated with an annual frequency of 2E-2 times per year.

The extreme design basis wind shall be derived from the design basis tornado. Tooele County is located in Tornado Intensity Region III as defined by Regulatory Guide 1.76, where the following design basis tornado characteristics are specified:

Design Basis Tornado Characteristics

Maximum Wind Speed	240 mph
Rotational Wind Speed	190 mph
Translational Speed	50 mph
Radius of Max. Wind Speed	150 ft
Pressure Drop	1.5 psi
Rate of Pressure Drop	0.6 psi/sec

3.2.8.2 Determination of Forces on Structures

Forces resulting from the design basis wind and the design basis tornado shall be considered in the design. The method used to convert wind loading into forces on a structure shall be in accordance with NUREG-0800 (Section 3.3.1, Wind Loadings, and Section 3.3.2, Tornado Loadings).

3.2.8.3 Ability of Structure to Perform Despite Failure of Structure Not Designed for Tornado Load

The PFSF shall be designed to ensure that SSCs that are not designed for tornado loads do not adversely affect the safety functions of SSCs that are classified as Important to Safety.

SSCs that are classified as Important to Safety but not designed for tornado loads shall be located so as to be protected by a SSC that is classified as Important to Safety and designed for tornado loads.

The Canister Transfer Building shall be designed to withstand tornado-generated wind loadings and missiles in order to protect Important to Safety SSCs housed within the building that are not designed for tornado loads.

3.2.8.4 Tornado Missiles

SSCs that are classified as Important to Safety shall be designed for tornado-generated missiles except as noted in Section 3.2.8.3.

The loaded concrete storage casks shall remain stable and the confinement boundary not breached when subjected to tornado-generated missiles.

The storage pads and Canister Transfer Building shall remain stable and structurally intact when subjected to tornado-generated missiles.

Tornado-generated missiles need not be considered in the design of the canister, overhead bridge and semi-gantry cranes, or transfer cask since the canister is protected by the storage cask and the cranes and transfer cask are protected by the Canister Transfer Building.

NUREG-0800, Section 3.5.1.4 requires that postulated tornado missiles include at least three objects: a massive high kinetic energy missile which deforms on impact, a rigid missile to test penetration resistance, and a small rigid missile of a size sufficient to just pass through any openings in protective barriers. To bound these three objects, NUREG-0800, Section 3.5.1.4 requires the applicant analyze the specific missiles defined as "Spectrum I" or "Spectrum II". Spectrum II missiles are used for the Canister Transfer Building since the type and velocity of the missiles specified are representative of the types of objects which might be found near the PFSF site. Therefore the postulated tornado missiles for the design of the Canister Transfer Building shall be in accordance with NUREG-0800, Section 3.5.1.4, for Spectrum II missiles for Region III. The tornado-generated missiles shall include:

A. 115 lb. wood plank (3.6" x 11.4" x 12' long) with horizontal velocity of 190 ft/sec.
B. 287 lb. 6" schedule 40 pipe with horizontal velocity of 33 ft/sec.
C. 9 lb. 1" diameter steel rod with horizontal velocity of 26 ft/sec.
D. 1124 lb. 13.5" diameter wooden utility pole with horizontal velocity of 85 ft/sec.
E. 750 lb. 12" schedule 40 pipe with horizontal velocity of 23 ft/sec.
F. 3990 lb. automobile with horizontal velocity of 134 ft/sec.

NOTES: Vertical velocities are 70% of horizontal velocities except for missile C. Missile C has the same velocity in all directions. Missiles A, B, C, and E are considered at all elevations. Missiles D and F are considered at elevations up to 30' above all grade levels within ½ mile of the structure.

The barrier design procedure associated with tornado-generated missiles shall be in accordance with Stone and Webster Topical Report, SWECO 7703, "Missile-Barrier Interaction", September 1977 (Reference 32), which has been submitted to and reviewed by the NRC for use at other nuclear facilities.

3.2.9 Water Level (Flood) Design

The site is located in Skull Valley, an area of western Utah with a semi-arid climate, receiving low annual precipitation. Precipitation ranges from 7 to 12 inches per year. The site has no flowing or intermittent streams nearby, however, there is evidence of minor drainage channels created by infrequent thunderstorms or snow melt runoff.

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protection. The depth of soil over bedrock is between 520 ft and 880 ft below the surface of the site (Reference 9).

3.2.10.1.8 Soil-Structure Interaction

Soil-structure interaction shall be considered in the design of soil-supported structures by including the effects of the soil properties established during the geotechnical investigation program and as represented by discrete soil springs or a finite element layered system as described in ANSI/ANS 57.9, Appendix C.

Soil boring logs and soil properties of the PFSF site are contained in Chapter 2, Appendix 2A.

3.2.10.2 Seismic-System Analysis

3.2.10.2.1 Seismic Analysis Methods

Seismic analysis methods shall be in accordance with standard practices and methods as described in ANSI/ANS 57.9, NUREG-0800, ASCE-4 (Reference 14), and others referenced herein.

The seismic response of each structure shall be determined by preparing a mathematical model of the structure and calculating the response of the model to the prescribed seismic input.

The HI-STORM storage system seismic analysis methods are described in the HI-STORM SAR, Section 11.2.1. The TranStor storage system seismic analysis methods are described in the TranStor SAR, Section 11.2.5. Site-specific cask stability analysis shall be performed to account for the site-specific seismic response spectra curves, soil-structure interaction, and the actual PFSF pad size and arrangement.

The concrete storage pads shall be analyzed with a dynamic seismic time history analysis using a finite element model with soil-structure interaction considered by the use of dynamic soil springs. Various combinations of cask placements shall be considered to determine the controlling load case.

The Canister Transfer Building shall be analyzed for seismic loads using a frequency response analysis and considering soil-structure interaction.

The overhead bridge and semi-gantry cranes shall be analyzed considering the Maximum Critical Load (maximum lifted load whose uncontrolled movement or release could adversely affect the operation of SSCs classified as Important to Safety) in combination with a seismic event in accordance with NUREG-0554 (Reference 15). A set of amplified response spectra curves at the crane rail locations shall be developed for use in the crane seismic analysis and design.

3.2.10.2.2 Natural Frequencies and Response Loads

The modal analysis considers the natural frequency of the system as well as the other significant modes of vibration. Response loads are determined from the appropriate response spectra at the calculated frequencies.

3.2.10.2.3 Procedure Used to Lump Masses

The inertial mass properties of each structure shall be modeled using the discretization of mass formulation whereby the structural mass and associated rotational inertia are discretized and lumped at node points of the model. Node points where masses are lumped shall be located at the center of gravity of the member or component represented in the model.

3.4.1 Spent Fuel Storage Systems

3.4.1.1 Canister

The canister is classified as Important to Safety, Classification Category A since it serves as the primary confinement structure for the fuel assemblies and is designed to remain intact under all accident conditions analyzed in Chapter 8.

3.4.1.2 Concrete Storage Cask

The storage cask is classified as Important to Safety, Classification Category B since it is designed to remain intact under all accident conditions analyzed in Chapter 8 and serves as the primary component for protecting the canister during storage and provide radiation shielding and canister heat rejection.

3.4.1.3 Transfer Cask

The transfer cask is classified as Important to Safety, Classification Category B since it is designed to support the canister during transfer lift operations and provide radiation shielding and canister heat rejection.

3.4.1.4 Lifting Devices

The lifting devices (lift yoke, trunnions, and canister lift attachments) are classified as Important to Safety, Classification B to preclude the accidental drop of a canister.

3.4.2 Cask Storage Pads

The cask storage pads are classified as Important to Safety, Classification Category C to ensure a stable and level support surface for the storage cask under normal, off-normal, and accident-level conditions.

3.4.3 Canister Transfer Building

The Canister Transfer Building is classified as Important to Safety, Classification Category B to protect the canister from adverse natural phenomena during shipping cask load/unload operations and canister transfer operations. The building shall provide physical protection from tornado winds and missiles, radiological shielding inside to workers during transfer operations, and support for the canister transfer cranes.

3.4.4 Canister Transfer Cranes

The overhead bridge and semi-gantry canister transfer cranes are classified as Important to Safety, Classification Category B to preclude the accidental drop of a shipping cask without impact limiters during load/unload operations or canister during the canister transfer operations.

3.4.5 Seismic Support Struts

The seismic support struts are classified as Important to Safety, Classification Category B to ensure that the casks will remain stable and will not topple in the event of an earthquake.

3.4.6 Design Criteria for Other SSCs Not Important to Safety

The design criteria for SSCs classified as Not Important to Safety, but which have security or operational importance, such as security systems, standby power systems, cask transport vehicles, flood prevention earthwork, fire protection systems, radiation monitoring systems, and temperature monitoring systems, are addressed in subsequent chapters of this SAR. These SSCs shall be required to comply with their applicable codes and standards to ensure compatibility with SSCs that are Important to Safety and to maintain a level of quality that shall ensure that they will mitigate the effects of off-normal or accident-level events as required.

The cask transporter is classified as not Important to Safety but is designed with several features that assure safety while transporting spent nuclear fuel. Potential failure mechanisms of the transporter could involve the drive-train, brakes, electrical system, or lift beam hydraulic ram. Of these potential failures, only those that could drop the cask have the possibility of damaging the cask and adversely affecting public health and safety. Because of this, the transporter is not permitted by design to lift a cask above the cask vendor's analyzed safe handling height. In addition, a Technical Specification is proposed to ensure that the casks will not be lifted above the vendor's analyzed safe handling height. Therefore, a failure of the cask transporter will not damage the spent fuel storage system or adversely affect the health and safety of the public, which is the basis for the transporter classification as Not Important to Safety.

The flood control berm is classified as not Important to Safety. Flooding due to PMF would not compromise the safety of the storage casks or the Canister Transfer Building if the berm was not constructed or if it failed since the cask systems are designed to withstand severe flooding and full submergence. The berm is provided to minimize stormwater flowing across the site for ease of operations and maintenance activities.

Complete blockage of air inlet ducts due to flooding is described in SAR Section 8.2.8, which shows that the HI-STORM inlet ducts can be blocked for 92 hours without adverse effects and the TranStor storage cask inlet ducts can be blocked for an unlimited time. PMF flows are mitigated in the Canister Transfer Building by locating the ground floor elevation above the maximum elevation of flood water. In addition, forces due to flowing water would be insignificant and would not affect the stability to the casks due to the shallow depth of the flow across the site.

The closed circuit television (CCTV) is classified as not Important to Safety. The function of the CCTV is to assist in assessment of unauthorized penetration within the protected area as required per 10 CFR 73.51 (Reference 30). As noted in NUREG-1497 (Reference 31), adequate assessment may also be provided through onsite assessment by security personnel if an acceptable justification of timely assessment can be provided. A failure of the CCTV system would be discovered immediately by security personnel as indicated by a loss of continuously observed surveillance capabilities. Appropriate compensatory measures would then be initiated, eg, sending security personnel to CCTV observation locations to provide timely onsite surveillance.

The PFSF radiation monitors are classified as not Important to Safety since they are not needed to prevent or mitigate any credible accident that would adversely affect public health and safety. The PFSF will utilize various types of radiation monitors including area monitors, thermoluminescent dosimeters (TLD), portable hand held monitors, personnel dosimetry, and portable airborne monitors. The purpose of the area radiation monitors is to detect and alarm high radiation conditions in the canister transfer building. The purpose of TLDs is to record radiation doses received at the radiation area boundary, owner controlled area boundary, and by PFSF personnel. The purpose of the portable hand held monitors is to provide surveillance of radiation levels near worker locations during transfer operations. The purpose of the personnel dosimetry, which is worn by all workers in the canister transfer area, is to measure worker accumulated dose while in the transfer area. The purpose of the portable airborne monitors is to ensure that, although the canisters are sealed, no airborne radioactivity is present during transfer operations. The use and presence of various types of monitors during facility operations provides defense in depth and will ensure that even if one fails, other monitors would detect high radiation conditions and alarm to provide safe working conditions for onsite personnel.

The temperature monitoring system is classified as not Important to Safety. The purpose of the temperature monitoring system is to provide continuous surveillance of each cask's temperature to ensure proper operation. In the event of a temperature monitor failure, the monitoring computer would not receive a signal. This would create an alarm informing personnel of a potential cask temperature problem. A temperature monitor system failure would alarm in the security monitoring area and security personnel would contact operations personnel. As discussed in SAR Section 8.2.8, under worst case conditions, cask temperature increases occur over several days, which would give operation personnel ample time to assess and resolve the problem.

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17. ASME Boiler and Pressure Vessel Code, Section III, American Society of Mechanical Engineers, 1992.

18. ACI-349, Code Requirements for Nuclear Safety Related Concrete Structures, American Concrete Institute, 1990.

19. ANSI/AISC N690-1994, Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, American Institute of Steel Construction, 1994.

20. NUREG-0612, Control of Heavy Loads at Nuclear Power Plants, U.S. Nuclear Regulatory Commission, 1980.

21. NUREG-1536, Standard Review Plan for Dry Cask Storage Systems, Nuclear Regulatory Commission, 1997.

22. Regulatory Guide 1.91, Evaluations of Explosions Postulated to Occur on Transportation Routes near Nuclear Power Plants, U.S. Nuclear Regulatory Commission, February 1978.

23. 29 CFR 1910.179, Overhead and Gantry Cranes, Occupational Safety and Health Standards (OSHA).

24. NUREG/CR-6407, (INEL-95/0551), Classification of Transportation Packaging and Dry Spent Fuel Storage System Components According to Importance to Safety, 1996.

25. U.S. NRC Regulatory Guide 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," March 1997.

26. PFS Letter, Parkyn to Delligatti (NRC), Request for Exemption to 10 CFR 72.102(f)(1), dated April 2, 1999.

27. Geomatrix Consultants, Inc., Development of Design Ground Motions for the Private Fuel Storage Facility, Private Fuel Storage Facility, Skull Valley Utah; March 1999.

28. Geomatrix Consultants, Inc., Fault Evaluation Study and Seismic Hazard Assessment, Private Fuel Storage Facility, Skull Valley Utah; Final Report, February 1999, 3 volumes.

29. PFS Letter, Parkyn to Delligatti (NRC), Request for Exemption to 10 CFR 72.102(f)(1), dated August 24, 1999.

30. 10 CFR 73.51, Requirements for the Physical Protection of Stored Spent Nuclear Fuel or High-Level Radioactive Waste.

31. NUREG-1497, Interim Licensing Criteria for Physical Protection of Certain Storage of Spent Fuel, November 1994.

32. Stone and Webster Topical Report, SWECO 7703, "Missile-Barrier Interaction", September 1977

33. PFSF Meteorological Data taken during 1997 and 1998.

TABLE 3.6-1
(Sheet 1 of 5)

SUMMARY OF PFSF DESIGN CRITERIA

DESIGN PARAMETERS	DESIGN CONDITIONS	APPLICABLE CRITERIA AND CODES
GENERAL		
PFSF Design Life	40 years	PFSF Specifications
Storage Capacity	40,000 MTU of commercial spent fuel	PFSF Specifications
Number of Casks	approximately 4,000 casks	PFSF Specifications
SPENT FUEL SPECIFICATIONS		
Type of Fuel	See Tables 3.1-1 and 3.1-2	HI-STORM SAR TranStor SAR
Fuel Characteristics	See Table 3.1-3	HI-STORM SAR TranStor SAR
STORAGE SYSTEM CHARACTERISTICS		
Canister Capacity	<u>HI-STORM</u> 24 PWR assemblies/canister 68 BWR assemblies/canister <u>TranStor</u> 24 PWR assemblies/canister 61 BWR assemblies/canister	HI-STORM SAR, Section 1.1 TranStor SAR, Section 1.1
Weights (maximum)	<u>HI-STORM</u> Storage Cask - 268,334 lbs. Loaded Canister - 87,241 lbs. Transfer Cask - 152,636 lbs. Shipping Cask - 153,080 lbs. <u>TranStor</u> Storage Cask - 222,200 lbs. Loaded Canister - 84,020 lbs. Transfer Cask - 126,230 lbs. Shipping Cask - 160,900 lbs.	 HI-STORM SAR, Table 3.2.1 " HI-STORM SAR, Table 3.2.2 Shipping SAR, Table 2.2.1 TranStor SAR, Table 3.2-1 " " Shipping SAR, Table 2.2-1

TABLE 3.6-1
(Sheet 2 of 5)

SUMMARY OF PFSF DESIGN CRITERIA

DESIGN PARAMETERS	DESIGN CONDITIONS	APPLICABLE CRITERIA AND CODES
STRUCTURAL DESIGN		
Wind	90 mph, normal speed	ASCE-7
Tornado	240 mph, maximum speed 190 mph, rotational speed 50 mph, translational speed 150 ft, radius of max speed 1.5 psi, pressure drop 0.6 psi/sec rate of drop	Reg. Guide 1.76
Tornado Missiles	115 lb. wood plank, 190 ft/sec 287 lb. 6" schedule 40 pipe, 33 ft/sec 9 lb. 1" diameter steel rod, 26 ft/sec 1124 lb. wooden utility pole, 85 ft/sec 750 lb. 12" schedule 40 pipe, 23 ft/sec 3990 lb. Automobile, 134 ft/sec	NUREG-0800, Section 3.5.1.4
Flood	N/A - PFSF is not in a flood plain and is above the PMF elevation	PFSF SAR Section 2.3.2.3
Seismic	0.53g, horz.(both directions) & 0.53 g vert. Design basis ground acceleration	10 CFR 72.102, Reg. Guide 1.165
Snow & Ice	P(g) = 45 psf	ASCE-7/County
Allowable Soil Pressure	Static = 4 ksf max Dynamic = Varies by footing type/size	PFSF SAR Section 2.6.1.12
Explosion Protection	N/A - PFSF is located beyond distances from transportation routes from where cargo explosions could cause overpressures > 1 psi.	Reg. Guide 1.91
Ambient Conditions	Low Temperature = -30°F Max. Annual Average Temp. = 51°F Average Daily Max. Temp. = 95°F Humidity = 0 to 100 %	NOAA Data-Salt Lake City UT Climate Data UT Climate Data

TABLE 3.6-1
(Sheet 3 of 5)

SUMMARY OF PFSF DESIGN CRITERIA

DESIGN PARAMETERS	DESIGN CONDITIONS	APPLICABLE CRITERIA AND CODES
HI-STORM 100 Cask System Load Criteria	Canister: / Internals: / Storage Cask: / Transfer Cask: } See HI-STORM SAR, Table 2.2.6	ASME III, NB ASME III, NG ASME III NF, ACI-349 ASME III NF, ANSI N14.6
TranStor Storage Cask System Load Criteria	Canister: / Internals: / Storage Cask: / Transfer Cask: } See TranStor SAR, Section 2.2.7	ASME III, NC ASME III, NG ANS 57.9, ACI-349 ANSI N14.6
Cask Storage Pads Load Combinations	<u>Normal Conditions</u> $U_c > 1.4D + 1.7L$ $U_c > 1.4D + 1.7L + 1.7H$ <u>Off-Normal Conditions</u> $U_c > 0.75(1.4D + 1.7L + 1.7H + 1.7T)$ $U_c > 0.75(1.4D + 1.7L + 1.7H + 1.7T + 1.7W)$ <u>Accident-Level Conditions</u> $U_c > D + L + H + T + (E \text{ or } A \text{ or } W_t \text{ or } F)$ $U_c > D + L + H + T_a$	ANSI/ANS 57.9 ACI-349
Canister Transfer Building Structure Load Combinations (Reinforced Concrete)	<u>Normal Conditions</u> $U_c > 1.4D + 1.7L$ $U_c > 1.4D + 1.7L + 1.7H$ <u>Off-Normal Conditions</u> $U_c > 0.75(1.4D + 1.7L + 1.7H + 1.7T)$ $U_c > 0.75(1.4D + 1.7L + 1.7H + 1.7T + 1.7W)$ <u>Accident-Level Conditions</u> $U_c > D + L + H + T + (E \text{ or } A \text{ or } W_t \text{ or } F)$ $U_c > D + L + H + T_a$	ANSI/ANS 57.9 ACI-349
Canister Transfer Building Structure Load Combinations (Structural Steel)	<u>Normal Conditions</u> S and $S_v > D + L$ or $D + L + H$ <u>Off-Normal Conditions</u> $1.3(S \text{ and } S_v) > D + L + H + W$ $1.5S > D + L + H + T + W$ $1.4S_v > D + L + H + T + W$ <u>Accident-Level Conditions</u> $1.6S > D + L + T + (W_t \text{ or } E)$ $S_v > D + L + T + (W_t \text{ or } E)$	ANSI/ANS 57.9 ANSI/AISC N690

TABLE 3.6-1
(Sheet 4 of 5)

SUMMARY OF PFSF DESIGN CRITERIA

DESIGN PARAMETERS	DESIGN CONDITIONS	APPLICABLE CRITERIA AND CODES
Canister Transfer Building Foundation Load Combinations	<u>Normal Conditions</u> $U_f > 1.4D + 1.7L + 1.7G$ $U_f > 1.4D + 1.7L + 1.7H + 1.7G$ <u>Off-Normal Conditions</u> $U_f > 0.75 (1.4D + 1.7L + 1.7H + 1.7T + 1.7G)$ $U_f > 0.75 (1.4D + 1.7L + 1.7H + 1.7T + 1.7W + 1.7G)$ <u>Accident-Level Conditions</u> $U_f > D + L + H + T + G + (E \text{ or } A \text{ or } W_t \text{ or } F)$ $U_f > D + L + H + T_a + G$	ANSI/ANS 57.9 ACI-349
Canister Transfer Crane Designs	Type I, single-failure-proof 200 ton overhead bridge crane 150 ton semi-gantry crane	ASME NOG-1, NUREG 0554, & NUREG 0612
Canister Transfer Crane Load Combinations	<u>Normal Conditions</u> $P_c = P_{db} + P_{dt} + (P_{lr} \text{ or } P_p)$ $P_c = P_{db} + P_{dt} + P_{lr} + (P_v \text{ or } P_{ht} \text{ or } P_{hl}) + P_{wo}$ <u>Off-Normal Conditions</u> $P_c = P_{db} + P_{dt} + P_a + P_{wo}$ <u>Accident-Level Conditions</u> $P_c = P_{db} + P_{dt} + P_{cs} + P_e + P_{wo}$ $P_c = P_{db} + P_{dt} + P_e + P_{wo}$ $P_c = P_{db} + P_{dt} + P_{wt}$	ASME NOG-1
THERMAL DESIGN		
Design Temperatures (°F) (maximum)	<u>HI-STORM</u> — <u>Norm</u> / <u>Off-norm</u> / <u>Acc</u> Stor. cask conc. 300 / 300 / 1200 Stor. cask steel 350 / 350 / 1350 Fuel Cladding 716 / 716 / 1058 <u>TranStor</u> Outer cask conc. 150 / 150 / 200 Inner cask conc. 200 / 225 / 350 PWR Cladding 621 / 1058 / 1058 BWR Cladding 673 / 1058 / 1058	HI-STORM SAR, Table 2.2.3 TranStor SAR, Table 4.1-1

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foundation with an effective Young's Modulus not exceeding 28,000 psi. The reference pad is a "stiffer" pad than the PFSF storage pad. Therefore, non-credible hypothetical tipover of a HI-STORM storage cask at the PFSF would result in an acceleration less than 45 g and lower stresses than those evaluated in the HI-STORM SAR.

Based on the reference target ISFSI properties, Holtec calculated the maximum drop height for a vertical end drop of the loaded HI-STORM storage cask that would result in a deceleration below 45 g. This height was determined to be 11 inches. There are no operations at the PFSF where a storage cask would be raised above the 11 inch analyzed drop height. The cask transporter, used to move storage casks from the Canister Transfer Building to the storage pads, lifts the cask approximately 4 inches above the surface. The transporter is designed to mechanically limit the heights below 10 inches. The cask drop analyzed in the HI-STORM SAR bounds the PFSF design criteria in Section 3.2.7 for accident drop loads since the HI-STORM storage cask will not be dropped from a height approaching 11 inches, and because the PFSF storage pads are not as stiff as that considered in the 11 inch vertical end drop analysis in the HI-STORM SAR.

For the canister, the peak acceleration of 45 g established for the side and end drops is bounded by the 60 g acceleration calculated for drop accidents analyzed in Section 2.7.1 of the HI-STAR 10 CFR 71 Shipping SAR (Reference 3). Since the accelerations are bounding, the stresses (produced by 60 g vertical and horizontal accelerations) analyzed by the HI-STAR stress analyses and determined to be acceptable also bound stresses that would result from the HI-STORM tipover and end drop accidents.

For the storage cask, HI-STORM SAR Section 3.4.4.3.2.3 evaluates the buckling capacity of the cask based on the 11 inch vertical end drop and resulting 45 g acceleration. No credit was taken for the structural stiffness of the radial concrete shielding. The minimum factor of safety for material allowable stresses for all portions of the cask structure is 1.10. The tipover event evaluated in the HI-STORM SAR specifies that the cask lid must remain in place due to the 45 g horizontal acceleration. HI-STORM SAR Section 3.4.4.3.2.2 demonstrates that the minimum factor of safety for the cask lid and lid bolts is 1.29, which exceeds the minimum required 1.10 factor of safety.

F. Tornado Winds and Missiles

Tornado wind and tornado missile loads are addressed in HI-STORM SAR Sections 3.1.2.1.1.5 and 3.4.8. The loads are based on a worst case design basis tornado in accordance with Regulatory Guide 1.76 (Reference 4) for Intensity Region I and postulated tornado-generated missiles in accordance with NUREG-0800 (Reference 5) for Spectrum I missiles. The site is located in Tornado Intensity Region III per Regulatory Guide 1.76, which has less severe tornado conditions than Region I. The postulated missile loads used in the HI-STORM analysis are the same as in the PFSF design criteria. Since the HI-STORM design tornado wind loads exceed the PFSF design criteria and tornado-generated missile loads bound the PFSF design criteria described in Section 3.2.8, the HI-STORM design meets PFSF design criteria.

G. Flood

Flood loads are addressed in HI-STORM SAR Sections 3.1.2.1.1.3 and 3.4.6. The HI-STORM storage system is designed to withstand hydrostatic pressure (full submergence) up to a depth of 125 ft and horizontal loads due to water velocity up to 16.24 fps without tipping or sliding. The PFSF is above probable maximum flood temperatures assumed in the TranStor analysis bound the PFSF specific environmental conditions contained in Section 3.2.6 for thermal loads.

D. Handling Loads

Handling loads for normal conditions are contained in TranStor SAR Section 3.4.4.1.4 and for off-normal conditions in TranStor SAR Section 11.1.5. The normal handling decelerations for the canister were assumed to be 0.5 g in all directions. This value is expected to encompass all normal handling operations at the PFSF. The maximum analyzed off-normal handling load is conservatively calculated to be 17.5 g, generated by a 2 fps horizontal impact from a crane handling operation. Since the maximum trolley speed for the overhead crane at the PFSF is 60 fpm (1 fps), the canister will not reach the 2 fps analyzed impact load. Therefore, the TranStor analysis bounds the site specific criteria.

E. Cask Drop and Tipover

Cask drop loads are addressed in TranStor SAR Section 12.2.2.8 and cask tipover loads are addressed in TranStor SAR Section 11.2.10. The storage cask and canister are capable of withstanding accidental drops up to 80 inches without breaching the containment boundary, preventing removal of fuel assemblies, causing a criticality accident, or causing a structural failure of the concrete cask so it cannot maintain its shielding function. The vendor does not consider drop heights below 18 inches to be a concern. There are no operations at the PFSF where a storage cask would be raised above the 18 inch drop height. The cask transporter, used to move storage casks from the Canister Transfer Building to the storage pad, lifts the cask about 4 inches above the concrete. The transporter is designed to mechanically limit the heights below 10 inches. Therefore, the cask drop in the TranStor analysis bounds the PFSF design criteria in Section 3.2.7 for accident drop loads.

Cask tipover, resulting from tornado winds (with a concurrent tornado-generated missile strike) or earthquake, are evaluated and the cask is capable of remaining stable during these events. Therefore, cask tipover in the TranStor analysis bounds the PFSF design criteria in Section 3.2.7 for non-credible hypothetical tipover event loads.

The non-credible hypothetical cask tipover analysis in the TranStor SAR is based on a storage pad that is conservatively assumed to be rigid when cask crushing is considered. However, to determine the forces on the cask and canister due to impact, the assumption is reversed and the cask is assumed to be rigid. In this case, the storage pad is considered to be a yielding surface and the pad target hardness is calculated in accordance with EPRI NP-7551 (Reference 12). The TranStor SAR calculates the impact load from a storage cask tipover as being 17 g. For conservatism, the maximum bounding value for any drop height is calculated to be 19.8 g. Using PFSF site-specific concrete and soil parameters, and applying the same target hardness methodology used by SNC, an acceleration of 16.2 g was calculated to result from a horizontal drop of a TranStor storage cask on to a PFSF storage pad from any drop height.

F. <u>Tornado Winds and Missiles</u>

Tornado wind and tornado generated missile loads are addressed in TranStor SAR Sections 2.2.2 and 11.2.3. The TranStor storage cask is designed for tornado winds and tornado-generated missiles, which include a design basis tornado in accordance with Regulatory Guide 1.76 (Reference 4) for Intensity Region I and postulated tornado-generated missiles in accordance with NUREG-0800 (Reference 5) for Spectrum I missiles. The PFSF is located in Tornado Intensity Region III per Regulatory Guide 1.76, which has less severe tornado conditions than Region I. The postulated missile loads used in the TranStor analysis bound the PFSF design criteria. Since the TranStor tornado wind loads exceed the PFSF design criteria and tornado-generated missile loads bound the PFSF design criteria described in Section 3.2.8, the TranStor design meets the PFSF design criteria.

G. Flood

Flood loads are addressed in TranStor SAR Section 11.2.4. The TranStor system is designed to withstand a flood up to a depth of 20-ft and a stream velocity of 24.6 fps without overturning the cask. The PFSF is above probable maximum flood conditions, therefore, the TranStor design meets the PFSF design criteria in Section 3.2.9 for flood conditions.

H. Earthquake

Earthquake loads are addressed in the TranStor SAR Sections 2.2.5 and 11.2.5. The TranStor SAR shows that the storage system will withstand the imposed loads and not begin to rock when subjected to a generic seismic event. A generic seismic event was defined for the TranStor system using response spectra curves from Regulatory Guide 1.60 (Reference 6) with a zero period acceleration of 0.38g horizontal (both directions) and 0.25g vertical. In addition, the cask vendor initially performed a site specific analysis and determined the HI-STORM storage casks will withstand the imposed loads and not tip over when subjected to the PFSF deterministic design earthquake (0.67g horizontal, 0.69g vertical – See Section 8.2.1). Also, a site specific analysis was performed for TranStor storage casks subjected to the design basis ground motion associated with the probabilistic seismic hazard analysis with the 2,000-yr return period (0.53g horizontal, 0.53g vertical). This analysis determined maximum rocking of the top of the cask of less than 1 inch and maximum sliding of less than 2.25 inches (Reference 64). The analyses concluded that the casks do not tip over, collide, nor slide off the storage pad for these earthquakes. Soil-structure interaction was considered in the site specific analyses. The seismic cask stability analyses are fully described in Section 8.2.1.

Since the cask is demonstrated to remain standing during an earthquake, the stresses in the basket can be evaluated by comparison to the off-normal handling analysis. The seismic accelerations are well bounded by the 17.5 g load used for the basket design during the off-normal handling event. Therefore, no additional evaluation of basket stresses is required.

Even though the storage cask will not tip over during an earthquake, the storage cask is conservatively analyzed for a hypothetical cask tip over event in TranStor SAR Section 11.2.10. The analysis shows that tip over results in deceleration that would not cause any critical damage to the storage cask or fuel basket.

Therefore, the TranStor storage system design meets the PFSF design criteria requirements in Section 3.2.10 for seismic design.

I. Explosion Overpressure

Explosion overpressure loads are addressed in TranStor SAR Section 11.2.8. Regulatory Guide 1.91 (Reference 9) requires a detailed review of the system for overpressures that exceed 1 psi. The TranStor storage system is analyzed and designed to withstand an explosion that could result in overturning or sliding the storage cask. The minimum pressure on the cask to produce this force was an overpressure of 5.4 psig. As shown in Section 8.2.4, the PFSF is not subject to explosions that are in excess of 1 psig. Since the PFSF will not see explosion pressures that exceed 1 psig, the TranStor design meets the PFSF design criteria in Section 3.2.7 for explosion accident loads as required per 10 CFR 72.122(c).

J. Fire

Fire is addressed in TranStor SAR Section 2.3.6. The TranStor storage system materials and location at the PFSF safely protects the spent fuel from fires in accordance with 10 CFR 72.122(c). The storage cask is highly resistant to the effects

4.2.3 Cask Storage Pads

The design criteria for the cask storage pads are described in Chapter 3. The analysis methods and resulting design of the pads are described below.

4.2.3.1 Design Specifications

The design of the cask storage pads is in accordance with ANSI/ANS-57.9 (Reference 14) and ACI 349 (Reference 15) as identified in Chapter 3.

The cask storage pads are independent structural units constructed of reinforced concrete. Each pad is 30 ft wide by 64 ft long and 3 ft thick. The size of the pad is based on a center to center spacing of 15 ft for the storage casks. The ends of the storage pad are provided with an additional 2 ft in length to support both tracks of the cask transporter on the pad. The pads are nearly flush with grade for direct access by the cask transporter. Each cask storage pad is capable of supporting 8 loaded HI-STORM or TranStor storage casks.

An independent modular pad design was chosen to simplify the pad analysis (i.e. minimize the number of cask placement combinations) and to minimize the effects of thermal expansion. The modular pad design also provides for ease of construction by limiting the number of concrete pad construction and/or expansion joints required and allows for staged construction of the facility.

The cask storage pad design is based on a maximum loaded storage cask weight of 360,000 lbs. This maximum weight envelopes the maximum loaded weight of both the TranStor and HI-STORM concrete storage casks proposed for use at the PFSF. The TranStor storage casks proposed for use at the PFSF have a maximum loaded weight of 297,200 lbs. (PWR fuel) and 307,600 lbs. (BWR fuel) as shown in the TranStor SAR

Table 3.2-1. The HI-STORM storage casks proposed for use at the PFSF are the MPC-24 (PWR fuel) and the MPC-68 (BWR fuel) with maximum weights of 348,321 lb. and 355,575 lb., respectively, as shown in the HI-STORM SAR Table 3.2-1.

The cask storage pad design also considers the weight of the loaded concrete storage casks in combination with the seismic loads due to the site-specific probabilistic seismic hazard assessment (PSHA) design basis earthquake (0.53g horizontal in two directions and 0.53g vertical – See Section 8.2.1.1).

4.2.3.2 Plans and Sections

The site plan, which shows the locations of the concrete storage pads, is shown in Figure 1.2-1. A typical concrete storage pad plan, cross section, and details are shown in Figure 4.2-7.

4.2.3.3 Function

The function of the cask storage pads is to provide a level and stable surface for placement and storage of the TranStor and HI-STORM concrete storage casks containing the spent fuel canisters.

4.2.3.4 Components

The components of the cask storage pads consist of the materials of construction, which include concrete with a minimum 28-day compressive strength of 3,000 psi and reinforcing steel with a minimum yield strength of 60,000 psi.

4.2.3.5 Design Bases and Safety Assurance

The design bases for the cask storage pads are identified in Chapter 3.

The cask storage pads are classified as being Important to Safety in order to provide the appropriate level of quality assurance in the design and construction. This provides for the safety assurance that the cask storage pads will perform their intended function.

4.2.3.5.1 Storage Pad Analysis

The reinforced concrete pads were analyzed and designed in accordance with nuclear industry standard structural analysis and design methods (Reference 16). The static and dynamic analyses for evaluating the concrete pad response displacements and internal stresses have used standard finite element analysis computer programs CECSAP (Reference 17) and SASSI (Reference 18) computer codes.

Static analyses have been performed for the dead load and design live (storage cask) loads using the CECSAP computer program. Dynamic analyses have been performed for the site-specific probabilistic seismic hazard assessment (PSHA) design basis earthquake using the CECSAP computer program. In addition, a separate dynamic analysis was performed using the SASSI computer program to more rigorously account for the effect of soil-structure interaction. These static and dynamic analyses confirm the structural adequacy of the reinforced concrete storage pad for supporting the storage casks when subjected to the design loading conditions. The results of the pad dynamic analysis using SASSI confirmed validity and indicated conservatism of the corresponding results using CECSAP.

The structural analyses of the pad used a three-dimensional flat-shell finite element model for the concrete pad. The finite element model mesh developed for the pad is shown in Figure 4.2-8. A total of 264 flat-shell finite elements have been used to model the concrete pad. Gross uncracked stiffnesses have been used for the model. The finite element mesh was developed with the consideration that it would produce reasonably refined distribution of internal forces and moments. Also, the nodal points of the mesh coincided with the locations of the static and dynamic loadings associated with one to eight casks to be applied on the pad. These loadings are lumped to four points on the outer circular perimeter of each cask corresponding to the four quadrants of the cask. Various cask loading patterns were considered to determine the maximum pad internal stresses.

To represent the soil support condition of the pads for the long-term static (i.e. dead and live) load conditions, vertical boundary soil springs tributary to each node of the pad finite element model were used in the CECSAP static finite element model. The spring stiffness values for the static loading cases were developed from the modulus of subgrade reaction of the supporting soil medium (References 71). The spring stiffness values were varied to account for uncertainties in the soil properties by using lower and upper bound conditions (Reference 40). For the short-term PSHA design basis earthquake loading condition, three-component (two horizontal and one vertical) boundary soil springs and dashpots representing the dynamic soil stiffnesses and radial damping characteristics of the supporting soil medium were used to connect to each node of the pad model. The soil spring stiffness (and its associated soil mass), and radial damping coefficient tributary to each node were derived from the lumped soil spring stiffness, mass, and damping coefficient values based on the procedure in ASCE-4 (Reference 20). For the dynamic analysis using the SASSI computer program, the soil support to the pad was represented by three-component (two horizontal and one vertical) complex-valued, frequency-dependent, dynamic soil impedance functions that are connected to each node of the pad finite element model. The soil impedance functions were computed numerically within the SASSI computer program based on the free-field profile and dynamic properties of the soil layers underlying the pad.

The pad structural analyses included both static and dynamic analyses. The static analysis evaluated the pad response stresses due to the dead and (cask) live loads. The dynamic analysis evaluated the pad response due to the PSHA design basis earthquake loadings. The pad responses obtained from these analyses were then combined to give the combined maximum response values in accordance with the applicable load combinations. The combined response values were used for checking the structural adequacy of the concrete pad and the dynamic soil bearing capacity and overturning and sliding stability. The static and dynamic pad analyses performed for the pad are separately described below.

A. Static Analysis

The static pad analysis, using the CECSAP finite element model of the pad shown in Figure 4.2-8, was conducted for the dead load equal to the gravitational dead weight of the pad and the live load of the casks. The static loading cases were performed under a bounding range of soil spring values (stiffness) to account for potential uncertainties in the soil properties used in the analyses. Each load case was evaluated for lower and upper bound soil values using the soil property provided in Reference 40 and 71.

The live loads from three loading patterns of 2, 4, and 8 fully-loaded casks were considered. The weight of one fully-loaded cask considered was 360 kips. To simulate the condition of one fully-loaded cask being transported onto the pad, one additional cask loading pattern consisting of 7 fully-loaded casks and one fully-loaded cask being lifted by a cask transporter on the pad having a weight of 135 kips (Reference 21) was also considered. For conservatism, a dynamic impact (or amplification) factor equal to 2.0 was used for the load of one fully-loaded cask plus the transporter to account for any dynamic effect that may arise while transporting the cask.

Based on the results of this analysis, the cask-loading pattern that produces the highest pad internal stresses is that of four casks on the pad and the worst-case loading that produces the largest soil bearing pressure is that of 7 casks plus one cask being carried by the transporter. The maximum response results obtained from the static analyses are summarized in Table 4.2-7.

Soil pressures beneath the storage pad were also verified to be within the acceptance criteria for static loading conditions. Actual soil bearing pressures were calculated beneath the pad and compared to the allowable soil bearing pressures identified in Section 2.6.1.12.1 for various static load combinations. The maximum static soil pressure was calculated to be 4.0 ksf under the static (dead plus live [snow plus 7 casks plus the loaded transporter]) loading condition. The maximum calculated static soil pressure is less than the minimum allowable soil bearing pressure for static loads (4.35 ksf, as shown in Table 2.6-6).

B. Dynamic Analysis

The dynamic analysis was performed to determine the pad response stresses under the PSHA design basis earthquake loading. The dynamic loading cases were analyzed for a bounding range of soil spring values (stiffness) to account for potential uncertainties in the soil properties used in the analyses. Each load case was evaluated for lower bound, best estimate, and upper bound soil values using the data and soil properties provided in References 40 and 71.

The global seismic time-history response analysis was performed utilizing a series of cask-pad-soil interaction models to represent the dynamic characteristics of one to eight casks supported on the pad, which is supported on the site soil. To account for uncertainties in the frictional resistance to horizontal movements of casks on the pad, the friction coefficient between the cask base and the concrete pad considered in these analyses was varied from a lower-bound value of 0.2 to an upper-bound value of 0.8. The case with the lower-bound friction results in an upper-bound estimate of the sliding displacements of the casks on the pad and a lower-bound estimate of the cask dynamic forces acting on the pad, whereas the upper-bound friction case results in a lower-bound estimate of the sliding displacements and an upper-bound estimate of the cask dynamic forces acting on the pad. Thus, for the purpose of determining the upper-bound seismic stresses in the pad, the cask dynamic force time histories resulting from the upper-bound friction case were conservatively used as the dynamic forcing function inputs to the pad.

The HI-STORM storage cask weight and center of gravity loadings bound those of the TranStor storage cask; therefore, the dynamic forcing function inputs were obtained from the Holtec site-specific cask stability analysis for the HI-STORM storage cask (Reference 7). These dynamic forcing time histories were evaluated for each cask at four points that are equally-spaced along the circular outer perimeter of the cask base. At each point, a set of three-component (two horizontal and one vertical) dynamic forcing time histories was evaluated, which represents the lumped dynamic reaction forces of the pad to the cask within the four quadrants of each circular cask-base area.

In evaluating the pad dynamic stresses due to the dynamic forces of the casks acting on the pad, the finite element model of the pad-soil system (using CECSAP) was used and the dynamic force time histories of the casks were applied on the pad as nodal forcing functions. To reasonably bound the various cask loading patterns, the same 2, 4, and 8-cask loading configurations that were considered in the static analyses were analyzed. The maximum values of the pad response shear forces and bending moments resulting from the analysis were then evaluated and used for checking the structural adequacy of the pad design. The maximum values of the three-component (two horizontal and one vertical) soil-spring reaction forces were also evaluated and used for checking the overturning stability, the soil bearing capacity, and the sliding stability of the pad for the dynamic loads due to the design basis ground motion. Refer to Section 2.6.1.12.1 for a discussion of these analyses.

To provide a comparison and an assessment of the accuracy and conservatism of the dynamic analysis results from the CECSAP pad-soil system model, an additional dynamic analysis was also performed for a selected dynamic loading case using the SASSI computer program. The dynamic response results obtained from this SASSI finite element analysis were compared with the corresponding results obtained from the CECSAP analysis. This comparison indicates that the CECSAP analysis results are conservative relative to the corresponding SASSI results by a margin of greater than 20 percent.

The results of the maximum dynamic response values obtained from the dynamic analyses described above are summarized in Table 4.2-8. Based on these results, the loading that produces the maximum dynamic pad internal stresses and soil pressures is that of two casks, and the loading that produces the largest seismic horizontal soil reaction forces is that of 8 casks. These values were included in the analyses of dynamic bearing capacity and sliding stability of the pad, which are discussed in Section 2.6.1.12.1.

The maximum dynamic soil pressures, which include earthquake loadings, were also calculated for the pad dead load plus 2 casks, 4 casks, and 8 casks. The resulting soil pressure distribution was converted to an average soil pressure over an effective pad width and compared to the allowable dynamic soil pressures. The maximum soil pressure under dynamic (dead plus live plus the PSHA design basis earthquake) loading condition is 6.53 ksf, which is below the minimum allowable dynamic soil bearing pressure for dynamic loads (10.4 ksf, as shown in Table 2.6-7).

The sliding stability of the cask storage pads was also analyzed using the dynamic forces applicable for the PSHA design basis earthquake and found to be adequate. Refer to Section 2.6.1.12.1 for details and results of these analyses.

4.2.3.5.2 Storage Pad Design

The storage pad design is a 3-ft thick reinforced concrete slab with #10 longitudinal and transverse horizontal reinforcing bars spaced at 12 inches on center each way at the top face and #10 longitudinal and transverse horizontal reinforcing bars spaced at 6 inches on center each way at the bottom face of the pad. The top and bottom face horizontal reinforcements are tied through the thickness of the pad by #8 vertical shear reinforcing bars spaced at 12 inches on center each way in two ways uniformly distributed over the entire pad. The concrete has a minimum 28-day compressive strength of 3,000 psi and the reinforcing steel has a minimum yield strength of 60,000 psi.

Static design moments are based on the 1.4D + 1.7L + 1.7H load combination. The design provides an ultimate static moment capacity in the longitudinal pad direction of $-M_{yy}$ = 332 k-ft/ft and $+M_{yy}$ = 182 k-ft/ft. The capacities exceed the demand moments of $-M_{yy}$ = 114 k-ft/ft and $+M_{yy}$ = 124 k-ft/ft. The moment capacity to demand ratios in the transverse pad direction is less than that for the longitudinal direction.

Static design shear values are also based on the 1.4D + 1.7L + 1.7H load combination. The design provides an ultimate static beam shear capacity of 134 k/ft and an ultimate static punching shear capacity of 134 k/ft. The capacities exceed the demand shears of V_u (beam) = 19 k/ft and V_u (punching) = 9 k/ft.

Dynamic (or accident-level) design moments are based on the D + L + H + E load combination. The design provides an ultimate dynamic (impulse or impactive) moment capacity in the longitudinal pad direction of $-M_{yy}$ = 367 k-ft/ft and $+M_{yy}$ = 202 k-ft/ft. The capacities exceed the demand moments of $-M_{yy}$ = 332 k-ft/ft and $+M_{yy}$ = 131 k-ft/ft. The design also provides a moment capacity in the transverse pad direction of $-M_{xx}$ = 350 k-ft/ft and $+M_{xx}$ = 201 k-ft/ft. These exceed the demand moments of $-M_{xx}$ = 348 k-ft/ft and $+M_{xx}$ = 154 k-ft/ft.

Dynamic (or accident-level) design shear values are also based on the D + L + H + E load combination. The design provides an ultimate dynamic beam shear capacity of 148 k/ft and an ultimate dynamic punching shear capacity of 148 k/ft. The capacities exceed the demand shears of V_u (beam) = 80 k/ft and V_u (punching) = 147 k/ft.

Therefore, the storage pad as designed provides adequate strength for accommodating the design loading conditions.

4.2.3.5.3 Storage Pad Settlement

The relationship of major foundations to subsurface materials is contained in Section 2.6.1.6. Storage pad soil settlement analyses are described in Section 2.6.1.12.1.

The in situ soil is suitable for supporting the cask storage pads, but settlements are expected to occur. Analyses were performed to calculate the estimated settlement of the storage pads from the weight of the pad with 8 fully loaded casks in place. The nominal soil bearing pressure for this case is approximately 1.9 ksf, and the total estimated settlement of the pad is approximately 3.3 inches.

In order to accommodate the total estimated settlement, the storage pads will be constructed 3.5 inches above adjacent finished grade. Exposed edges of the pad will be chamfered, and the crushed rock surface materials will be feathered to meet the edges of the raised pads for transporter access.

Uniform downward settlement has no adverse effect on either the pad or the casks, it only lowers the final elevation of the storage pad. The storage pads will be constructed 3.5 inches above the surrounding grade to allow for settlements and yet maintain the surface drainage scheme at the site. The first pads constructed and loaded will be monitored for settlements to confirm the calculated settlements and make future adjustments, if necessary. The temporary uniform differential settlement of the pad from partial cask placements causes no loss of structural integrity to the pad. The storage pad is not susceptible to subsurface failures associated with liquefaction since the site is not subject to liquefaction, as discussed in Section 2.6.4.8.

4.2.3.5.4 Cask Stability

Cask stability ensures the storage casks will not tip over or slide excessively during a seismic event. The generic cask stability analyses in the HI-STORM SAR and TranStor SAR do not consider soil-structure interaction, which can amplify seismic accelerations. Consequently, site-specific cask stability analyses, performed by both Holtec and SNC (References 61 and 64) demonstrate the storage casks will not tip over or slide excessively during the PFSF design basis ground motion. The cask stability analyses are described in detail in Section 8.2.1. The cask storage pad is designed for the loads generated from the site-specific cask stability analyses and will provide the required support for the storage casks.

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4.3 AUXILIARY SYSTEMS

4.3.1 Ventilation and Offgas Systems

The canister-based storage technologies use a sealed (welded) canister design that precludes the need for ventilation or off-gas systems. No canisters will be opened at the site, therefore no ventilation system is required.

4.3.2 Electrical Systems

4.3.2.1 Major Components and Operating Characteristics

Normal electrical power will be provided to the PFSF via an upgraded 12.5 kV offsite distribution power line, which runs parallel to Skull Valley Road. A new electrical line will be constructed parallel to the site access road to furnish 12.5 kV to a 480 volt site transformer located at the site. The line will be run on new wooden power poles that will be installed by Utah Power & Light. Electrical power onsite downstream of the utility meter will be run underground and installed by contractors. The lines will either be underground service cable laid and buried in trenches or run in plastic conduit that is installed in underground concrete ductbanks per the National Electric Code (NEC) (Reference 67).

Step down transformers will be used to provide 480 and 120/240 volt services as required. No switching stations will be necessary. The normal power will be provided for lighting, general utilities, security system, HVAC loads, crane loads, and miscellaneous equipment. Cable size and power loading will be determined by the requirements in the NEC.

Emergency backup power is provided at the PFSF by a 480 volt diesel-generator. The emergency power supply is limited to the security system, emergency lighting loads, storage cask temperature monitoring system, and the site communications system. The diesel generator fuel supply is sized to provide continuous operation for a minimum 24 hour period per IEEE 692 (Reference 68). The diesel generator is located in the Security and Health Physics Building. A battery charger is provided with automatic and manual charge control to maintain fully charged diesel generator starting batteries when the unit is stopped.

An Uninterruptible Power Source (UPS) is utilized to support the security loads until the diesel starts and comes up to speed. The UPS system is a 120 volt, single phase system with integral batteries and battery charger. The UPS system is designed for a minimum of 1 hour operation without replacing or recharging batteries per IEEE 692. The UPS system is located in the Security and Health Physics Building.

4.3.2.2 Safety Considerations and Controls

In the event of a loss of offsite power, the UPS system is designed to automatically switch over to the battery source without loss of output voltage. When the diesel generator comes up to speed, the UPS automatically switches back to its normal source (which is then from the diesel generator) without loss of output voltage or battery recharge.

The diesel generator is provided with starting batteries maintained to supply sufficient capacity to consecutively crank the engine a minimum of five times. When the diesel generator starts, an automatic transfer switch transfers the security, emergency, and temperature monitoring loads to the generator when the diesel comes up to speed. Transfer back to normal offsite power takes place after the normal power is restored for a minimum of 30 minutes.

Electrical power is not classified as Important to Safety since the storage systems do not require electrical power for operation. In addition the cranes and operating equipment have been designed to maintain adequate safety provisions for handling spent fuel canisters in the event of a loss of power as discussed in Section 8.1.1.

In the event of a lightning strike, the most probable target is the 130 foot tall light poles that provide the lighting for the storage area. The light poles are metal and therefore act as a conductor. The poles are grounded to ensure that the current from a lightning strike is properly conducted to ground per the NEC.

4.3.2.3 Restricted Area Lighting

The lighting system will be designed to maintain a minimum lighting distribution of 0.2 foot-candles per 10 CFR 73.50 (Reference 69) throughout the Restricted Area (RA) such that sufficient lighting is provided to meet the following design objectives:

- Security of the site
- Safety of personnel and canisters
- CCTV to distinguish shapes, objects, and movement
- Human eye observation
- Lighting coverage of entire site per 10 CFR 73.51 requirements
- Minimize shadows around and under canisters
- Lighting of perimeter, double security fences, and the area immediately outside of this fence. These areas are the most critical for CCTV observation.

Note: Poles for site lighting cannot be located in close proximity to security fences thus eliminating a means to breach the security of the site. This results in the lighting for the perimeter, double security fences and the area immediately outside of this fence being more visible since they are aimed out to and past the Restricted

Area perimeter fence. This is minimized as much as possible during final, fine-tuning of the lighting installation.

The facility lighting system will consist of 130' mast lighting with 1000W HPS symmetrical patterned fixtures. These fixtures were chosen for efficiency and economy (they provide the greatest light distribution with the least number of fixtures).

Additional perimeter fence lighting is provided by 1000W and 400W HPS floodlights (with asymmetrical patterns) mounted at 130' for the 1000W fixtures and 40' for the 400W fixtures.

In three locations, 40' poles with a single luminare will be placed to provide lighting for roadway and parking facilities. These are 400W HPS fixtures and are aimed low in an effort to eliminate, horizontal glare (brightness) from the fixture.

4.3.3 Air Supply Systems

An air supply system is provided at the PFSF in the Canister Transfer Building and Operation and Maintenance Building for maintenance purposes. The system will be designed and installed in accordance with ASME B31.1 (Reference 70). There are no SSCs classified as being Important to Safety that require compressed air for operation.

4.3.4 Steam Supply and Distribution System

A steam supply system is not provided at the PFSF. The system will be designed and installed in accordance with ASME B31.1 (Reference 70). There are no SSCs classified as being Important to Safety that require steam for operation.

4.3.5 Water Supply System

A water supply system is provided at the PFSF for normal facility services and operation and maintenance functions. Water will be supplied using surface storage tanks fed from one or more wells drilled on-site. In the event that onsite water quantity or quality are inadequate, potable water will be obtained directly from the Reservation's existing supply or an additional well or wells will be drilled east of the site and outside of the OCA, where water supplies are likely to be more satisfactory. The water distribution piping and plumbing within the buildings will be provided in accordance with the Uniform Plumbing Code (Reference 25). There are no safety related SSCs classified as being Important to Safety that require water for operation.

4.3.6 Sewage Treatment System

A sanitary drainage system will be provided at the PFSF in accordance with the Uniform Plumbing Code (Reference 25) to transmit waste from the buildings to a septic system. The drainage lines will be installed underground and sloped to facilitate drainage.

Two septic tank and drain field systems will be provided to collect and process sanitary waste water from the facility. The systems will be located near the Security and Health Physics Building for the storage facility and near the Administration Building for the Balance of Facility. The systems will be sized for the maximum number of personnel expected on site during normal operating periods. The septic system is expected to process less than 5,000 gallons per day.

4.3.7 Communications and Alarm Systems

The communication systems consist of normal telephone service in all the buildings, a site public address system, and a short-wave radio system for security. The main telephone panel will be located in the administration building and will provide for 25 telephone lines. The service will be provided from the existing underground service located along the Skull Valley Road and will be routed underground parallel to the site access road. The telephone service will be used to provide normal communication to and from the site, emergency communications with local authorities, and on-site voice paging. The communication systems provide a means to contact the local law enforcement authorities for security purposes and for emergency responses on site in the event of an "ALERT", with notifications and follow-up communications.

In the event of an emergency, facility personnel and visitors on site are notified by an announcement over the onsite communications system (intercom). Offsite emergency response personnel are notified by means of personal pagers and/or using the notification list of telephone numbers located in the Emergency Plan implementing procedures. Alarms at the PFSF are only used on area radiation monitors to notify nearby personnel of doses that exceed the alarm setpoint.

Portable two-way radios are used by security personnel to maintain continuous communications with the Security and Health Physics Building while on patrol. The communication system is in accordance with proposed rule 10 CFR 73.51 (Reference 23).

4.3.8 Fire Protection System

4.3.8.1 Design Basis

Fires that could affect SSCs classified as Important to Safety are postulated to result from diesel fuel sources originating from the cask transporter or shipping cask transport vehicles (heavy haul tractor/trailer or railroad locomotive). SSCs affected include the storage casks in the yard and the shipping and storage system components and cranes

in the Canister Transfer Building. Scenarios for a fire in both locations considering fire location, intensity, and duration have been analyzed in Section 8.2.5. The analysis determined that the fires will not compromise the safety provisions of the SSCs.
No other major fire fuel sources are located in areas near SSCs classified as Important to Safety.

The Canister Transfer Building is constructed of noncombustible materials and is considered a Construction Type II – Fire Rated per the Uniform Building Code (UBC) (Reference 25). The building is designed to limit the potential effects from a diesel fuel fire with curbs and sloped floors located to contain spilled diesel fuel away from SSCs. The Canister Transfer Building is designed with a fire detection system to aid in the mitigation of fires. Portable fire extinguishers are located in the building and yard areas to facilitate fire suppression. The fire detection system is designed in accordance with NFPA 72E (Reference 24).

Fire zone classifications of the building are established in accordance with the UBC and NFPA 101 (Reference 66) and are shown on Figure 4.3-1. The Canister Transfer Building is classified with multiple purpose occupancy and is divided into 3 fire zones, which correspond to the specific occupant classifications.

Fire Zone 1 is classified as a Group H, Division 3 occupancy (hazardous area with a quantity of combustible liquids in excess of the exempt amounts listed in UBC Table 3-D that represent a high fire hazard). This zone consists of the transfer cells, crane bay, cask load/unload bay, and the cask transporter bay. The transfer cells and crane bay do not have any ignition sources however, the cask load/unload bay and cask transport bay house equipment containing diesel fuel from the heavy/haul tractor/trailer and the cask transporter respectively. The diesel fuel is a Class II combustible load. The cask transporter bay will contain up to 50 gallons of diesel fuel in the cask transporter, which is less than the exempt amount. The cask load/unload bay will contain up to 300 gallons of diesel fuel in the heavy haul tractor/trailer, which is more than the exempt amount of 120 gallons, thus the hazard classification. The canisters contain spent nuclear fuel, which is considered an "other health hazard." There are no limits on the quantity of "other health hazards" materials if stored or used in closed containers such as the canister.

The gross area of Fire Zone 1 is less than the maximum allowable area, above which would require a fire sprinkler system. However, to ensure the safest environment for SSCs, a water-foam sprinkler system will be installed in the cask load/unload bay where the potential exists for spillage of up to 300 gallons of diesel fuel. In addition, the cask load/unload bay floor is sloped to one of four sumps located at the ends of each bay and a 1 inch high threshold at the entrance into the transfer cell / crane bay area will be utilized to retain any spilled diesel fuel.

Another provision to ensure a safe environment for SSCs will be to design the walls and sliding doors between the canister transfer cells and the cask transporter bay as fire rated. The transfer cells walls as identified on Figure 4.3-1 will be 2-hour fire rated and the sliding doors will be 1 ½-hour fire rated to prevent any fire that could occur in the transporter bay from affecting an exposed canister during the transfer process.

No sprinklers are located near the transfer cells, which are considered a sprinkler exclusion area to avoid the possibility of spraying down a canister and dislodging possible contamination.

The crane bay, cask transporter bay, and transfer cells will contain fire extinguishers for fire suppression.

Fire Zone 2 is classified as a Group S, Division 1 occupancy (moderate hazard combustible material storage) and consists of the low level waste storage room. This area consists of storage containers (55-gallon drums) of ordinary combustibles that will be sealed and kept in the storage area. The gross area of Fire Zone 2 is less than the maximum allowable area containing closed containers, above which would require a fire sprinkler system. Therefore, the room will only use fire extinguishers for fire suppression.

Fire Zone 3 is classified as a Group B occupancy (business use) consisting of the office and building services areas of the building. The gross area of Fire Zone 3 is less than the maximum allowable area, above which would require a fire sprinkler system. Therefore, this zone will only use fire extinguishers for fire suppression.

The fire zones will be separated from one another by 1 hour rated fire barrier walls. Doors between fire zones will be ¾ hour fire rated doors.

The foam-water sprinkler system will be designed in accordance with NFPA 16 (Reference 65) and NFPA 13 (Reference 26). The fire pumps and water supply tanks will be provided in accordance with NFPA 20 (Reference 28) and NFPA 22 (Reference 29) respectively. The portable fire extinguishers will be provided in accordance with NFPA 10 (Reference 30). All fire protection systems will be designed to the latest code in affect at the time of the design.

4.3.8.2 System Description

The foam-water sprinkler system provided in the Canister Transfer Building load/unload bay will be fed water from one of two fire pumps at a fire pump house located outside the restricted area near the Security and Health Physics Building. The foam supply will be located immediately outside of the Canister Transfer Building where it will be connected to the water supply lines. Water for the pumps is supplied by a primary and a backup water tank. One pump is powered by an electric motor, the other by a diesel engine in the event of a loss of electrical power.

Fire hydrants are located near the buildings to support fire suppression of the buildings. The PFSF is served by at least one fire truck located at the site and one truck located at the Goshute Village 3.5 miles from the site to suppress fires that may occur around the site such as brush fires.

The fire detection system consists of photo-sensitive smoke detectors located in all the facility buildings. The smoke detectors are interconnected within each building and are connected to a central alarm panel located in the Security and Health Physics Building. Annunciation of the smoke alarms occurs within both the building where the detector is located and the central alarm panel. A trip of the fire detection system in the Canister Transfer Building will automatically set off the building's foam-water sprinkler system.

Smoke from a fire in the Canister Transfer Building will be removed by the building's ventilation exhaust fans.

4.3.8.3 System Evaluation

An evaluation of potential fires affecting SSCs classified as Important to Safety is shown in Section 8.2.5. The analysis concludes that these fires will not produce an unsafe condition or preclude the ability of SSCs from performing their safety related function. The foam-water sprinkler system further ensures that fires that could occur in the Canister Transfer Building load/unload bay will be extinguished within minutes.

4.3.8.4 Inspection and Testing Requirements

Preoperational and periodic operational testing and inspection of the fire detection and fire suppression systems will be performed in accordance with requirements of Section 9.2.

4.3.8.5 Personnel Qualification and Training

Training and qualification requirements associated with the testing, inspection, and operation of the fire systems will be in accordance with the requirements of Section 9.3.

4.3.9 Maintenance System

4.3.9.1 Major Components and Operating Characteristics

The PFSF has relatively few maintenance requirements because of the passive nature of the storage system's design. Major components at the PFSF that require routine periodic maintenance include the overhead bridge crane, semi-gantry crane, transfer equipment, and fire suppression system located in the Canister Transfer Building, the rail cars or heavy haul tractor/trailer units, the cask transporters, the backup diesel generator located in the Security and Health Physics Building, and the temperature monitoring equipment, fire pumps, and fire engine.

Periodic inspection and maintenance is also required to ensure the storage cask air ducts are not blocked from snow, dirt, debris, or small animal nesting per the operation controls and limits given in Chapter 10.

4.3.9.2 Safety Considerations and Controls

Routine maintenance procedures ensure that timely maintenance is performed according to equipment manufacturer's standards. The Operations and Maintenance Building is designed to facilitate activities performed on equipment and provide a safe environment. Ladders and platforms mounted on the walls and cranes in the Canister Transfer Building are used to access the cranes for maintenance and inspection activities. PFSF procedures prevent maintenance of the cranes or transfer equipment near casks loaded with spent fuel to minimize personnel radiation doses. Maintenance and inspection of the temperature monitoring system at the storage casks or the storage cask air vents are controlled by PFSF procedures to ensure that the work is performed ALARA.

4.3.10 Cold Chemical Systems

There are no chemical systems required or provided at the PFSF.

4.3.11 Air Sampling Systems

Since the spent fuel is totally contained within the canisters, there is no need for air sampling systems or airborne monitors except for the hand held monitor use to analyze the air sample taken from the shipping cask prior to being opened.

4.3.12 Gas Utilities

Propane will be used to provide fuel to all gas heating units located in the PFSF buildings rather than natural gas due to the remote location of the site. Propane for heating the Canister Transfer Building will be stored in one 2,000 gallon propane fuel storage tank, located inside the perimeter road and outside of the nuisance fence, near the southeast corner of the perimeter road approximately 450 ft east-southeast of the nearest point on the Canister Transfer Building. Propane for heating the Security and Health Physics Building will be stored in one separate and independent 1,000 gallon propane fuel storage tank, located inside the perimeter road and outside of the nuisance fence, approximately 450 ft east of the nearest point on the Canister Transfer Building and approximately 125 ft from the southeast corner of the Security and Health Physics Building. These two tanks will be separated from each other by at least 300 ft, which is considered more than sufficient distance to prevent a single projectile, such as a tornado-driven missile, from impacting both tanks. Both propane storage tanks are further than 1,000 ft from the nearest storage casks.

The storage tanks will be above-ground, designed in accordance with the requirements of NFPA 58. The effects of a postulated explosion involving 2,000 gallons of propane assumed to leak from the tank that supplies the Canister Transfer Building are analyzed in Section 8.2.4. Propane for heating the Operations and Maintenance Building and the Administration Building will be similarly stored, in propane tanks located near these structures. NFPA 58 requires that propane tanks between 50 and 2,000 gallon capacity be located at least 25 ft away from any building, adjacent container, or adjacent property. Since the amount of propane stored will be less than 10,000 lbs., no threshold levels that would invoke compliance with hazardous and toxic chemical regulations will be exceeded. The propane heating system will be installed in accordance with NFPA requirements. Outdoor piping between the tanks and the buildings will be located below ground and coated or wrapped.

4.3.13 Diesel Fuel Supply

In general, all fueling activities at the PFSF comply with applicable regulations. Operation and use of the stored fuel will be in accordance with 29 CFR 1910 (OSHA) regulations to ensure employee health and safety requirements are met. Prior to fueling, a management plan and procedures will be developed to ensure that personnel are properly trained and fuel deliveries are carried out in accordance with the plan.

4.3.13.1 Fueling of on-site vehicles used at the PFSF

As stated in SAR Section 8.2.4.1, a diesel fuel oil storage tank will be located inside the restricted area (RA), and will supply diesel fuel oil for the cask transporter. This tank will be located near the RA fence, approximately 200 ft northeast of the northeast corner of the Canister Transfer Building and approximately 700 ft from the nearest storage casks. The outdoor tank will be above-ground, mounted on a concrete pad, with a double wall, having all necessary equipment for pumping and dispensing diesel fuel. The tank will have a capacity of approximately 1000 gallons and will store low grade sulfur No. 2-D diesel fuel. The tank includes a double wall for primary and secondary spill containment requirements, fill and venting requirements, and fire prevention requirements in accordance with NFPA 30, "Flammable and Combustible Liquids Code." The tank will be designed in accordance with the requirements of UL-142, "Above Ground Tanks for Flammable and Combustible Liquids." The tank will also be designed in accordance with UL-2085, "Insulated Secondary Containment for Aboveground Storage Tanks, Protected." This code requires that the tank meet 2-hour liquid-pool furnace fire tests, vehicle impact, and projectile resistance criteria. The station tank will be supplied with fuel from a regional bulk fueling service.

4.3.13.2 Fueling of locomotives used on the Low Corridor Rail Line

The PFSF does not include an on-site diesel fuel storage tank for the locomotives. Rather, the locomotives at the PFSF are fueled outside the restricted area (RA) via a regional bulk fueling service that will deliver fuel to the PFSF approximately every two weeks with a tanker truck. Use of the fueling service will eliminate the need to store large quantities of fuel required for the locomotives near the PFSF as well as fuel station maintenance. The fueling service must comply with EPA and OSHA regulations and must provide containment and clean up for any spills in accordance with the regulations.

4.3.13.3 Fueling of heavy-haul vehicles used for the Intermodal Transfer Point

The heavy-haul vehicles will be fueled via a self-contained diesel fuel filling tank located near the Operations/Maintenance Building. The tank will be the same as the tank described above for the transporter vehicles and will meet the same criteria per NFPA 30, UL-142, and UL-2085 except that it will have a capacity of approximately 1200 gallons. The station tank will be supplied with fuel from a regional bulk fueling service.

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walls and doors, equipment lay-down areas, storage cask delivery and staging platform, mechanical and electrical equipment areas, and personnel offices and restroom areas.

4.7.1.4.1 Seismic Support Struts

The seismic support struts are rigid strut assemblies that secure the shipping and transfer casks to the Canister Transfer Building columns during transfer operations. The struts ensure that the casks will remain stable and will not topple in the event of an earthquake. The struts are designed to resist the horizontal forces due to the seismic accelerations developed in the seismic analysis of the building (Reference 62). The casks do not require seismic restraint in the vertical direction since the upward seismic forces are less than the deadweight of the casks.

The struts are connected to the shipping cask after it is moved into the transfer cell. Struts are also attached to the transfer cask when it is placed on top of the shipping cask or storage cask, prior to disconnecting the transfer cask from the crane. Each cask utilizes two struts, vertically positioned near the cask center of gravity, that provide lateral restraint in two orthogonal directions. Figure 4.7-7 is a schematic diagram of the support struts.

The support struts are procured as standard sway strut assemblies that conform to ASME III, NF requirements for Class 2 nuclear grade supports. The struts consist of a rigid tubular body with threaded eye rods on both ends. Each strut is pinned to a bracket that is secured to the cask and to the building columns. At the building columns, the brackets are welded to steel plate secured to the column with anchor rods.

4.7.1.5 Design Bases and Safety Assurance

The Canister Transfer Building is classified as being Important to Safety to provide the safety assurance commensurate with canister transfer activities. The design bases for the Canister Transfer Building are described in Chapter 3.

4.7.1.5.1 Structural Design

The building structure has been analyzed and critical areas have been designed for the critical loads cases. The final design of the Canister Transfer Building will be completed during the detailed design phase of the project. A preliminary design evaluation determined the worst loading case and areas of the structure. During the detailed design phase, all load cases as described in Chapter 3 and all areas will be addressed in detail. The rationale for selection of the worst loading case for the preliminary evaluation is provided below:

The load combinations for reinforced concrete design of the Canister Transfer Building, given in Section 3.2.11.4.1 are as follows:

a.) $U_c > 1.4\ D + 1.7\ L$

b.) $U_c > 1.4\ D + 1.7\ L + 1.7\ H$

c.) $U_c > 0.75(1.4\ D + 1.7\ L + 1.7\ H + 1.7\ T)$

d.) $U_c > 0.75\ (1.4\ D + 1.7\ L + 1.7\ H + 1.7\ T + 1.7\ W)$

e.) $U_c > D + L + H + T + E$

f.) $U_c > D + L + H + T + A$

g.) $U_c > D + L + H + T + W_t$

h.) $U_c > D + L + H + T + F$

i.) $U_c > D + L + H + T_a$

Based on a review of the above load combinations, it is obvious that combination b) includes and envelopes combination a) and combination d) includes and envelopes combination c). Loads A, F, and T_a are considered insignificant for the Canister Transfer Building (CTB) design as discussed below:

Accident Loads (A) - The accident loads to be considered for the PFSF, as discussed in SAR Section 3.2, are explosion overpressure, drop/tipover, accident pressurization, and fire. As discussed in SAR Section 8.2.4, the overpressure from a credible explosion is bounded by the differential design pressure of 1.5 psi due to a tornado. As discussed in SAR Section 8.2.6, a storage cask vertical drop from a height greater than 10 inches is not credible, nor is a storage cask tipover. A storage cask drop from a height of 10 inches would generate loads in a local area only and would not control the design of the 5-ft thick CTB foundation mat. As discussed in SAR Section 8.2.10, accident pressurization is not a credible accident and will not effect the design of the CTB. SAR Section 8.2.5 discusses a fire in the CTB. There are no loads from a fire that need to be considered in the structural design of the CTB.

Flood Loads (F) - As stated in SAR Section 3.2.11.4, flood loads are not applicable to this site.

Accident-level Thermal loads (T_a) – There are no accident level thermal loads applicable to the structural design of the CTB.

Since loads A, F, and T_a are insignificant for the Canister Transfer Building, the load combinations above can be reduced to:

b) $U_c > 1.4 D + 1.7 L + 1.7 H$

d) $U_c > 0.75\ (1.4\ D + 1.7\ L + 1.7\ H + 1.7\ T + 1.7\ W)$

e) $U_c > D + L + H + T + E$

g) $U_c > D + L + H + T + W_t$

Load combinations d) and g) address wind loads. The design basis tornado wind (W_t) is 240 mph and the design wind (W) is 90 mph. Since all the other loads in combinations d) and g) are minor or non contributors in the horizontal direction as compared to the wind loads, it can be seen that combination g) above, which includes the higher tornado wind loads, will be more critical than combination d), which only includes design wind loads.

Load combination b) only affects elements subjected to vertical loads. For the roof, assuming a 1 foot thick slab with a density of 150 psf and a 50 psf live load, the uniform load on the slab in load combination b) is calculated to be 1.4 (150 psf) + 1.7 (50 psf) = 295 psf. For load combination e) with the vertical acceleration of the roof being approximately 0.9 g, the uniform load is 150 psf + 50 psf + 0.9 (150 psf) = 335 psf. This demonstrates that for vertical loads, load combination e) will be more critical than load combination b). Therefore, the only two load combinations that could govern the design are:

e) $Uc > D + L + H + T + E$

g) $Uc > D + L + H + T + W_t$

Comparison of these two load combinations can be made on a global basis. Because of the larger exposed building area in the E-W direction, the horizontal load due to tornado wind will be greater in that direction. The exposed building area is approximately (270')(90') = 24,300 ft^2. The lateral force due to tornado wind is determined from NUREG-0800, Section 3.3.2 as:

Lateral force = 0.00256 V^2 x wall pressure coefficient x wall area

Where wall pressure coefficient = 0.8 (windward wall) + 0.5 (leeward wall)

Therefore, the lateral force = 0.00256 $(240)^2$ (0.8 + 0.5)(24,300 ft^2) = 4,658,135 lb. ≅ 4,658 kips. By comparison, the lateral force due to the design basis earthquake in the E-W direction is 36,500 kips. The earthquake force is taken from Calculation 05996.02-SC-5 (Reference 44) and excludes the force due to acceleration of the base mat.

Out of plane pressures on the exterior walls due to tornado loads are caused by the 240 mph wind velocity and the 1.5 psi pressure drop. The worst pressure is outward on the side walls and is equal to 0.00256 $(240)^2$ (0.7) + 1.5 psi (144 in^2/ft^2) = 319 psf. The out of plane seismic inertia load, based on a typical horizontal acceleration of 0.9 g results in an equivalent pressure for a 2 ft. thick wall of 2'(150 pcf)(0.9) = 270 psf. Although the pressure due to tornado is slightly higher than that due to seismic loads, the shear in the walls due to seismic are much greater and seismic loads will govern the design. For completeness, bending in the exterior walls due to tornado will be checked at the final design stage, and some additional reinforcing may be required in local areas. Effects of tornado missiles are addressed in the calculation for design of reinforcing steel for the CTB (Reference 47).

The Canister Transfer Building is a large and massive building consisting of exterior reinforced concrete walls 2'-0" thick, a reinforced concrete roof 1'-0" thick, and a solid reinforced concrete mat foundation 5'-0" thick. The interior partitions that make up the low level waste holding area will be constructed of concrete or concrete masonry. The equipment and office areas on the east side of the building will utilize steel framed partition walls covered with gypsum board. The total weight (static load) of the building and foundation is approximately 75,000 kips (Reference 44) or 37,500 tons.

The following provides verification that the site specific and operational criteria of the PFSF are enveloped by the Canister Transfer Building analysis and design.

A. Dead Loads

The Canister Transfer Building will be designed for the self weight of the structure and all permanently attached equipment.

B. Live Load

The Canister Transfer Building will be designed for the following live loads:

- Snow and ice loads - 45 psf per County Building Department exposure C, importance factor = 1.2 (Category IV) per ASCE-7
- Bridge crane and semi-gantry crane loads
- Normal crane handling loads and transfer operations Normal wind load - 90 mph, exposure C, importance factor = 1.15 (Category IV) per ASCE-7
- Vehicle loads (including impact loads)
  - Fully loaded cask transport vehicle
  - H20-44 truck per AASHTO
  - Heavy haul tractor/trailer
  - Rail car and prime mover
- Equipment Loads
  - Concrete storage cask (with loaded canister and transfer cask)
  - Transfer cask (with loaded canister)
  - Shipping cask (with loaded canister and transfer cask)

The overall seismic analysis of the building and foundation does not specifically include the additional weight of the shipping casks, transfer casks, and storage casks. However, an allowance of 5 percent of the mass of the mat was included in the lumped mass model to account for miscellaneous equipment and minor structural elements not discretely included in the mass calculations. The heaviest cask is a loaded concrete storage cask with a maximum weight of approximately 177 tons (Section 4.7.2.5.1). Although the loaded concrete storage casks are very heavy, each would equal only about 0.5 percent of the total mass of the structure. In addition, the casks will be located directly on the mat foundation and will have very little effect on the seismic response of the building itself.

The Canister Transfer Building is provided with three bays that are used for canister transfer operations. Shipping casks containing canisters will be moved immediately from the heavy haul tractor-trailer or rail car to the canister transfer bays. If the canister transfer bays are in-use, a maximum of two loaded shipping casks can be parked in the rail bays. Therefore, the maximum number of loaded casks within the entire building would be five at any one time (3 storage and 2 shipping). Empty shipping casks will be returned immediately or stored on the trailer or rail car outside of the Canister Transfer Building. There will be a maximum of four metal transfer casks (two for each cask vendor), but their weight is relatively insignificant, when not loaded.

For the design of the mat foundation, two worst-case load combinations were investigated. These are described in Section 4.7.1.5.3. Ground floor live loads (i.e., casks at various locations) were neglected in both of the load combinations considered. This is conservative because the maximum bending moments in the mat foundation occur at the intersection with the exterior walls, and are positive (tension on bottom face). The bending moments in the mat foundation away from the walls are negative (tension on top face). Application of live loads, including the weight of the casks, will result in bending moments that counteract the bending moments from these other critical load cases. Therefore, it is conservative to omit these loads in the analysis of the Canister Transfer Building mat foundation for the two load combinations considered. A calculation describing the mat foundation loading cases and designs is contained in Reference 46.

Crane loads will be increased to account for lateral and longitudinal impact forces.

C. Lateral Soil Pressure

Below grade portions of the Canister Transfer Building will be designed for loads from lateral soil pressure, including loads in excess of geostatic pressures resulting from the presence of adjacent surcharges or vehicular traffic.

D. Thermal Loads

The Canister Transfer Building will be designed to accommodate the site-specific extreme temperatures. Expansion joints will be provided as required to accommodate thermally induced movements in the structure.

E. Tornado Winds and Missiles

The Canister Transfer Building will be designed to protect all Important to Safety SSCs (see Chapter 3, Table 3.4-1) housed within the building from the effects of tornado winds and tornado-generated missiles. The Canister Transfer Building will be designed for the 240 mph wind speed and 1.5 psi pressure drop site specific design basis tornado event. The tornado wind speed will be converted to wind pressures in accordance with the provisions of ASCE-7 (Reference 31). Tornado wind and tornado pressure drop will be considered to act simultaneously. The worst case wind and pressure distribution acting on the structure as a whole and on individual building elements will be determined based on the physical size of the structure in relation to the size and characteristics of the design basis tornado. The structure will be designed to withstand the tornado wind and pressure drop by means of its static strength without the need to resort to venting of the structure.

The Canister Transfer Building will be designed to resist the effects of both horizontal and vertical impacts of the design basis tornado-generated missiles. Building components will be of sufficient strength and size to withstand the missile impact without compromising the strength and stability of the structure as a whole and to prevent penetration of the missile and spalling of the concrete face interior to the point of impact. The walls and roof that form the tornado missile barrier are shown in Figure 4.7-8. The building layout as well as specifically designed labyrinths will prevent tornado missiles from entering through door or ventilation openings in the walls and roof and potentially impacting or damaging the fuel canisters, single failure proof cranes and their supports, or other Important to Safety SSC's housed within the building.

F. Earthquake

The Canister Transfer Building has been analyzed for the PFSF design basis ground motion (0.53g horizontal, 0.53g vertical – See Section 3.2.10.1.1). The structure has been modeled and analyzed using a three-dimensional seismic analysis. The dead loads from the bridge and semi-gantry cranes will be located so as to produce the highest design loads and member stresses within the structure. Lifted loads from the cranes will be included in the seismic analysis. Results from the seismic analysis are used in the design of the building.

G. Fire

The postulated fire accident for the Canister Transfer Building is discussed in SAR Section 8.2.5. Since the Canister Transfer Building will be equipped with fire detection and suppression systems and be constructed of reinforced concrete, which has both a high thermal inertia and is inherently noncombustible, the postulated fire accident will have no effect on the structural strength or stability of the Canister Transfer Building structure as required per 10 CFR 72.122(c). Details of the fire protection system are discussed in SAR Section 4.3.8.

H. Lightning

The Canister Transfer Building is approximately 77 feet tall and is a possible lightning target. A lightning Risk Assessment performed in accordance with NFPA 780 determined that the Canister Transfer Building at the PFSF has a "moderate to severe risk factor." The risk assessment was based on the following criteria:

- The building houses the handling of hazardous materials
- The building construction consists of reinforced concrete w/ concrete roof
- The building extends more than 50 ft above adjacent structures or terrain
- The area topography is flat ground
- The building contains critical operating equipment
- The lightning frequency Isoceraunic level for the site location in Utah has 31 – 40 mean annual number of days with thunderstorms

Therefore the Canister Transfer Building will be designed with lightning protection features in accordance with NFPA 780. An air terminal lightning protection system will be installed on the building to protect the building from damage from a lightning strike. Air terminals will be erected on the ridge and perimeter of the upper roof and on the perimeter and interior of the lower roof areas. The air terminals will be interconnected to a main conductor cable that will provide a two-way path to ground for any of the terminals. The main conductor cable will be connected to down conductors that extend to ground rods around the perimeter of the building. All lightning protection materials will use NFPA 780 Class II materials since the building exceeds 75 ft in height. A lightning protection system as described above will ensure that lightning strikes will not prevent any SSCs that are important to safety from performing their safety function.

4.7.1.5.2 Shielding Design

The Canister Transfer Building is designed to provide radiological shielding during the transfer operations. A portion of the building is divided into canister transfer cells where the transfer operations are performed. The cells are surrounded by concrete shield walls that are designed to limit the radiation doses from the canister transfer operations to personnel outside of the cell to 2 mrem/hr, which is below the 5 mrem/hr dose level that establishes a "radiation area" per 10 CFR 20.1003. Large sliding doors for moving shipping and storage casks in and out of the cell are made of steel with a sandwich layer of neutron shielding. Personnel access openings into the cells are designed with a labyrinth of concrete to mitigate streaming of radiation.

A shielding analysis will be performed assuming canisters containing design basis fuel involved in canister transfer operations to determine transfer cell wall and cell door thickness requirements. The analysis will consider attenuation of the radiation doses through the shield walls and doors to locations outside the cell.

4.7.1.5.3. Structural Analysis

The preliminary design phase of the Canister Transfer Building included the conceptual drawings shown in the Figure 4.7-1 and design criteria identified in Chapter 3 and summarized in Table 3.6-1. The methodology and reference standards identified for use in the building seismic analysis is described in Section 3.2.10. Load combinations for the building design are shown in Section 3.2.11.4.

The detailed design phase of the Canister Transfer Building is based on the conceptual drawings and design criteria generated under the preliminary design phase. The first consideration in the detailed design was the selection of the critical load combinations. It was judged that the critical load cases would be those including the ISFSI design basis ground motion, since the building is subjected to high seismic loads and relatively low (Zone 3) tornado loads. A seismic analysis of the structure was performed to determine the seismic loads for the building design, and to generate in-structure response spectra for the design of the overhead bridge crane and semi-gantry crane, both supported on the Canister Transfer Building walls. The seismic analysis was performed following the guidelines of ASCE-4 (Reference 20). To perform the analysis, the first step was to develop three acceleration time histories (N-S, Vertical, and E-W) which are required to be consistent with the site ground response spectra and independent of one another. The time histories were developed from a near-source recording of the 1980 M 6.9 Irpinia, Italy normal-faulting earthquake. The original recordings were rotated in fault-normal and fault-parallel orientations and then scaled to match the 2,000-year return period design response spectra using both frequency domain (Reference 36) and time domain (Reference 37) approaches. The final time histories were then verified to meet the requirements of the Section 3.7.1 of the Standard Review Plan (Reference 38) and ASCE-4. The analysis is documented in Calculation 05996.02-G(PO18)-3 (Reference 39). The final time histories used in the seismic analysis of the Canister Transfer Building are shown in the calculation.

The building is founded on a layered soil medium, so it was necessary to consider soil-structure interaction effects. To accomplish this, the complex frequency method, as described in ASCE-4, was used. Impedance functions were developed to represent the subgrade, using the layered dynamic soil properties described in Calculation G(P018)-2 (Reference 40).

The impedance functions were developed, using the Stone & Webster computer program REFUND (Reference 41), by considering the foundation mat as a rigid structure located at the surface of the soil profile. These assumptions are appropriate since the building foundation is a five-foot thick concrete mat located at grade. Development of the impedance functions is documented in calculation SC-4 (Reference 42). A three-dimensional lumped mass model was developed to represent the structure. Lumped masses are assigned at the base mat (El. 95'-0"), the lower roof (El. 130'-0"), the crane elevation (El. 170'-0") and the upper roof (El. 190'-0"). Additional mass points were added at El 170'-0" to simulate local flexibility of the walls supporting the crane in the E-W direction and at El. 190'-0" to simulate the local flexibility of the roof in the vertical direction.

The impedance functions and the lumped mass model were combined, and the analysis was performed using the Stone & Webster computer program FRIDAY (Reference 43). The three input acceleration time histories were applied simultaneously as free field motions at the surface of the soil profile. Results of the analysis included displacement and acceleration time histories at each of the lumped mass points of the structural model. In-structure response spectra were developed from the acceleration time histories. The analysis was performed for three conditions, using best estimate, low range and high range soil properties. These soil properties were developed in Reference 40 to address possible uncertainties in the soil parameters and in the soil-structure analysis. The results of all three load cases were enveloped for worst-case conditions. The resulting enveloped in-structure response spectra were then peak broadened by +/- 15%. The zero period accelerations (ZPA) at each point of the lumped mass model and response spectra at El. 170'-0", which is the bridge crane support location are presented in the dynamic analysis described in calculation SC-5 (Reference 44).

The detailed analysis of the building was performed using the ANSYS computer program (Reference 45) with a 3-dimensional finite element model. First, a model of the soil was developed, extending 360 feet below the mat and approximately 360 feet to all sides of the mat. The soil is modeled with three-dimensional elastic solid elements, which were assigned properties consistent with the best estimate properties used in the seismic analysis. This model was condensed to a super-element that was coupled with the structural model. Compression-only elements were used to join the common nodes of the soil model and the base mat of the structural model. The structural model of the concrete building was developed from elastic plate elements (for slabs and walls) and elastic beam elements (for beams and columns). Initial wall and slab thickness and beam and column sizes were determined from hand calculations. Minimum wall thickness of two feet and roof thickness of one foot were selected based on tornado missile requirements. The typical size of the plate elements is five feet square.

Two critical load cases were considered. The first is that which produces the worst downward loading on the roof, and includes dead load, live load, and the vertical seismic load acting downward. The vertical seismic load is developed by applying as a static load the enveloped ZPA accelerations from the seismic analysis to the mass of the structure. Included in this load combination is 40% of the enveloped ZPA acceleration in each of the two horizontal (N-S and E-W) directions. This load combination governs the design of the roof, some of the walls, and portions of the base mat. The second load case was selected because it had the greatest overturning potential. It includes dead load, reduced live load, the enveloped E-W ZPA acceleration, 40% of the enveloped vertical ZPA acceleration upward, and 40% of the enveloped ZPA acceleration in the N-S direction. This load combination governs the design of portions of the base mat, crane support beams and some walls. Selected results of the analyses are presented in Figures 10 through 16. The finite element analysis, including the soil model and building model, is described in calculation SC-6 (Reference 46).

Results of the analysis were used to design the reinforcing steel for the concrete walls, slabs, beams and columns (pilasters). In general, the reinforcing required was not excessive. Highly stressed areas are in the roof slab, in the N-S walls where the roof beams intersect the wall, in the crane support beams, in the E-W shear walls, and in the corners of the base mat. The design of the reinforcing steel is described in calculation SC-7 (Reference 47).

Mat Foundation Stability Analyses and Settlement

In addition to the finite element, soil-structure interaction analysis described above, conventional static and dynamic stability analyses of the building mat foundation were performed. These included bearing capacity, overturning, and sliding stability analyses These analyses, performed in Calculation G(B)-13 (Reference 48), are described in detail in Section 2.6.1.12.2, and the results are discussed below. These analyses indicate that the building is stable and it will not be adversely affected by the estimated settlements.

The bearing capacity analyses were performed for the mat founded on a layered soil medium using both 'effective stress' and 'total stress' soil parameters for the various soil layers identified in the PFSF Storage Facility Design Criteria. Several load cases were considered, which consisted of combinations of vertical static, vertical seismic in upward and downward directions, and horizontal seismic in E-W and N-S directions. Loads developed in Calculation SC-5 (Reference 44) were used in these analyses. As in the structural analyses discussed earlier, seismic loads used were based on 100% of the enveloped ZPA acceleration in one direction, combined with 40% of the enveloped ZPA accelerations in each of the other two directions. Minimum factors of safety of 3.0 for the static load case and 1.1 for the seismic load cases are required against a bearing capacity failure of the foundation in soil. The load combination of full static, 40% seismic uplift, and 100% horizontal seismic in E-W, and 40% horizontal seismic in N-W direction was the most critical load case. This load case resulted in an actual soil bearing pressure of 2. 5 kips per square foot (ksf), compared with an ultimate bearing capacity of 4.3 ksf. The resulting factor of safety against a bearing capacity failure for this load case is 1.7, compared with the minimum allowable factor of safety for seismic loading cases of 1.1. For the static load case, a factor of safety in excess of 10 was obtained, exceeding the minimum required factor of safety of 3.0 by a wide margin.

A settlement analysis was performed for the Canister Transfer Building for the static dead and live loads. A total building settlement of 3.0 inches is estimated over the life of the building. The settlement will be generally uniform. Of the total building settlement, approximately 1.9 inches will occur within a few years after construction and an additional 1.1 inches over the life of the building. The settlement analysis is described in calculation G(C)-14 (Reference 49).

The sliding stability of the Canister Transfer Building is discussed in detail in Section 2.6.1.12.2. The Canister Transfer Building will be founded on clayey soils that have an adequate amount of cohesion to resist sliding due to the dynamic forces from the design earthquake. As shown in Figures 2.6-21 through 2.6-23, however, some of the soils underlying the building may be cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. Analyses were performed to address the possibility that sliding may occur along a deeper slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

Because of the magnitude of the dynamic forces resulting from the soil-structure interaction analyses, the factor of safety against sliding of this building would be less than 1 if it were founded on cohesionless soils. Where the factor of safety against sliding is less than 1, the displacements the building may experience were calculated using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes.

In these analyses, several conservative assumptions were made, and even with this high level of conservatism, the estimated relative displacement of the building ranged from 0.5 inches to 1.2 inches. Motions of this magnitude, occurring at the depth of the silty sand/sandy silt layer, would likely not even be evident at the ground surface. For the building to slide, a surface of sliding must be established between the horizontal sliding surface in the silty sand/sandy silt layer and through the overlying clayey layer. In the simplified model used to estimate these displacements, the contribution of this surface of sliding through the overlying clayey layer to the dynamic resistance to sliding motion is ignored, as is the passive resistance that would act on the embedded portion of the building foundation and the block of soil that is postulated to be moving with it. It is likely, moreover, that should such slippage occur within the cohesionless soils underlying the building, it would minimize the level of the accelerations that would be transmitted through the soil and into the structure. In this manner, these cohesionless soils would act as a built-in base shear isolation system. Any decrease in these accelerations as a result of this would increase the factor of safety against sliding, which would decrease the estimated displacements as well. Further, since there are no important-to-safety systems that would be severed or otherwise impacted by movements of this small amount as a result of the earthquake, such movements do not adversely affect the performance of the Canister Transfer Building.

4.7.2.2 Plans and Sections

The canister transfer bridge and semi-gantry cranes are shown in Figures 4.7-5 and 4.7-6 respectively.

4.7.2.3 Function

The function of the canister transfer cranes is to assist in the canister transfer operations at the PFSF. A description of the canister transfer operations is contained in Chapter 5.

The overhead bridge crane performs the following activities:

- Remove the impact limiters and personnel barrier from the shipping cask and move them to a laydown area, and
- Upright and remove the shipping cask from the rail car or heavy haul trailer and move the cask into a canister transfer cell.

The overhead bridge crane or the semi-gantry crane performs the following activities:

- Remove the lid from the shipping cask,
- Lift the transfer cask and place on top of the shipping cask, then lift the canister into the transfer cask,
- Lift the transfer cask containing the canister off the shipping cask and onto the top of the storage cask,
- Lower the canister into the storage cask, and
- Remove the transfer cask from on top of the storage cask and place the lid on top of the storage cask.

4.7.2.4 Components

The major components of the overhead bridge crane are the bridge, trolley, main hoist, and auxiliary hoist. The major components of the semi-gantry crane are the gantry frame, trolley, main hoist, and auxiliary hoist.

4.7.2.5 Design Bases and Safety Assurance

The canister transfer cranes are classified as being Important to Safety to provide the safety assurance commensurate with shipping cask and canister lifting activities. The design bases for the canister transfer cranes is described in Chapter 3. Each crane has sufficient capacity to lift the maximum lifted load the crane is designed for during transfer operations. Based on maximum weights presented by Holtec (HI-STORM SAR Tables 3.2.1 and 3.2.2, HI-STAR shipping SAR Table 7.1.1) and by SNC (TranStor SAR Table 3.2-1 and TranStor shipping SAR Table 2.2-1), the maximum lifted loads are addressed in the following Sections.

Since the cranes are classified as Important to Safety, they must be capable of performing their intended functions under all loading conditions including off-normal and accident conditions.

The failure of a crane during canister transfer operations is discussed in SAR Section 8.1.1.3, which shows that the cranes will not drop their loads under off-normal conditions.

The crane operations are designed not to exceed the handling loads (live loads) assumed in the HI-STORM and TranStor SARs. SAR Section 8.1.4.3 assumes an off-normal handling load is generated from a 2 fps horizontal impact. The crane design parameters limit the high speed of the trolley to less than 60 fpm (1 fps).

SAR Section 8.2.1.2 shows that the cranes maintain their structural integrity and functionality under seismic conditions. However, it is not a design requirement that the crane be operable during an earthquake nor that it be operable after an earthquake. The following is mandatory:

a) The crane bridge (gantry) and trolley are provided with suitable restraints so that they do not leave their rails during an earthquake.
b) No part of the crane shall become detached and fall during an earthquake.
c) The crane load shall not lower in an uncontrolled manner during or as the result of an earthquake.

Additionally, the crane design specification requires that the crane design include the ability to manually release the hoist, emergency, bridge, gantry, and trolley brakes to allow for controlled lowering and positioning of the load in the event of an emergency.

4.7.2.5.1 Maximum Loads Applicable to the Overhead Bridge Crane

The weight of loaded shipping cask, impact limiters, cask support cradle, and personnel barrier is approximately 142 tons (HI-STAR system) and 138 tons (TranStor system).

The weight of loaded shipping cask and shipping cask lifting yoke is approximately 121 tons (HI-STAR system) and 118 tons (TranStor system).

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associated mechanical equipment. The trolley load girt is of welded plate box construction and directly supports the main hoist upper block reactions.

The semi-gantry crane is designed with double bridge girders spanning 35 ft supported along one end on rails 55 ft above the building floor and with gantry legs mounted on rails at the other end. The bridge girders are welded plate box sections rigidly connected to box section end ties, which are pinned to the bridge trucks to equalize the load to each truck at the wall supported end and rigidly connected to the gantry legs at the gantry end. The gantry legs connect to the bridge trucks at the floor through a load equalizing end tie. The gantry legs are constructed of welded plate box sections, which taper from the girder end tie connections to the equalizing sill connections. The bridge trucks are rigid box structures, each enclosing two 30 inch diameter wheels, connected with pins at each end of the equalizing sill. The trolley spans 15 ft and is supported from rails mounted along the bridge girder centerlines. The trolley consists of 2 box section end trucks with 2 wheels each, which are rigidly connected at the midspan with a load girt. A deck plate across the top of the trucks and load girt is used for mounting the rope drums, hoist motors and brakes, the upper blocks, and other associated mechanical equipment. The trolley load girt is of welded plate box construction and directly supports the main hoist upper block reactions.

The main hoist for both cranes use a 16 part reeving configuration allowing two independent wire ropes to wind simultaneously on the hoist drum. Each rope supports the lifted load with a force of 1/16 of the payload weight. The 25 ton auxiliary hoist uses a similar 8 part reeving configuration. The bridge crane main hoist utilizes a 1 5/8 inch diameter rope and the semi-gantry crane main hoist utilizes a 1 3/8 inch diameter rope.

The crane uses a festooned cable system. The cable is fixed to the trolleys and a strain system ensures that wear through sharp cable bends and direct strain on connections is minimized.

All hooks are forged carbon steel and are designed with a 10 to 1 safety factor "sister" type with pin hole and safety latches. Each hook is mounted on a load bearing trunnion separate from the rope sheave axle and swivels freely on an antifriction thrust bearing. To ensure safe and smooth transitions when connecting or disconnecting lift beams, both cranes use main and auxiliary hooks of the same size and dimensions.

The reeving arrangements of the wire rope systems are redundant and balanced so that failure of one rope system does not cause significant lateral motion or energy at the load block.

The seismic analysis indicated no uplift from a seismic event on either the bridge crane or the semi-gantry crane. However, the cranes are designed with lateral restraints that consist of side bars mounted next to the crane rails. The side bars prevent any lateral movement of the bridge wheels and therefore, prevent the wheels from leaving the rails.

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69. 10 CFR 73.50, Requirements for Physical Protection of Licensed Activities.

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TABLE 4.2-6

SUMMARY OF TRANSTOR SYSTEM THERMAL HYDRAULICS EVALUATION (°F)

CASE	AMBIENT AIR		OUTER	INNER	CANISTER	MAX CLAD**	
	INLET	OUTLET	CONCRETE	CONCRETE	SHELL	PWR	BWR
Normal Condition Generic Limits	N/A	N/A	150	200	N/A	621	673
Steady-State Normal Condition Storage	75	171	85	188	274	613	664
Off-normal Condition Generic Limits	N/A	N/A	150	225	N/A	1058	1058
Steady-State Severe Cold	-40	44	-19	50	181	557	521
Steady-State Severe Hot	100	210	132	221	314	657	636
½ Inlet Ducts Blocked	75	194	103	200	298	645	622
Accident Short-Term Condition Generic Limits	N/A	N/A	200	350	N/A	1058	1058
Extreme Hot Ambient Temperature (12 hours max.)	125	237	157	249	337	675	657
All Inlet Ducts Blocked	75	N/A	108	348	468	778	774

** Based on the highest burnup and shortest cooling time of all the fuels considered in the TranStor SAR and is therefore conservative for PFSF fuel.

TABLE 4.2-7

STATIC PAD ANALYSIS MAXIMUM RESPONSE VALUES

LOADING CONDITIONS		MAXIMUM MOMENT (k-ft/ft)	MAXIMUM SHEAR FORCE (k/ft)	MAXIMUM SOIL PRESSURE (k/ft^2)
Dead Load		0.0	0.0	0.45
Live Load	2 Casks	73.2	7.9	1.81
	4 Casks	66.8	8.7	2.22
	8 Casks	53.0	6.4	2.05
	8 Casks + Transporter	50.3	11.3	3.60

Notes:

1. Values for maximum moment and shear taken from Reference 16 (page 51).
2. Values for maximum soil pressure taken from Reference 16 (page 235 and 236) and include the weight of the storage pad.

TABLE 4.2-8

DYNAMIC PAD ANALYSIS MAXIMUM RESPONSE VALUES

(based on PSHA design basis earthquake – See Section 8.2.1.1)

PSHA DESIGN BASIS EARTHQUAKE LOADING	MAXIMUM MOMENT (k-ft/ft)	MAX. SHEAR FORCE (k/ft)	MAXIMUM SOIL PRESSURE (k/ft^2)	MAX. HORIZONTAL TOTAL SOIL REACTION (kips) (Y-direction)	MAX. HORIZONTAL TOTAL SOIL REACTION (kips) (X-direction)
2 Casks	224.9	49.9	2.22+0.45DL	540	681
4 Casks	144.2	33.1	1.97+0.45DL	923	1,036
8 Casks	335.5	68.6	3.92+0.45DL	1,791	1,855

Notes:

1. Values for maximum moment and shear taken from Reference 16 (page 51).
2. Values for maximum soil pressure taken from Reference 16 (page 241).
3. Values for maximum horizontal total soil reaction taken from Reference 16 (pages 238 and 239).

TABLE 4.7-1

PHYSICAL CHARACTERISTICS OF THE HI-TRAC TRANSFER CASK

PARAMETER	VALUE
Inside Diameter	68.75 inches
Outside Diameter	94.625 inches
Height	203.50 inches
Materials of Construction	Steel (inner and outer shell) Lead (gamma shield) Water (neutron absorber)
Weight (empty)	152,636 lb
Maximum Working Dose Rate[1] (1 meter from surface) Side	 42 mrem/hr

1. Dose rates are based on HI-TRAC design basis zircaloy clad fuel for normal conditions.

FIGURE WITHHELD AS SENSITIVE
UNCLASSIFIED INFORMATION

Figure 4.3-1

CANISTER TRANSFER BUILDING
FIRE ZONES & BARRIERS

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

FIGURE WITHHELD AS SENSITIVE
UNCLASSIFIED INFORMATION

Figure 4.7-8

CANISTER TRANSFER BUILDING
MISSILE BARRIERS

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

CHAPTER 7

RADIATION PROTECTION

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- The location of the Canister Transfer Building inside the RA minimizes the route between the handling facility and storage pads, provides for minimal other traffic on the route, and maintains substantial distance from the OCA boundary.

- There are no radioactive liquid wastes associated with the PFSF.

As shown in Section 7.3.3.5, the design of the PFSF assures that dose rates at the OCA fence are sufficiently low that individuals at the fence will not exceed 25 mrem per year whole body dose, in compliance with the requirements of 10 CFR 72.104.

The PFSF building ventilation systems are not designed for any special radiological considerations since there is no credible scenario for which a significant radioactive release could occur. Shielding of the canisters is provided by the storage casks and by the shipping and transfer casks during canister receipt, transfer and, offsite shipping operations. Shielding is provided in the design of the Canister Transfer and the Security and Health Physics Buildings for additional radiation dose protection.

The general area inside the RA fence is a restricted area, as defined by 10 CFR 20, and will be controlled in accordance with applicable requirements of 10 CFR 20, with personnel dosimetry required. Certain areas within the RA will be designated as Radiation Areas, and specific locations within the RA have the potential to be High Radiation Areas, and will be posted and controlled in accordance with applicable requirements of 10 CFR 20. The cask load/unload bay, crane bay, cask transporter bay, and canister transfer cells inside the Canister Transfer Building will be designated as Radiation Areas whenever loaded canisters are present in these areas, since the potential exists for dose rates to exceed 5 mrem/hr in these areas. Upon removal of the impact limiters from the shipping casks in the cask load/unload bay of the Canister

Transfer Building, the potential exists for dose rates in the vicinity of the top and/or bottom of the casks to exceed 100 mrem/hr in localized areas, and these localized areas will be posted as High Radiation Areas, with necessary controls applied. The external walls of the Canister Transfer Building, adjacent to the east, south, and west sides of the cask load/unload bay, are 2 ft thick concrete, with steel roller bay doors in the truck/rail entrance/exits at the east and west ends of the bay. Due to distances from the shipping casks when their impact limiters are removed, dose rates outside the Canister Transfer Building will be well below 100 mrem/hr. The concrete walls of the cask load/unload bay, and steel roller bay doors, will reduce dose rates outside the building to levels as low as is reasonably achievable.

It is anticipated that the canister transfer cells within the Canister Transfer Building (Figure 4.7-1) will be posted as High Radiation Areas during canister transfer operations, since the dose rates in the cells could potentially exceed 100 mrem/hr in localized areas 30 cm from cask surfaces. Due to distances between cask surfaces and the crane bay, cask transporter bay, and areas external to the Canister Transfer Building, dose rates will be well below 100 mrem/hr without credit for the shield walls that surround the canister transfer cells. The north wall of cell no. 1 (an external wall), and the west cell walls of all three cells (adjacent to the cask transporter bay), will be 2 ft thick concrete. The walls between the cells, the south wall of cell no. 3, and the east walls of all three cells, will be 1 ft thick concrete. The sliding doors will be steel with a polyethylene (or similar) shield, as necessary, to minimize neutron doses. The walls and doors provide radiation shielding that will limit the dose rates outside of the canister transfer cells during transfer operations to as low as is reasonably achievable.

The east wall of the crane bay is 2 ft thick concrete, and it is expected that dose rates in the rooms and offices east of this wall will be less than 5 mrem/hr, even when shipping cask movements and canister transfer operations are in progress, and will not require posting as Radiation Areas. Dose rates in the vicinity of low level waste storage containers are expected to be insignificant, due to the relatively low quantities of radioactivity that will be stored in this area resulting from incidental cleanup of any contamination. Nevertheless, the 2 ft thick concrete north and east walls, and 1 ft thick concrete south and west walls of the Low Level Waste Room will assure that dose rates outside this room are as low as is reasonably achievable. No credit is taken for shielding by other walls of the Canister Transfer Building.

7.3.2 Access Control

The PFSF is designed to provide access control in accordance with 10 CFR 72. Access control to the RA is provided for both personnel radiological protection and facility physical protection. The physical protection program is covered in the Security Plan, which is classified and submitted as part of the License Application under separate cover.

The access control boundaries for the controlled and restricted areas are established along the site fence lines (see Figure 1.1-2, the PFSF Site Plan). The RA is that space which is controlled for purposes of protecting individuals from exposure to radiation or radioactive materials and for providing facility physical security. The boundary for the RA is the security fence where the dose rate is less than 2 mrem/hr, in accordance with 10 CFR 20.1301. The controlled area is the area inside the site boundary (delineated by the OCA fence). The dose rate beyond the OCA fence is less than 25 mrem/yr, in accordance with 10 CFR 72.104.

Access to the RA is controlled through a single access point in the Security and Health Physics Building (see Figure 1.2-1, the PFSF General Arrangement). Personal dosimetry is issued and controlled in this building to individuals entering the RA. Provisions exist in this building for donning and removing personal protective equipment, such as anti-contamination clothing and/or respirators, which could be necessary in the event of contamination in the Canister Transfer Building as a result of off-normal or accident conditions. Provisions for personnel decontamination are also contained in the Security and Health Physics Building. The RA also includes the cask storage area and Canister Transfer Building. In accordance with the PFSF Radiation Protection Program (Section 7.5), radiation protection personnel will monitor radiation levels in the RA and establish access requirements as needed.

7.3.3 Shielding

The storage systems are designed to maintain radiation exposures ALARA. The HI-STORM storage cask design objectives specified in Section 2.3.5.2 of the HI-STORM SAR are maximum contact dose rates of 35 mrem/hr on the side, 10 mrem/hr at the top, and 50 mrem/hr at the air vents. The TranStor design dose limits specified in Section 2.3.5.2 of the TranStor SAR are 15 mrem/hr 1 meter from the side of a TranStor storage cask (30 mrem/hr for stainless steel clad fuel) and 200 mrem/hr 1 meter above the center of the cask cover lid.

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CHAPTER 8

ACCIDENT ANALYSIS

In the preceding chapters, the design and operational features of the PFSF storage and handling systems that are classified as Important to Safety were identified and discussed. This chapter provides a description of the analyses performed for off-normal operating conditions and for a range of hypothetical accidents. The evaluations of off-normal events and accidents demonstrate that the PFSF structures, systems and components (SSCs) classified as Important to Safety are capable of performing their required functions for a wide range of postulated conditions satisfying the requirements of 10 CFR 72.122(b).

ANSI/ANS 57.9 (Reference 1) defines four categories of design events that establish the requirements to satisfy operational and safety criteria. A Design Event I is associated with normal operation. Design Event I conditions are addressed in Chapter 4. A Design Event II is associated with off-normal operations that can be expected to occur with moderate frequency, or on the order of once during a calendar year of PFSF operation. The Design Event II conditions are described in Section 8.1. A Design Event III is associated with infrequent events that could be reasonably expected to occur during the lifetime of the PFSF. These are described in Section 8.2. A Design Event IV is associated with plant-specific design phenomena, including natural phenomena and man-induced low probability events. These are also described in Section 8.2. Section 8.4 provides a discussion of the basis for selection of off-normal and accident conditions that are evaluated in this chapter.

The conservative nature of the assumptions and methods used in the analyses of off-normal and accident conditions represent an upper bound for the PFSF design basis events. The analyses demonstrate that the PFSF satisfies the applicable design criteria and regulatory limits. Therefore, the reported values of parameters, such as temperatures and stress levels, envelope the values that would actually be experienced for the various postulated accident conditions.

The results of the off-normal and accident analyses described in this chapter are based on analyses documented in more detail in the HI-STORM 100 Cask System SAR (Reference 2) and the TranStor Storage Cask System SAR (Reference 3).

8.1 OFF-NORMAL OPERATIONS

This section addresses events designated as Design Event II as defined by ANSI/ANS-57.9. The following are considered off-normal events:

- Loss of external electrical power,
- Off-normal ambient temperatures,
- Partial blockage of storage cask air inlet ducts,
- Operator error, and
- Off-normal contamination release.

There is no release of radioactive fission products from inside the canister or abnormal radiation levels associated with these off-normal operations. The only calculated consequence arises from the postulated release of surface contamination from the canister exterior, as discussed in Section 8.1.5. The resultant committed effective dose equivalent (CEDE) and the committed dose equivalent (CDE) to the maximally exposed organ at the Owner Controlled Area (OCA) boundary are shown to be less than 0.1 mrem in Section 8.1.5.3, well below the 10 CFR 72.106 criteria of 5 rem for accidents. Assuming an off-normal condition resulting in release of contamination to the atmosphere occurs on the order of once per year, total annual dose consequences at the OCA boundary from this event and radiation emanating from storage casks (Section 7.6) will not exceed 25 mrem, in accordance with 10 CFR 72.104.

8.2 ACCIDENTS

Design events of the third and fourth types as defined in ANSI/ANS-57.9 are considered in this section. A Design Event III consists of those infrequent events that could reasonably be expected to occur during the lifetime of the PFSF. A Design Event IV consists of natural phenomena and human-induced low probability events that are postulated because their consequences may result in the maximum potential impact on the immediate environs but are not necessarily credible. Hypothetical accidents, which are analyzed in this section, are also considered as Design Event IV. Their consideration establishes a conservative design basis for SSCs classified as important-to-safety.

The following accident or class III and IV design events are considered in this chapter:

- Earthquake,
- Extreme wind,
- Flood,
- Explosion,
- Fire,
- Hypothetical storage cask drop / tip-over,
- Canister Leakage Under Hypothetical Accident Conditions,
- 100% blockage of air inlet ducts,
- Lightning,
- Hypothetical accident pressurization, and
- Extreme environmental temperature.

Each of these accidents are described in the following sections. These evaluations show that the release of radioactive material is controlled in compliance with 10 CFR 72.106 and 72.126(d).

8.2.1 Earthquake

An earthquake is classified as a natural phenomenon Design Event IV as defined in ANSI/ANS-57.9.

8.2.1.1 Cause of Accident

Earthquakes are associated with faults in the upper crust of the earth's surface. Earthquake magnitudes and associated ground motions in Utah are based on historical and pre-historic data and are contained in maps and tables as referenced in Section 2.6. The PFSF is located west of the Rocky Mountain Front (approximately 104° west longitude) as described in 10CFR Part 72.102 and the site area has the potential for seismic activity. Consequently, the site has been evaluated for geological and seismological characteristics to determine the appropriate seismic design criteria (Sections 2.6 and 3.2.10). SSCs classified as Important to Safety are required to be designed to resist the effects of the design basis ground motion in accordance with the requirements of 10CFR 72.122(b).

In the original license application submittal, a PFSF site specific earthquake was calculated for the PFSF site using the deterministic methodology of 10 CFR 100 Appendix A. This earthquake was characterized by response spectrum curves developed specifically for the site with a zero period acceleration of 0.67 g horizontal (two directions) and 0.69 g vertical. The response spectrum curves for the PFSF original site specific deterministic design earthquake are documented in Reference 28.

The regulations for siting nuclear power plants (10 CFR 50 Appendix S and 10 CFR 100.23) were amended in 1997 to allow the use of the probabilistic seismic hazard assessment (PSHA) methodology in order to recognize the inherent uncertainties in geologic and seismologic parameters that must be addressed in determining the

8.2.1.3 Accident Dose Calculations

The PFSF design basis ground motion is not capable of damaging the canisters or storage casks during canister storage operations. While the HI-STORM storage cask was explicitly analyzed for and shown to withstand the PFSF design basis ground motion, the TranStor storage cask was analyzed for and shown to withstand the PFSF deterministic design earthquake. Since accelerations associated with this seismic event are significantly higher than those associated with the PFSF design basis ground motion (Section 8.2.1.1), the TranStor canisters and storage casks will also safely withstand the design basis ground motion. The Canister Transfer Building structure is designed to withstand the PFSF design basis ground motion. Additionally, the overhead bridge crane, semi-gantry crane, and canister downloader are designed to comply with the single-failure-proof criteria, which requires them to withstand the PFSF design basis ground motion with the maximum critical load in the lifted position during the seismic event, without dropping the load (Section 3.2.10.2.10). No radioactivity would be released in the event of an earthquake and there would be no resultant dose.

8.2.2 Extreme Wind

The extreme design basis wind is derived from the design basis tornado. Extreme wind is classified as a natural phenomenon Design Event IV as defined in ANSI/ANS-57.9.

8.2.2.1 Cause of Accident

Extreme winds due to passage of the design tornado, defined in Section 3.2.8, are postulated to occur as a severe natural phenomenon.

8.2.2.2 Accident Analysis

The site is located in tornado Region III as defined in Regulatory Guide 1.76 (Reference 15). The design basis tornado loading for this region is defined as a tornado with a maximum wind speed of 240 mph and a 1.5 psi pressure drop occurring at a rate of 0.6 psi/sec, including the effects of postulated Spectrum I or II tornado generated missiles that could be created by the passage of the tornado as identified in Section 3 of NUREG-0800 (Reference 16).

Storage Casks

The HI-STORM and TranStor storage systems are designed to withstand loads associated with the most severe meteorological conditions, including extreme winds, pressure differentials, and missiles generated by a tornado. Results of the evaluation of effects of a tornado on the HI-STORM and TranStor storage systems are described in their SARs (References 2 and 3, respectively). Both storage systems are designed to the design basis tornado criteria for tornado Region I (Maximum wind speed of 360 mph and 3.0 psi pressure drop occurring at a rate of 2.0 psi/sec), which substantially envelopes the Region III criteria for the PFSF.

The HI-STORM and TranStor SARs demonstrate that the 360 mph wind loading on the cask area produces insufficient forces to tip over the casks. Spectrum I missiles are assumed to impact a storage cask in a manner that produces maximum damage. The combination of tornado winds with the most massive Spectrum I missile, a 3,968 lb (1,800 kg) automobile traveling at 126 mph, was also evaluated in accordance with Section 3 of NUREG-0800. The wind tipover moment was applied to the cask at its maximum rotation position following the worst-case missile strike. Calculations presented in the HI-STORM and TranStor SARs determined that the restoring moment far exceeded the overturning moment and the storage casks would not tip over.

While the calculations demonstrate that design missiles could not cause the storage casks to tip over, they could inflict localized damage. The HI-STORM and TranStor SARs demonstrate that none of the Spectrum I design missiles are capable of penetrating the storage cask and striking the canister, and canister confinement would not be affected. However, design missiles could cause a localized reduction in shielding. SNC calculated worst case damage to a TranStor storage cask of 5.69 inch deep penetration from the 8 inch diameter design missile (TranStor SAR Section 11.2.3). The TranStor and HI-STORM SARs conclude that while tornado missiles could cause localized damage to the radial shielding of the storage casks resulting in increased dose rates on contact with the affected area, the damage will have negligible effect on the dose at the OCA boundary.

Based on the above, the HI-STORM and TranStor storage systems meet the general design criteria of 10 CFR 72.122(b), which states that SSCs classified as Important to Safety must be designed to withstand the effects of tornadoes without impairing their capability to perform safety functions. Since tornado winds and tornado generated missiles do not have the capability to damage the canister, a tornado strike on or about loaded storage casks will not result in a release of radioactivity.

Canister Transfer Building

The Canister Transfer Building shields and protects the SSC's housed within it and the canister transfer activities, which take place inside, from the effects of severe natural phenomena. The Canister Transfer Building is designed to withstand the effects of the Region III design basis tornado wind and pressure drop forces, as well as the effects of Spectrum II tornado missiles as defined in Regulatory Guide 1.76 and Section 3 of NUREG-0800 (see Section 3.2.8).

The building provides this protection by means of thick reinforced concrete walls and roof of sufficient strength to withstand the design basis wind, pressure drop, and missile forces. Additional missile protection is provided by the interior reinforced concrete walls and missile / shielding doors and/or labyrinths.

8.2.2.3 Accident Dose Calculations

Extreme winds in combination with tornado-driven missiles are not capable of overturning a storage cask nor of damaging a canister within a storage cask. The Canister Transfer Building is designed to withstand wind forces and missiles associated with the Region III design basis tornado, protecting canister transfer operations from the effects of tornadoes. Therefore, no radioactivity would be released in the event of a tornado. Dose rates at the OCA boundary would not be affected by damage to storage casks from tornado-driven missile strikes .

It is assumed that it would take two technicians 30 minutes to repair the worst case damage resulting from impact to a TranStor storage cask by a design missile (a penetration 5.69 inches deep and 8 inches in diameter), by filling the damaged area with grout. SNC shielding calculations (TranStor SAR Sections 5.4.8 and 11.2.3) predict surface dose rates of less than 300 mrem/hr in the center of the damaged area for all design fuel cases for a concrete cask with 5.69 inches of concrete removed. A

8.2.4 Explosion

Explosion is classified as a human-induced Design Event IV as defined in ANSI/ANS-57.9.

8.2.4.1 Cause of Accident

Potential for Offsite Explosions

Section 2.2 "Nearby Industrial, Transportation and Military Facilities", indicates that the only facility which could contribute to the potential for significant explosions located within 5 miles of the PFSF is the Tekoi Rocket Engine Test facility. There are no chemical processing plants, petroleum refineries, natural gas facilities, or munition depots that could contribute to the potential for significant explosions located within 5 miles of the PFSF. The Tekoi Test facility is located approximately 2.5 miles south-southeast of the PFSF. This facility is used periodically to test rocket engines mounted on stationary bases and to test explosives. As discussed in Reference 57, the Tekoi test site encompasses two operational areas, the high hazard explosive test area and the static test range. The static test range consists of three bays. Bay 1 is used for machining of large rocket motors containing Class 1.1 propellant. Bays 2 and 3 are used for static testing of full scale rocket motors of both explosive Class 1.1 and 1.3 propellants. The high hazard explosive test site tests all classes of explosives and intentional detonations are an inherent part of the testing. Although the high hazard test area has an explosive limit of 200 lbs. Class 1.1 explosive, this limit can be increased if necessary for a specific test (Reference 57). Bay 1 and Bay 2 of the static test area have explosive limits of 100,000 lbs and 50,000 lbs Class 1.1 explosives, respectively. The worst case explosion potential is from Bay 3, which has the highest approved explosive limit of 1.2 million pounds of Class 1.1 explosive material (Reference 57). Bay 3 is also the closest bay to the PFSF, at a distance of 2.3 miles.

Hickman Knolls, with an elevation of approximately 4,873 ft, is situated directly between the PFSF (elevation 4,465 ft) and the Tekoi Test facility (elevation approximately 4,600 ft). Overpressures resulting from the Tekoi Test facility would decay substantially prior to reaching the PFSF due to the greater than 2 miles intervening distance, and would not produce significant overpressures at the PFSF as discussed in the following section.

The northern perimeter of the Dugway Proving Grounds is approximately 9 miles from the PFSF and the Tooele Army Depot (south area) is approximately 21 miles from the PFSF. There is no interstate highway, railroad (other than the rail which may be installed specifically for shipments of spent fuel shipping casks to and from the PFSF), or river traffic within the vicinity of the PFSF. The nearest interstate highway and commercial rail line are about 24 miles to the north of the facility. The Skull Valley Road, which runs north and south through the Skull Valley Indian Reservation to the east of the PFSF and provides entrance to the site access road, is 1.9 miles from the Canister Transfer Building and 2.0 miles from the nearest storage pad. The worst-case explosion potential at the PFSF is considered to be from an accident associated with the transportation of explosives along the Skull Valley Road (elevation approximately 4,580 ft, with no obstacles intervening between PFSF).

Potential for Onsite Explosions

A diesel fuel oil storage tank will be located inside the RA, and will supply diesel fuel oil for onsite vehicles, including the cask transporter. This tank will be located near the RA fence, approximately 200 ft northeast of the northeast corner of the Canister Transfer Building and approximately 700 ft from the nearest storage casks. A double-wall subbase diesel fuel oil tank will be mounted on the backup diesel generator skid in the Security and Health Physics Building to provide fuel for operation of the backup diesel generator. This area will be protected with a fire suppression system designed to NFPA 13 requirements for water sprinklers. A fire involving the indoor tank will not affect structures, systems or components outside of the Security and Health Physics Building. The outdoor tank will be above-ground, and will be designed in accordance with the requirements of NFPA 30, with dikes around the tank to contain fuel in the event of leaks or spillage.

While unlikely, it is considered possible that collision or tornado-driven missile impact with the outdoor tank could result in tank rupture and spillage of diesel fuel oil. If there were an ignition source at the location of the spilled diesel fuel, it would be possible to initiate a fire, though diesel fuel is difficult to ignite due to its low volatility. Rupture of a storage tank and spillage of diesel fuel does not create the potential for an explosion. It is planned to use Grade Low Sulfur No. 2-D diesel fuel oil in both applications (onsite vehicles and backup diesel generator), which has a flash point of 126°F (52°C) per Reference 48. Diesel fuel is not a flammable liquid (defined as a liquid having a flash point below 100°F), but falls into the classification of a Class II combustible liquid which has a flash point above 100°F and below 140°F (Reference 49). The flash point is defined as the lowest temperature at which the vapor pressure of the liquid is just sufficient to produce a flammable mixture at the lower limit of flammability above the surface of the liquid. In recognition of the relatively high flash point of diesel fuel oil (at above-ambient temperatures), NFPA 30 does not require use of explosion proof electrical equipment in the vicinity of diesel fuel oil. While spilled diesel fuel could burn it could not detonate, and therefore an explosion associated with diesel fuel oil is not considered to be a credible event. The outdoor diesel fuel oil storage tank is sufficiently removed from the Canister Transfer Building and the storage casks (nearest important-to-safety structures, systems, and components) that radiant heat energy from a diesel fuel oil fire at the storage tank would not result in damage.

Propane for heating the Canister Transfer Building is stored in one 2,000 gallon propane fuel storage tank, located inside the perimeter road and outside of the east-southeast of the nearest point on the Canister Transfer Building. Propane for heating the Security and Health Physics Building is stored in one separate and independent 1,000 gallon propane fuel storage tank, located inside the perimeter road and outside of the nuisance fence, approximately 450 ft east of the nearest point on the Canister Transfer Building and approximately 125 ft from the southeast corner of the Security and Health Physics Building. These two tanks will be separated from each other by at least 300 ft, which is considered more than sufficient distance to prevent a single projectile, such as a tornado-driven missile, from impacting both tanks. Both propane storage tanks are further than 1,000 ft from the nearest storage casks. The storage tanks will be above-ground, designed in accordance with the requirements of NFPA 58. Propane is stored as a liquefied petroleum gas with the tank pressurized to the vapor pressure of the propane liquid, whose temperature will be close to the average ambient daily temperature. The vapor pressure of commercial propane is 132 psig at 70°F and 216 psig at 105°F (Table 5-5E of Reference 49). Relief valves on the tanks will be set at approximately 275 psig. Propane is classified as a flammable liquid, and at standard atmospheric pressure (14.7 psia) commercial propane has a boiling point of minus 51°F (Table 5-5E of Reference 49). It is heavier than air, with propane vapor having a specific gravity of 1.52 at 60°F (Table 5-5E of Reference 49, with specific gravity air = 1). NFPA 58 requires that propane tanks between 50 and 2,000 gallon capacity be located at least 25 ft away from any building, adjacent container, or adjacent property.

8.2.4.2 Accident Analysis

Offsite Explosions

Regulatory Guide 1.91 (Reference 17) provides guidance for calculating safe distances from transportation routes, based on calculated overpressures at the nuclear site created by postulated explosions from transportation accidents. The Regulatory Guide indicates that overpressures which do not exceed 1 psi at the storage site would not cause significant damage and states that "under these conditions, a detailed review of the transport of explosives on these transportation routes would not be required." Using the methodology of Regulatory Guide 1.91, the nearest transportation routes are located much further from the PFSF than the distances required to exceed 1 psi overpressure. Based on this Regulatory Guide, the maximum probable hazardous solid cargo for a single highway truck is 50,000 lb, and detonation of this quantity of explosives could produce a 1 psi overpressure at a distance of approximately 1,660 ft (0.31 mile) from the detonation. Since the Skull Valley Road is 1.9 miles from the Canister Transfer Building and 2 miles from the nearest storage pad, explosions involving vehicles travelling on this road would not produce significant overpressures at these locations.

The effects of explosions on the storage systems are discussed in the HI-STORM and TranStor SARs, and it is determined that the canisters are protected from the effects of explosions. Overpressures of substantially greater than 1 psi would be required to cause damage to the cask storage systems. The Canister Transfer Building is designed to withstand extreme winds, pressure drops of 1.5 psi, and missiles associated with the design tornado. The effects of credible explosions occurring on the Skull Valley Road, with resultant overpressures less than 1 psi at the PFSF, would not challenge the Canister Transfer Building's structural integrity. Therefore, the canister storage and transfer systems meet the general design criteria of 10 CFR 72.122(c), as it applies to explosion, which states that structures, systems, and components Important to Safety must be designed and located so that they can continue to perform their safety functions effectively under credible fire and explosion exposure conditions.

The hazards that the Tekoi Rocket Engine Test Facility ("Tekoi") could pose to the PFSF would arise from a rocket motor 1) exploding while being tested, 2) exploding while being transported to Tekoi, or 3) escaping from its test stand and striking the PFSF. Regulatory Guide 1.91 (Reference 17) states the following:

> "A method for establishing the distances referred to above can be based on a level of peak positive incident overpressure (designated as P_{so} in Reference 1) below which no significant damage would be expected. It is the judgement of the NRC staff that, for structures, systems and components of concern, this level can be conservatively chosen at 1 psi (approximately 7 kPa). Based on experimental data on hemispherical charges of TNT cited in Reference 1, a safe distance can then be conservatively defined by the relationship
>
> $$R\ kW^{1/3}$$
>
> where R is the distance in feet from an exploding charge of W pounds of TNT. When R is in feet and W is in pounds, k = 45."

Reference 1 of Regulatory Guide 1.91, Army Technical Manual on Explosion Effects, is Reference 52 of this Chapter. Based on the Regulatory Guide 1.91 methodology, the overpressure produced by an explosion involving 1.2 million lbs of explosive material decays to 1 psi at a distance of 4,782 feet from the explosion. Reference 57 indicates that the overpressure from such an explosion would decay to 0.5 psi at a distance of 7,970 ft (approximately 1.5 miles). Since the distance from Tekoi to the PFSF site is over 2 miles, the overpressure at the PFSF from the worst-case explosion at Tekoi would be less than 0.5 psi. In Regulatory Guide 1.91, the NRC has established an overpressure of 1.0 psi as a safe threshold overpressure for explosions postulated to occur near nuclear power plants.[1] Thus, rocket motor explosions at Tekoi would pose no significant hazard to the PFSF.

Likewise, explosions of rocket motors in transit on Skull Valley Road or the Tekoi access road would pose no significant hazard to the PFSF. Since the Skull Valley Road is 1.9 miles from the Canister Transfer Building and 2 miles from the nearest

[1] Since overpressure causes greater damage to structures at comparable distances than heat or blast fragments, overpressure governs the safe offset distance. Reg. Guide 1.91 at 1.

storage pad, explosions involving vehicles travelling on this road would not produce overpressures in excess of 0.5 psi at these locations. The nearest approach of the access road on the Tekoi site to PFSF is further than 2 miles. Thus, an explosion on either the Skull Valley road or the Tekoi access road of the largest motor that could be tested at Tekoi would not create an overpressure of 1.0 psi at the PFSF.

A rocket motor escaping its test stand at Tekoi and striking the PFSF is not a credible event given the design and safety procedures employed at Tekoi and the intervening distance and terrain between Tekoi and the PFSF. Tekoi is conservatively designed to prevent rocket motors from escaping. The safety design features include a large thrust block into which the motor is directed and embedded structural steel to restrain and to retain the motor in place. Further, safety procedures require the careful inspection of the facility before each rocket motor is tested. In nearly 25 years of operation no rocket motor has escaped a test stand at Tekoi.[2]

Moreover, even in the highly unlikely event a motor were to escape, it is extremely unlikely that it would strike the PFSF. First, at a distance of more than 2 miles from Tekoi, the PFSF Restricted Area would comprise a small fraction of the potential area to which an escaped rocket motor might fly. Second, any rocket motor flying in the direction of the PFSF would likely strike Hickman Knolls – located between the PFSF and Tekoi. Therefore, it is an extremely remote possibility that a rocket motor escaping the test stand would strike the PFSF. And given the highly unlikely possibility that a rocket motor would escape a test stand in the first place, it is not credible that the PFSF would be struck by a rocket motor escaping from the Tekoi facility.

[2] In the early 1960s, a rocket motor escaped from a test stand (but did not leave the test range) at the Bacchus Works in Magna, Utah, the facility where Hercules, Inc., the prior owner of Tekoi, had conducted tests before Tekoi was built. After that incident the safety features described above – the thrust block and restraining structural steel members – were installed at the test site to prevent such events from recurring.

Consideration was given to any potential effects at the PFSF from a smoke plume originating at Tekoi from rocket motor testing or explosions. Any smoke plume from Tekoi would be greatly diluted by the time it reaches the PFSF restricted area, over two miles away. Based on a comparison of the χ/Q plume dispersion factor for a range of 150 meters from the point of origin to that for a range of 2 miles (3,219 meters) from the point of origin, the density of smoke or concentration of particulates in air would be over 150 times lower at a range of 2 miles. This is based on a comparison of plume dispersion factors estimated in accordance with NRC Regulatory Guide 1.145 (Reference 6), conservatively assuming atmospheric conditions conducive to maintaining a concentrated smoke plume at a distance from the burn site: atmospheric stability class F, wind speed of 1.0 meter/sec and without credit for plume meander. Furthermore, the dilution between the Tekoi test facility and the PFSF would be significantly greater due to the intervening Hickman Knolls, which would cause greater dispersion of the smoke in air traveling from Tekoi toward PFSF. It is concluded that the effects of smoke at the PFSF from rocket motor testing, testing accidents, or explosions at the Tekoi test facility would be negligible.

Onsite Explosions

It is conservatively assumed that the larger of the two propane tanks, which supplies propane for heating the Canister Transfer Building, contains its complete inventory of 2,000 gallons of liquefied propane and that it ruptures. While such rupture is unlikely, it is considered possible should a projectile, such as a tornado-driven missile, strike the tank, or in the event a vehicle leaves the perimeter road and strikes the tank. Since this tank is separated by a minimum distance of 300 ft from the 1,000 gallon propane tank that supplies propane for heating the Security and Health Physics Building, simultaneous rupture of both tanks is not considered to be credible. At 60°F, one gallon of propane liquid weighs 4.24 lbs (Table 5-5E of Reference 49). The total weight of propane in the larger tank is (2,000 gal) (4.24 lb/gal) = 8,480 lbs. It is also conservatively assumed that a large fraction of this propane mixes with air so that it is in an explosive concentration (in range of 2.15% to 9.60%, per Table 5-5E of Reference 49), ignites, and is involved in an explosion. The magnitude of the postulated explosion is assessed using the TNT energy equivalent methodology, with the TNT energy equivalence of 8,480 lbs of propane estimated as follows: Based on Table 5-5E of Reference 49, the total heating value of commercial propane after vaporization is 21,591 Btu/lb. Therefore, 8,480 lbs of propane has a total heating value of 1.83 E8 Btu, which is equal to 4.61 E10 calories. Regulatory Guide 1.91 (Reference 17) indicates that investigations led to estimates that less than one percent of the calorific energy of hydrocarbon gas/air vapor clouds that exploded was released in blast effects. It is conservatively assumed that 25% of the vapor is in a flammable gas-air mixture having concentrations ranging from the lower flammable limit of 2.15% to the upper flammable limit of 9.60% (Table 5-5E of Reference 49) and that 10% of the total heat of combustion of this flammable mixture is released in blast effects.

Energy Released in Blast = (4.61 E10 cal) (0.25) (0.10) = 1.15 E9 cal.

Trinitrotoluene (TNT) has a "heat of explosion" of 1,050 cal/g (Reference 51). The equivalent weight of TNT that would release 1.15 E9 calories of heat energy is:

(1.15 E9 calories) / (1.05 E3 cal/g) = 1.10 E6 g = 2,425 lbs

The overpressure effects of postulated detonation of this weight of TNT can be assessed using Figure 4-12 of Reference 52, "Shock-Wave Parameters for Hemispherical TNT Surface Explosion at Sea Level". This Reference 52 Army Technical Manual on Explosion Effects is Reference 1 of Reg. Guide 1.91, and provides the basis for Figure 1 of the Reg Guide. Figure 4-12 of Reference 52 presents overpressures at various scaled ground distances from TNT detonations, with varying weights of TNT, and defines the scaled ground distance as $Z_G = R_G / W^{1/3}$, where R_G is the actual ground distance and W is the weight of TNT (lbs). $W^{1/3} = (2{,}425)^{1/3} = 13.43$

While the storage casks can withstand a much higher overpressure before they begin to slide or tip, the Canister Transfer Building is designed to withstand a pressure differential of 1.5 psi due to a tornado (Sections 3.2.8.1 and 3.2.8.3) and an even higher load due to a seismic event. Therefore, the limiting overpressure for important-to-safety structures that could be impacted by a propane explosion is considered to be 1.5 psi. A 1.5 psi peak positive incident pressure corresponds to a scaled ground distance, Z_G, of approximately 32, based on Figure 4-12 of Reference 52. Solving the above equation for R_G:

$$Z_{G}, = R_G / W^{1/3}\ , \ 32 = R_G / 13.43\ , \ R_G = (32)\ (13.43) = 430 \text{ ft}$$

Thus, based on the TNT energy equivalence approach and Reference 52, the resulting overpressure from a propane explosion involving 8,480 lbs of propane (equivalent to 2,425 lbs of TNT) leaked from the larger of the two propane storage tanks will not exceed 1.5 psi at important-to-safety structures as long as the 2,000 gallon propane tank is located a distance of at least 430 ft from the Canister Transfer Building and storage casks. Both the 2,000 gallon propane tank that supplies the Canister Transfer Building and the 1,000 gallon propane tank that supplies the Security and Health Physics Building will be sited at a distance of approximately 450 ft from the Canister Transfer Building, and further than 1,000 ft from the nearest storage casks. This assures that postulated worst case explosion of propane assumed to have leaked from the larger tank will not produce overpressures greater than 1.5 psi and will not challenge the integrity of the storage casks or the Canister Transfer Building. The scenario evaluated above is bounding, in that postulated leakage and explosion of the 1,000 gallon propane inventory associated with the smaller storage tank, located at approximately the same distance from the Canister Transfer Building as the larger tank, would generate smaller overpressures at the important-to-safety structures.

8.2.4.3 Accident Dose Calculations

Since there is no potential for significant overpressures occurring at the PFSF as a result of nearby explosions, there would be no damage to the cask storage or transfer systems and no resultant dose.

8.2.5 Fire

Fire is classified as a human-induced Design Event IV as defined in ANSI/ANS-57.9.

8.2.5.1 Cause of Accident

The only combustible material at the PFSF storage pads during storage operations is insulation on the temperature monitoring instrumentation wiring, which is present in insignificant quantities at each storage cask. No combustible or explosive materials are allowed to be stored on or near the storage pads. The PFSF Restricted Area (RA) is cleared of vegetation and the entire RA surfaced with compacted gravel. The concrete pads and storage casks are located a minimum distance of 150 ft from the outer edge of the RA (i.e., the inner fence surrounding the RA); the Canister Transfer Building is located a minimum distance of 112 ft from the outer edge of the RA. The area between the outer edge of the RA and the outer edge of the perimeter road (50 ft distance, see Figure 1.2-1) is also covered with crushed rock. The only significant sources of combustibles that would be present inside the RA would be: 1) the diesel fuel in the tanks of any heavy haul trucks transporting shipping casks to/from the PFSF site; 2) the diesel fuel in the tanks of any train locomotive transporting shipping casks to/from the PFSF site; 3) the diesel fuel in the cask transporter vehicle that would move casks from the Canister Transfer Building to the storage pads; 4) the diesel generator fuel tank inside the Security and Health Physics Building; and 5) the diesel fuel storage tank, which would be located at least 50 ft inside the inner fence surrounding the RA, approximately 200 ft northeast of the Canister Transfer Building and 700 ft east of the nearest storage casks.

The two propane tanks that supply propane to heat the Canister Transfer Building and the Security and Health Physics Building (described in Section 8.2.4.1) are located outside of the RA east of the Canister Transfer Building, between the nuisance fence and the perimeter road.

The effects of wildfires in the vicinity of the PFSF and the effects of fires involving combustibles and transient combustibles located in the RA are evaluated below.

Wildfires

A report has been prepared for PFS to evaluate potential wildfires in Skull Valley (Reference 40). The report discusses the annual probability of wildfires, as well as range fire magnitudes, duration, propagation and heat generation. The first section of the report (pages 1 through 3) provides historical information on the number of occurrences and size of wildfires in Skull Valley. Figure 1 of the report shows the individual fires by size that occurred in Skull Valley between 1980 and 1997 while Figure 2 shows the fire occurrence by size class for the same time period. The report concludes the following regarding probability of occurrence and severity of wildfires in Skull Valley:

- Fires occur every year in Skull Valley
- The number of fires are very low on average at about 6 per year over the Skull Valley area
- The chance, on a percentage area basis, of a worst case fire even encountering the perimeter of the PFSF is well below 1% in a given year
- With the use of planning (fuel modification and fuel breaks) and current attack methods (aerial slurry drops), it is highly improbable that a fire would ever reach the site perimeter
- With a 100-ft fuel break, no heat damage could be caused to either equipment , structures, or any life form on the ground

The latter section of the report (pages 4 through 7) addresses fire magnitude, duration, propagation, and heat generation and concludes the following:

- Data available describes fuel loading at about 5,000 lb/acre
- With this fuel loading, flame lengths up to a maximum of 28-ft are possible for very short periods of time
- Maximum fire temperatures are reached near the soil surface and decrease rapidly above the top of the primary fuel (grass)
- Rate of spread is highly variable, but where heavy fuel is available can be as high as 590 ft/min for short runs
- Fire intensity is normal for this fuel type and fuel load
- Fires in this fuel type can be easily modified by reducing fuel load, i.e., planting a crested wheatgrass buffer around all areas where fire might present a problem
- With implementation of only minimal fuel modifications as mentioned above, wildfire will pose no hazard to the PFSF

The crushed rock surface of the RA and of the contiguous area out to the outer edge of the perimeter road provides a fire break of at least 200 ft to the concrete pads, where the storage casks are located, and a fire break of 162 ft to the Canister Transfer Building. In addition, the spent fuel, equipment, and the PFSF personnel inside the RA will be protected from wildfires by a barrier of crested wheat grass that PFS will plant around the RA. The barrier will be 300 ft wide and will run outward from the outer edge of the perimeter road around the RA. A barrier of crested wheat grass would remain in place with little maintenance after it is planted. Crested wheat grass is fire resistant and thus would eliminate or greatly reduce the effect of any wildfire approaching the PFSF. Because of the distance that would separate a wildfire from the Canister Transfer Building and the casks containing spent fuel at the PFSF, a wildfire would pose no direct threat to the spent fuel casks or the SSCs important to safety in the Canister Transfer Building. The magnitude and duration of temperatures resulting from a wildfire at both the storage pads, and the storage casks located there, and at and within the Canister Transfer Building would be far less than those of the design basis fire, discussed below, for which the casks are designed to withstand (Reference 40).

Furthermore, a wildfire could not cause a fire or explosion on site that would threaten the spent fuel casks or SSCs important to safety. The location of the diesel fuel storage tank, at least 50 ft inside the inner fence around the RA, provides a 100 ft firebreak between the outer edge of the perimeter road and the tank, with the crested wheat grass barrier providing an additional 300 ft between a wildfire and the storage tank. At that distance a wildfire would not ignite or explode the diesel fuel in the tank. The diesel emergency generator tank will be a double-walled tank located inside the Security and Health Physics Building, which has reinforced concrete masonry construction, located 50 ft inside the crested wheat barrier, or 350 ft from a wildfire. A wildfire would not ignite or explode the fuel in the diesel emergency generator tank. All other diesel fuel sources would be farther than 100 ft inside the edge of the crested wheat grass barrier, and would similarly not be threatened by a wildfire due to their distance from a fire, even if it were assumed the wildfire somehow penetrated this grass barrier.

The two propane storage tanks that supply propane to heat the Canister Transfer Building and the Security and Health Physics Building (described in Section 8.2.4.1) are located east of the Canister Transfer Building between the nuisance fence and the perimeter road. In order to assure the propane tanks are adequately protected from a wildfire, PFS will install a crushed rock surface, that will be devoid of vegetation that could propagate a wildfire, which will extend 100 ft east of the east perimeter road and 100 ft south of the south perimeter road in the southeast corner of the facility. This will assure that there is a minimum of 120 ft crushed rock fuel break (20 ft compacted crushed rock road encompassed by a 100 ft crushed rock fire barrier) around the propane storage tanks. In accordance with Reference 40, a 100 ft fuel break provides adequate protection from wildfires and no heat damage would be caused to either equipment, structures, or any life form on the ground. The crested wheat fire barrier, which extends out 300 ft from the perimeter road (200 ft beyond the edge of the crushed rock fire barrier discussed above), provides additional protection and it is concluded that the propane tanks would safely contain the propane and there would be no ignition or explosion of the propane fuel in the event of wildfires in the vicinity of the PFSF. Section 8.2.4.2 includes an analysis of the effects of a postulated explosion of the larger propane tank (2,000 gallons) that supplies propane to the Canister Transfer Building, which determined that overpressures at the important-to-safety structures would be less than 1.5 psi and would not adversely impact these structures.

A wildfire in the vicinity of the PFSF would not cause the evacuation of PFSF security personnel. By virtue of the 300 ft crested wheat grass barrier surrounding the PFSF RA and the distance between the outer edge of the perimeter road around the RA, the heat from a wildfire would not pose a threat to any personnel inside the RA. PFSF security personnel will have appropriate emergency breathing apparatus available such that the smoke from a wildfire near the PFSF will not force them to evacuate.

Combustion Sources Inside the Restricted Area

Movement of a storage cask from the Canister Transfer Building to a storage pad involves the use of a diesel-powered cask transporter, whose fuel tank has a capacity of 50 gallons of diesel fuel. The worst-case fire at the storage pads involves a postulated spill and ignition of this diesel fuel in the vicinity of a storage cask. The accident scenario involving a storage cask in the following section assumes that the fuel tank of the transporter vehicle ruptures, resulting in 50 gallons of diesel fuel spilled, which is postulated to ignite and burn.

The combustibles of key concern in the Canister Transfer Building are the transient combustibles associated with the diesel fuel tanks of the cask transporter and the heavy haul vehicle tractor. For rail delivery/retrieval of shipping casks, the train locomotives are required by administrative procedure to stay out of the Canister Transfer Building. The design of the building and its surroundings will assure that any diesel fuel spilled outside the building will not flow into the building, which could create a fire hazard. The heavy haul vehicle tractors have saddle tanks with a total capacity of up to 300 gallons of diesel fuel. Spillage of diesel fuel does not create the potential for

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locomotive staged at the PFSF ruptures and diesel fuel spills onto the ground and ignites outside the Canister Transfer Building. The PFSF railroad line nearest the spent fuel storage pads is 107 ft from the closest pads, at the south end of the PFSF Restricted Area (Fig. 1.2-1). Thus, a fire associated with a locomotive would be approximately 100 ft from the nearest spent fuel storage casks. As a result of the distance, the heat flux impinging on the storage casks (Section 21, Chapter 6 of Reference 39) and the effects of such a fire on the storage casks at the PFSF would be much less than those resulting from the postulated fire in which 50 gallons of diesel fuel is assumed to encircle a storage cask and burn, which is described above. Therefore, the storage casks would retain their integrity and there would be no release of radioactivity from storage casks, even in the highly unlikely event of a diesel fuel fire associated with a locomotive.

Canister Transfer Building

A fire in the Canister Transfer Building would have a negligible effect on storage casks on the storage pads because of the concrete construction of the building walls and the distance between the Canister Transfer Building and the storage pads. The Canister Transfer Building is approximately 425 ft from the nearest storage pad.

The Canister Transfer Building contains minimal combustible loading, except when a heavy haul tractor or cask transporter is present in the building. Transient combustibles associated with these vehicles are up to 300 gallons of diesel fuel inside the saddle tanks of the heavy haul tractor, and up to 50 gallons of diesel fuel inside the fuel tank of the cask transporter. In the event of rail delivery/retrieval of shipping casks, the train engines are required by administrative procedure to stay out of the Canister Transfer Building. Although it is highly unlikely that a fuel tank could rupture and spilled diesel fuel ignite, the design of the Canister Transfer Building includes provisions to address these scenarios, as discussed below.

The first postulated fire scenario in the Canister Transfer Building is assumed to involve 300 gallons of diesel fuel from ruptured fuel tanks of a heavy haul tractor in the shipping cask load/unload bay. The heavy haul vehicles enter and exit the cask load/unload bay at the south end of the Canister Transfer Building and do not approach a transfer cell where canister transfer operations are conducted. Building design measures assure that any diesel fuel spilled in the cask load/unload bay will remain in the bay and cannot enter a transfer cell. It is not credible therefore that the postulated 300-gallon diesel fuel fire discussed above would affect spent fuel storage casks or transfer casks containing loaded spent fuel canisters, since the spent fuel storage casks at the PFSF will be located either on the concrete storage pads or in a canister transfer cell, but not in the shipping cask load/unload bay, and a loaded transfer cask would only be located in a canister transfer cell. The Canister Transfer Building design includes automatic fire detection and suppression systems, described in Section 4.3.8. The fire suppression system, which provides coverage of the cask load/unload bay, consists of a water-foam sprinkler system. As discussed in Section 4.3.8.1, no sprinklers are located near the transfer cells, which are considered a sprinkler exclusion area to avoid the possibility of spraying down a canister and dislodging possible contamination. Even if no credit were taken for the automatic fire detection/suppression systems in the cask load/unload bay, an analysis of a 300-gallon diesel fire in the cask load/unload bay, based on information in the Fire Protection Handbook (Reference 39) shows that the fire would burn out in less than 10 minutes and would not threaten any systems, structures, or components (SSCs) important to safety even without the operation of automatic fire detection and suppression systems.

The cask load/unload bay is approximately 198 ft. long and 48 ft. wide. Diesel fuel spilled into this bay would tend to spread forming a relatively thin layer. In calculating a fuel burn time, it is conservative to assume that the diesel fuel forms a relatively deep pool, in that a deep pool burns longer than a shallow pool, and a 1 inch depth is considered to be a very conservative assumption. A 300-gallon volume of liquid at a depth of 1 inch would occupy an area of 481 sq. ft., represented by a circle with a 12.4 ft. radius. This surface area is only 5% of the total area of the cask load/unload bay, so the walls of the bay would not confine the spilled fuel within a smaller and deeper pool. Any drain sumps in the cask load/unload bay that could potentially collect diesel fuel from postulated rupture of the heavy haul tractor fuel tanks will be located so as to assure that burning of diesel fuel in these sumps will not threaten SSCs important to safety.

Assuming that this pool of 300 gallons of diesel fuel is ignited and burns, the duration of combustion can be calculated using the 0.15 inch/minute fuel consumption rate assumed in Section 11.2.4 of the HI-STORM SAR, based on data from Reference 19. A 1 inch deep pool of diesel fuel will be consumed in 1 in. / 0.15 in./min. = 6.67 minutes. Figure 7-9B of Reference 39 provides time-temperature curves for different types of fires from slight to moderate to severe. Temperature curve E of this figure is for the "standard exposure fire - severe", and includes fires fueled by flammable liquids. This standard fire time-temperature curve, which is also shown in Figure 7-9A of Reference 39, reaches a temperature of 1,000 °F at 5 minutes and a temperature of 1300 °F for a 10 minute burn duration. For the calculated 6.67 minute burn duration, a peak temperature of approximately 1200 °F would be reached. This fire would not threaten any SSCs important to safety at the PFSF in a way that could cause a radioactive release. Neither storage nor transfer casks are located in the shipping cask load/unload bay, as stated above, and the shipping casks are required to be demonstrated capable of safely withstanding the effects of an exposure fire that burns at 1475°F for 30 minutes per 10 CFR 71.73(c)(4). Further, as shown in Figure 4.7-1, the overhead crane is located approximately 70 ft above the floor, the semi-gantry crane is located approximately 55 ft above the floor, and the roof is approximately 90 ft above the floor of the Canister Transfer Building. Therefore, the only credible significant impact a fire might have is that it could cause a loss of electrical power to

SSCs inside the Canister Transfer Building. Section 8.1.1.3 shows, however, that a loss of power would not cause an accident that would result in a release of radioactivity, even if it occurred while canister transfer operations were in progress.

The second postulated fire scenario in the Canister Transfer Building is assumed to involve 50 gallons of diesel fuel from ruptured fuel tanks of the cask transporter in one of the three canister transfer cells. A fire involving up to 50 gallons of diesel fuel could burn for up to 3.6 minutes duration (as discussed previously), consuming the entire fuel inventory. The cask transporter enters a transfer cell for the purposes of moving an empty storage cask into the cell, and moving a loaded storage cask out of the cell and out to the storage pad. During canister transfer operations, the cask transporter is prevented from entering a transfer cell by shield doors on either side of the transfer cell. PFSF procedures will require that the shield doors remain closed when a canister transfer operation is in progress. Building design measures assure that any diesel fuel spilled in the Canister Transfer Building main bay outside of a transfer cell will not run into a transfer cell. A cask transporter could enter a transfer cell when the canister is in the shipping cask and its lid bolted in place, or when the canister is in the storage cask and the storage cask lid has been bolted in place. As noted above, the shipping casks are required by regulation to be demonstrated capable of safely withstanding the effects of an exposure fire that burns at 1475°F for 30 minutes, with spent fuel remaining within temperature limits and no breach of the confinement barrier. Therefore, short duration fires in a transfer cell resulting from postulated rupture of the cask transporter's diesel fuel tanks and ignition of the pool of fuel would not result in breach of the shipping cask confinement and there would be no release of radioactivity. Fires involving shipping casks can result in reduction of neutron shielding, as discussed in Chapter 5 of both vendors' shipping cask SARs (References 5 and 20). Storage casks are relatively impervious to the effects of fires, as discussed above, and there would be no damage to the canister confinement or the spent fuel for fires in the vicinity of a loaded storage cask. The occurrence of a fire in a transfer cell while a canister is in a transfer cask is

8.2.11 Extreme Environmental Temperature

Extreme environmental temperature is classified as a natural phenomenon Design Event IV as defined in ANSI/ANS-57.9.

8.2.11.1 Cause of Accident

The extreme environmental temperature is assumed as a constant ambient temperature caused by extreme weather conditions. It was conservatively assumed that the temperature persists for a sufficiently long time for the storage casks to reach thermal equilibrium.

8.2.11.2 Accident Analysis

The extreme environmental temperature condition is analyzed, and results reported, in the HI-STORM and TranStor storage cask SARs. The accident condition considers an environmental temperature of 125°F with full solar insolation for a sufficiently long time to reach steady-state conditions. In reality, this weather condition could not be maintained long enough, since the storage cask needs several days to reach equilibrium. The short-term fuel cladding temperature limit of 1058°F and the short-term concrete temperature limit of 350°F are not violated for either the HI-STORM or the TranStor storage casks.

The maximum steady-state temperatures of key storage system components for both the HI-STORM and the TranStor storage casks are provided in Table 8.2-1. As discussed in the HI-STORM and TranStor SARs, the component temperatures are all within the temperature limits. Internal pressure for this condition is bounded by the hypothetical accident pressurization discussed in Section 8.2.10.

8.2.11.3 Accident Dose Calculations

There is no effect on the shielding performance of the system as a result of this condition, since the concrete temperature does not exceed the 350°F short-term concrete temperature limit. There is no effect on the criticality control features or the confinement function of the system as a result of this event. Based on this evaluation, there are no radiological consequences for this accident.

8.4 BASIS FOR SELECTION OF OFF-NORMAL AND ACCIDENT CONDITIONS

ANSI/ANS 57.9 (Reference 1), the regulatory guidance in Sections 12.4.1 and 12.4.3 of NUREG-1567 (Reference 56), and the storage system vendor SARs (References 2 and 3) were used as the basis for selecting off-normal and accident conditions to ensure all relevant or potential scenarios were considered.

Section 12.4.1 of NUREG-1567 indicates that examples of off-normal and accident conditions that should be considered in the SAR include those provided in ANSI/ANS-57.9. As described in the introduction to this Chapter 8 and Section 8.2, ANSI/ANS-57.9 served as a basis for identifying accidents and classifying them into off-normal conditions or accidents. ANSI/ANS-57.9 Design Event II conditions are described in PFSF SAR Section 8.1. ANSI/ANS-57.9 Design Events III and IV are described in Section 8.2. The examples of off-normal occurrences and accidents provided in ANSI/ANS-57.9 are included in this chapter, where applicable to the PFSF.

The regulatory guidance of NUREG-1567, Section 12.4.3, was also used as a basis for selecting off-normal and accident conditions to ensure all relevant or potential scenarios were considered. Consideration of the eight bullet items listed in Section 12.4.3 included the following:

Site Characteristics

Consideration was given to the following in the analysis of the PFSF site characteristics:

- Thermal analyses of the effects of abnormally high ambient temperatures on the storage system considered climactic conditions of the area, and temperatures were selected to bound average daily (day/night) maximum temperatures that could occur over a period of several days (Section 8.1.2).
- As described in Sections 2.6 and 8.2.1, a seismological evaluation of the PFSF

siting area was performed. Although the HI-STORM and TranStor cask storage systems have been analyzed for generic design earthquakes (DE) selected by each vendor and described in their respective SARs, both storage systems were also analyzed for the PFSF site specific design basis ground motion. The site specific design basis ground motion is discussed in Section 3.2.10 and is represented by response spectra curves developed specifically for the PFSF site.

- Section 8.2.2.2 indicates that for the extreme wind accident, the maximum wind speed and pressure drop analyzed by the storage system vendors substantially envelopes the site specific requirements defined in Regulatory Guide 1.76 for tornado Region III. The Canister Transfer Building is designed to withstand the effects of the Region III tornado and pressure drop forces.

- In the case of flooding, Section 8.2.3 assesses the site specific effects of flooding as well as providing information regarding capabilities of the vendor storage casks, which are designed to withstand severe flooding including full submergence.

- The explosion analysis in Section 8.2.4 considers the effects of explosion at the stationary rocket engine test facility located approximately 2.5 miles south-southeast of the PFSF site. The effects of explosions from a transportation accident on the Skull Valley Road, 2 miles from the nearest storage pad, and from propane postulated to have leaked from the largest on-site storage tank, were also evaluated.

- Fires evaluated in Section 8.2.5, and design measures to mitigate the effects of fires, are based on specific fire hazards associated with the PFSF, including wildfires in the vicinity.

- The hypothetical storage cask drop/tipover not only discusses the vendors' generic analyses, but also factors associated with site-specific storage pad concrete and soil parameters.

- Section 2.2 evaluates the threat of aircraft crashes and determines that the probability of an aircraft impacting the PFSF is below applicable NRC regulatory standards and guidance and therefore is not considered to be a credible event. As also discussed in Section 2.2, other activities associated with military facilities and military ranges in the vicinity of the PFSF pose no credible hazard to the facility. The analyses of the hazard posed by aircraft and other military activities in the vicinity of the PFSF are based on the specific characteristics of the PFSF site and aircraft operations, including military, in the area.

Automatic and Manual Safety Features

Analysis of the canister transfer operations identified the need for single-failure-proof canister and transfer cask lifting equipment which was incorporated into the design of lifting devices. As discussed in Section 8.1.1.3, the overhead bridge crane, semi-gantry crane, canister downloader and associated lifting devices used to handle shipping casks, transfer casks and canisters in the Canister Transfer Building are all designed to meet the criteria for single-failure-proof lifting devices and to hold the lifted load in place in the event of loss of electrical power. The cranes are seismically qualified to assure dropped loads will not occur in the event of the design basis ground motion.

Necessary Instrumentation and Control Features

Analysis of the method for detecting blockage of the storage cask air paths at existing ISFSIs smaller than the PFSF (i.e., daily visual inspection of the cask vents) identified a desire for an alternative method that has a significant ALARA benefit. It was determined to use a cask temperature monitoring system that continuously monitors the temperature of the casks. This allows adequate monitoring of the cask thermal performance without subjecting operators to a daily radiation dose. To provide assurance of the availability and reliability of the temperature monitoring system, the following features are provided: backup power, procedures to periodically calibrate components and test the operability of the monitoring system, and a daily review of the monitoring output to detect trends of increasing cask temperature.

Although important for ALARA purposes, the temperature monitoring system is not classified as Important to Safety. In the event of failure of the system, a supervised alarm and detection system (which is a separate alarm from the high temperature alarm) will notify operators who will provide visual inspections of the affected cask(s) until the monitoring system is repaired.

The visual inspection of the cask vents to verify no blockage will still be performed, but on a quarterly basis. The use of the temperature monitoring instrumentation to reduce the frequency of visual inspections from a daily to a quarterly activity will greatly reduce the radiation exposure of personnel.

As discussed in Section 7.3.5, airborne radioactivity concentrations will be detected by continuous air monitors located in the exhaust of each canister transfer cell. The continuous air monitors will include local alarms to warn operating personnel in the unlikely event of an airborne release, remote alarm in the Security and Health Physics Building alarm station to ensure coverage at all times, and charting capability to provide data necessary to quantify any release.

Sequences of Operations and Projected Contingency Actions

Analysis included a review of the sequence of operations associated with shipping cask receipt, canister transfer, and movement of the storage casks to the storage pads. The "stacked cask" configuration, where the transfer cask is supported on the shipping cask or storage cask, was thoroughly assessed and it was decided that the Canister Transfer Building should be qualified to withstand the effects of tornado winds and tornado-driven missiles to shelter this configuration in the operations sequence from the effects of tornadoes. Section 8.2.1.2 identifies several requirements associated with canister transfer operations that were the result of consideration of a seismic event occurring at different stages of the canister transfer sequence. For instance, prior to disconnecting the crane from the shipping cask after it is placed in a canister transfer cell, seismic support struts are secured to the cask. The stacked cask configuration was evaluated for stability in the event of a DE. During the canister transfer operation, the crane is not disconnected from the transfer cask that is supported by the shipping cask or storage cask until seismic support struts are connected to the transfer cask to assure its stability.

The analysis considered the occurrence of fires (Section 8.2.5) involving 1) wildfires in the vicinity of the PFSF; 2) 300 gallon fuel capacity heavy haul vehicle with a shipping cask in the cask load/unload bay; 3) 50 gallon fuel capacity cask transporter in a canister transfer cell; 4) cask transporter with a storage cask enroute to the storage pads; and 5) cask transporter with a storage cask on the storage pads. Several restrictions resulted from the assessment of potential fires, including the cask transporter not permitted in a canister transfer cell while transfer operations are in process and, for rail delivery/retrieval of shipping casks, the train locomotives are required by administrative procedure to stay out of the Canister Transfer Building.

The sequence of operations was also considered in regards to loss of electrical power as described in Section 8.1.1.3, which states:

> "It is postulated that a loss of external electrical power event could occur during the canister transfer operations that are conducted in the PFSF Canister Transfer Building. This could take place at any point in the transfer sequence. Consideration is given to the loss of power: (1) while a loaded shipping cask, with the impact limiters removed, is being unloaded off the heavy haul trailer or rail car; (2) while the canister is being raised from the shipping cask into the transfer cask; (3) while the loaded transfer cask is being moved from above the shipping cask to above the

storage cask; and (4) while the canister is being lowered from the transfer cask into the storage cask."

Characteristics of Facilities and Equipment

The location of the cooling air inlet ducts at the bottom of the storage casks, a characteristic of their design, gives rise to consideration for potential duct blockage due to buildup of material on the storage pads due to high winds, tornado, heavy snow, and flooding, evaluated in Sections 8.1.3 and 8.2.8. Section 8.1.4 evaluates bumping of the canister against the sides of the shipping or storage cask during canister transfer, which relates to characteristics of the cask configuration during this operation. Characteristics of the fuel and canister were accounted for in assessing the possibility and consequences of canister leakage and canister pressurization accidents, discussed in Sections 8.2.7 and 8.2.10, respectively. The characteristics of the heavy haul vehicle and cask transporter (fuel tank capacities) were considered in evaluating the consequences of postulated fires, as discussed above. Vertical drop of a storage cask (Section 8.2.6) considered characteristics of the cask transporter as well as the storage pads.

Consequences of Failures of Structures, Systems, and Components (SSCs)

Consideration of failure of lifting devices led to the decision to require that these devices meet single-failure-proof requirements to avoid the consequences of dropped casks and/or canisters.

Although it was determined that canister leakage with 100% of the fuel rod cladding assumed to have failed is not a credible event, the consequences of this hypothetical accident are evaluated in Section 8.2.7. The consequences of this accident bound those of credible accidents that could occur at the PFSF.

Consideration was also given to equipment malfunctions. One condition that was assessed was postulated malfunction of the transfer cask doors during the canister transfer operation. This was not included as an off-normal event since it involves routine operation that causes a schedular delay, but does not challenge safety. The results of this assessment are included in the following paragraphs:

Assessment of Transfer Cask Door Malfunction

It is very unlikely that the transfer cask doors would fail due to the simplicity and inherent reliability of the door design and their opening/closing mechanism. The TranStor transfer cask doors slide open and closed along greased rails which support and align the doors. The TranStor system uses a hydraulic operator to provide the necessary force to open/close the transfer cask doors. The doors of the HI-TRAC transfer cask are equipped with multiple wheels that run along guided rails, enclosed, with no obstacles or protrusions. The doors are housed in such a way that they cannot come off the tracks. The reduced friction associated with the wheels enables the HI-TRAC doors to be manually opened and closed by the operators, and handles are provided on the doors for this purpose. The transfer cask doors of both vendors' transfer casks will be tested during the preoperational testing program to verify that they operate smoothly and there are no obstructions or misalignment that could cause jamming.

In the event the transfer cask sliding doors fail to close after the canister has been hoisted up into the transfer cask in preparation for a transfer operation, the transfer operation will cease, the canister will be lowered back down into the underlying cask, the transfer cask removed from the cask, the lid placed back on the cask, and the transfer cask doors or door operating mechanisms repaired.

In the event the transfer cask sliding doors are closed and fail to open when it is desired to lower the canister from the transfer cask into an underlying storage or shipping cask, then actions would be taken to make necessary repairs and open the doors with the canister in the transfer cask while the transfer cask is supported by the underlying cask. Dose considerations would be associated with this repair operation, as dose rates at the side of transfer casks are relatively high compared to dose rates on the sides of storage casks (as indicated in Tables 7.3-1 through 7.3-4), and use of temporary shielding may be desirable. There is no hurry or time constraint associated with this operation, as the canister can remain housed in the transfer cask indefinitely without posing a safety concern. Corrective action would be carefully planned and executed in a deliberate controlled manner that assures doses to personnel involved are maintained ALARA. In the case of the TranStor transfer cask, if the hydraulic operator is broken and unable to open the sliding door, then the hydraulic operator would be repaired. This repair could either be performed in place, or the hydraulic operator mechanism could be removed, taken to another location (e.g. a low dose area) where the repair could be effected, then returned to the transfer cask, re-mounted and hydraulic force applied to open the sliding doors. In the case of the HI-TRAC transfer cask, if operators are unable to slide the doors open manually, a portable cable winch could be connected and used to provide the additional force necessary to slide the doors open even if a wheel is jammed or a wheel bearing seized.

Historical Considerations

Several accident conditions were evaluated not because they represent credible scenarios, but based on historical considerations - the fact that these conditions were considered in the licensing basis of other ISFSIs and/or in the PFSF storage cask vendor SARs. The hypothetical storage cask tipover and hypothetical accident pressurization are examples of accidents considered partly as the result of historical reasons, which do not represent credible accident scenarios. Section 12.4 of NUREG-1567 states that "Credible accident level events and conditions should be analyzed (or bounded by design basis accidents) to demonstrate that the consequences do not exceed the limits of 10 CFR 72.106(b). (Design basis accidents are the subset of all credible accidents that bound the entire spectrum of accidents that could occur in terms of the nature and consequences of accidents.) Instead of providing analysis for every credible accident scenario, the SAR may choose to characterize and analyze the subset of design basis events." Section 12.4.1 of this NUREG states that "Credibility is the determinant for analysis and satisfaction of criteria for accident-level events and conditions." The PFSF SAR exceeds these requirements and analyzes several incredible accident scenarios, largely due to historical precedent.

Consequences of Human Error

Section 8.1.4 assesses consequences of the occurrence of postulated operator error during the canister transfer operation. As stated in this section:

> "Load drops by the overhead bridge crane, the semi-gantry crane, or the canister downloader are not considered credible because of the single-failure-proof design of these lifting systems. Postulated events are: (1) while lifting the canister out of the shipping cask and into the transfer cask, personnel error could result in lifting the canister too high so it contacts the top of the transfer cask; (2) during placement of the canister into the storage cask, improper operation of the crane or canister downloader may cause a lateral impact against the inside of the storage cask (this could also occur during transfer of the storage cask to a storage pad, where an inadvertent movement could cause lateral impact of the canister against the inside of the storage cask); and (3) during canister lowering into the storage cask with the transfer cask improperly aligned with the storage cask, the canister could encounter interference, such as catching on the edge of the storage cask."

It is considered that the off-normal contamination release event could occur as the result of operator error.

In addition to ANSI/ANS 57.9, and the regulatory guidance in Sections 12.4.1 and 12.4.3 of NUREG-1567, the storage system vendor SARs were used as a basis for selecting off-normal and accident conditions

Conclusion

Based on the above, the accident analysis in the PFSF SAR is based on a thorough review of a wide range of accident level events and conditions in accordance with the regulatory guidance in NUREG-1567. This approach ensured that relevant or potential off-normal and accident scenarios are considered in the PFSF SAR.

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30. Federal Guidance Report No. 12, External Exposure to Radionuclides in Air, Water, and Soil, EPA 402-R-93-081, U.S. Environmental Protection Agency, September 1993.

31. Interim Staff Guidance-5, Normal, Off-normal, and Hypothetical Accident Dose Estimate Calculations for the Whole Body, Thyroid, and Skin, U.S. NRC Spent Fuel Project Office, October 6, 1998.

32. ANSI N14.5, Radioactive Materials - Leakage Tests on Packages for Shipment, American National Standards Institute, 1977.

33. Topical Safety Analysis Report for the Holtec International Storage, Transport, and Repository Cask System, (HI-STAR 100 Cask System), Holtec Report HI-941184, Docket 72-1008, Revision 8, August 1998.

34. NUREG/CR-6487, Containment Analysis for Type B Packages Used to Transport Various Contents, prepared for the U.S. NRC by Lawrence Livermore National Laboratory, November 1996.

35. NUREG-1617, Standard Review Plan for Transportation Packages for Spent Nuclear Fuel, Draft Report for Comment, March 1998.

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37. Regulatory Guide 1.111, Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases from Light-Water-Cooled Reactors, Revision 1, July 1977.

38. Interim Staff Guidance-3, Post Accident Recovery and Compliance with 10 CFR 72.122(l), U.S. NRC Spent Fuel Project Office, October 6, 1998.

39. Fire Protection Handbook, Sixteenth Edition, National Fire Protection Association, 1986.

40. Report by Carlton M. Britton, dated February 8, 1999; This report is Attached to the Response to PFSF Safety RAI No. 2, SAR 8-3, submitted to the NRC by PFS letter J. Parkyn to Director, Office of Nuclear Material Safety and Safeguards, dated February 10, 1999.

41. PFS Letter, Parkyn to U.S. NRC Document Control Desk, Request for Exemption to 10 CFR 72.102(f)(1), dated August 24, 1999.

42. Holtec Report No. HI-992277, Multi-Cask Response at the PFS ISFSI, From 2000 Year Seismic Event, Revision 0, dated August 20, 1999.

43. Interim Staff Guidance-12, Buckling of Irradiated Fuel Under Drop Conditions, U.S. NRC Spent Fuel Project Office, May 21, 1999

44. PFSF Calculation No. 05996.02-UR-5, Dose Rate Estimates from Storage Cask Inlet Duct Clearing Operations, Revision 0, Stone & Webster.

45. PFSF Calculation No. 05996.01-UR-3, Postulated Release of Removable Contamination from Canister Outer Surfaces - Dose Consequences, Revision 2, Stone & Webster.

46. PFSF Calculation No. 05996.02-UR-009, Accident Dose Calculations at 500m and 3219m Downwind for Canister Leakage Under Hypothetical Accident Conditions for the Holtec MPC-68 and SNC TranStor Canisters, Revision 1, Dade Moeller & Associates.

47. PFSF Calculation No. 05996.02-UR-010, RESRAD Pathway Analysis Following Deposition of Radioactive Material From the Accident Plumes, Revision 1, Dade Moeller & Associates.

48. American Society for Testing and Materials (ASTM) Standard D975-1997, Standard Specification for Diesel Fuel Oils.

49. Fire Protection Handbook, Sixteenth Edition, National Fire Protection Association, 1986.

50. (deleted)

51. Rudolph Meyer, Explosives, 3rd Edition, 1987.

52. Department of the Army Technical Manual TM 5-1300, "Structures to Resist the Effects of Accidental Explosions," June 1969.

53. Sierra Nuclear Corporation Calculation PFS01-10.02.04, Soil Structure Interaction Analysis for Evaluation of TranStor Storage Cask Seismic Stability, Revision 0, dated July 24, 1997.

54. Sierra Nuclear Corporation Calculation PFS01-10.02.05, TranStor Storage Cask Seismic Stability Analysis for PFS Site, Revision 0, dated July 24, 1997.

55. Holtec Report No. HI-992295, TranStor Dynamic Response to 2000 Year Return Seismic Event, Revision 0, dated September 17, 1999.

56. NUREG-1567, Standard Review Plan for Spent Fuel Dry Storage Facilities, Draft Report for Comment, October 1996.

57. Alliant Techsystems Inc. letter, C.F. Davis to J.L. Donnell, Private Fuel Storage Facility – Skull Valley Goshute Reservation, dated June 26, 1997.

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TABLE 8.1-1

STORAGE SYSTEM OFF-NORMAL MAXIMUM AMBIENT TEMPERATURE EVALUATION

	HI-STORM Storage System Temperatures (°F)	TranStor Storage System Temperatures (°F)
Ambient Air	100	100
Storage Cask Air Outlet	206	210
Storage Cask Outer Surface	151	132
Storage Cask Inner Concrete	192	221
Canister Outer Surface	327	314
Fuel Clad	765	657

Note: The above results are for the bounding canister temperature case (PWR or BWR) for the respective vendor.

TABLE 8.1-2

PARTIAL BLOCKAGE OF STORAGE CASK AIR INLET DUCTS TEMPERATURE EVALUATION

	HI-STORM Storage System Temperatures (°F)	TranStor Storage System Temperatures (°F)
Ambient Air	80	75
Storage Cask Air Outlet	202	194
Storage Cask Outer Surface	135	103
Storage Cask Inner Concrete	186	200
Canister Outer Surface	322	298
Fuel Clad	754	645

Note: The above results are for the bounding canister temperature case (PWR or BWR) for the respective vendor.

TABLE 8.2-1

STORAGE SYSTEM EXTREME ENVIRONMENTAL TEMPERATURE EVALUATION

	HI-STORM Storage System Temperatures (°F)	TranStor Storage System Temperatures (°F)
Ambient Air	125	125
Storage Cask Air Outlet	231	237
Storage Cask Outer Surface	176	157
Storage Cask Inner Concrete	217	249
Canister Outer Surface	352	337
Fuel Clad	790	675

Note: The above results are for the bounding canister temperature case (PWR or BWR) for the respective vendor.

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LIST OF APPENDICES

The PFSF project has been developed on a phased basis. Steps I and II, which involved preliminary investigations, predated the formation of the PFSLLC. Step III began with the formation of the PFSLLC and concluded with the filing of the License Application. This step was funded by direct payments to the PFSLLC from member utilities pursuant to Subscription Agreements. Step IV includes the NRC licensing proceeding as well as detailed design and preparation of bid specifications. The budget for Step IV is approximately $23 million, including contingencies, to be funded by direct payments to the PFSLLC from the member utilities pursuant to Subscription Agreements. These Step IV payments will be made on a quarterly basis. Given the relatively small size of this payment for any participating utility, there is the reasonable assurance that the PFSLLC will obtain Step IV funding.

Step V represents the construction of the PFSF. The budget for this phase is $100 million and includes site preparation; construction of the access road, administration building, security and health physics building, operations and maintenance building, canister transfer building and storage pads; procurement of canister transfer and transport equipment; and transportation corridor construction. The Step V budget also includes necessary personnel costs, licensing fees, and host benefits, as well as a contingency amount.

Step V will be funded through several mechanisms. An additional $6 million in equity contributions is planned from PFSLLC members pursuant to Subscription Agreements. The bulk of the Step V costs is expected to be funded through Service Agreements with PFSF customers (including both PFSLLC members and non-members). Payments under each Service Agreement will be spread out over the period of time from construction through spent fuel delivery. No construction will proceed unless Service Agreements committing for a significant quantity of spent fuel storage have been signed. Raising the non-equity portion of Step V costs through Service Agreements will allow the PFSLLC to avoid financing costs for construction. The PFSLLC, however, retains the option to finance the non-equity portion of Step V costs through debt financing secured by Service Agreements. Construction of the facility shall not commence until funding (equity, revenue, and debt) is fully committed that is adequate to construct a facility with the initial capacity as specified by the PFS to the NRC. Construction of any additional capacity beyond this initial capacity amount shall commence only after funding is fully committed that is adequate to construct such additional capacity (Parkyn 1998, and Gaukler 1999). Therefore, unless PFSLLC members and non-members have committed to a significant quantity of storage, construction of the PFSF will not begin. Thus, there will be reasonable assurance that the PFSLLC will obtain Step V funding.

Step VI, the operational phase of the PFSF, will also be funded through the Service Agreements. The significant costs of this phase will include procurement and/or fabrication of canisters ($432 million) and storage casks ($134 million). These components will be obtained on an as-needed basis, to coincide with the schedule for moving spent fuel to the PFSF. All capital costs associated with the storage of any spent fuel will be paid by the customer pursuant to the Service Agreement prior to the acceptance by the PFSLLC of that spent fuel. PFSLLC shall not proceed with operation of the facility unless it has in place long-term Service Agreements with prices sufficient to cover the operating and maintenance costs of the facility, for the entire term of the Service Agreements, including amortization of any debt used to construct the facility (Gaukler 1999). Since the PFSF will not accept spent fuel for storage without prior payment through Service Agreements of the necessary capital costs for transportation and storage, there is reasonable assurance that the PFSLLC will obtain the necessary Step VI costs.

The on-going operations and maintenance cost for spent fuel in storage at the PFSF will be paid by the customer on an annual basis as required by the Service Agreements. The annual operations and maintenance cost is estimated to be $49 million for a 20-year facility operating life and $31 million for a 40-year life. The elements that make up the estimated annual operation and maintenance costs include the following: labor, operations support, storage canisters, storage casks, transportation fees, transport and storage consumables, maintenance and parts, regulatory fees, quality assurance and other expenses, low-level radioactive waste disposal, contingencies, radiological decommissioning funds, non-radiological decommissioning fund, and associated costs of operating a facility. Note that the O&M costs of $49 million per year for a 20 year facility life and of $31 million per year for a 40 year life include such high-priced items as the storage system canisters / casks and shipping rates. When these canister fees are extracted, the routine annual O&M costs are approximately $10 million per year. The O&M costs noted above are based on a nominal design capacity case of 15,000 Mtu. All dollars expressed are in current year dollars at the time of the license application submittal (1997).

The customers of PFS will be signing Service Agreements, which will include escalators that are tied to specific costs of doing business at the site. Services, such as labor and utilities, will be tied to nationally published indices for the regional area in Utah. Costs, such as Nuclear Regulatory Commission and insurance fees, will be escalated at actual escalation numbers. Therefore, customers will be responsible for the actual costs of ensuring operating and maintenance funding for the facility on a year-by-year basis as long as their fuel is stored. Member utilities also sign separate Customer Agreements to ensure that these same restrictions apply.

The Service Agreements will provide assurance for the continued payment of these costs by requiring the customers to provide annual financial information, meet creditworthiness requirements, and, if necessary, provide additional financial assurances (such as an advance payment, irrevocable letter of credit, third-party guarantee, or a payment and performance bond).

1.7 DECOMMISSIONING FUNDING ASSURANCE

The PFSF will be operated under a "start clean, stay clean" philosophy, with contractual obligations in the Service Agreement with each customer and PFSF administrative procedures to assure that no radioactive contamination is introduced into the facility. Thus the intention is to maintain the PFSF free of radiological contamination at all times. During the operational phase of the facility, all radioactive contamination will be removed immediately upon its discovery. The cost estimate for decommissioning nonetheless conservatively assumes that certain areas and components will require decontamination.

The method of funding decommissioning activities consists of two components: storage cask decommissioning and decommissioning for the remainder of the facility. The costs for decommissioning each storage cask is estimated at $17,000. This amount will be prepaid into an externalized escrow account under the Service Agreement with each customer, prior to shipment of each spent fuel canister to the PFSF. The full amount of potential decommissioning costs will thus be collected in a segregated account prior to the receipt of each spent fuel canister at the PFSF. This method of funding provides for prepayment of the storage cask decommissioning costs prior to any potential exposure of the storage cask to radiation or radioactive material, and therefore prior to the need for any decommissioning. As storage cask decommissioning is completed, the amount of funds in the escrow account will be adjusted periodically to reflect the remaining decommissioning efforts. This method of funding complies with the requirements of 10 CFR 72.30(c)(1).

The costs of decommissioning the remainder of the facility and site is estimated to be $1,631,000, which will be funded through a letter of credit coupled with an external sinking fund. Customers will be required under the Service Agreements to pay the costs to decontaminate any portion of the facility for which they may be responsible for contaminating. As the actual costs of decontamination and decommissioning are paid into the external sinking fund, the letter of credit will be reduced by an equivalent amount. This funding method complies with the requirements of 10 CFR 72.30(c)(3).

The per-canister fee and the amounts of the escrow account, external sinking fund and letter of credit will be reviewed and adjusted annually to account for inflation and any changes in the scope or cost of decommissioning. The escrow account, letter of credit and external sinking fund will be established in conformance with the guidance of NRC Regulatory Guide 3.66.

1.8 SITE LOCATION AND COMPLETION DATES

The proposed PFSF is located on the Skull Valley Indian Reservation which is within Tooele County, Utah, 27 miles west-southwest of Tooele City. The site is located 1.5 miles west of the Skull Valley Road. It is anticipated that the PFSF will be issued a specific license to receive, transfer and possess spent fuel in accordance with the requirements of 10 CFR 72 prior to June 2002 in order to commence operation of the PFSF. Construction of the PFSF is scheduled to start in September 2000, with completion by December 31, 2001. The construction and preoperational testing will be completed in time to support operation of the facility in 2002.

The construction start date of September 2000 is a schedule delay of nine months from that reported in the original License Application (dated June 20, 1997). This start date has been changed to be consistent with the NRC staff review schedule and issuance dates for the Safety Evaluation Report and Final Environmental Impact Statement. Although consistent with the NRC staff review, an exemption to 10 CFR 72.40 may be required to initiate construction activities prior to the license issuance. If determined to be necessary, a request for an exemption will be submitted in compliance with 10 CFR 72.7.

1.9 RESTRICTED DATA

This application does not contain any Restricted Data or other defense information, and it is not expected that any will be included in the future. The applicant agrees that it will not permit anyone to have access to such information if it does become included and will not permit any individual to have access to Restricted Data until the Office of Personnel Management, the successor to the Civil Service Commission, shall have made an investigation and a report to the Nuclear Regulatory Commission on the character, association, and loyalty of such individual, and the Nuclear Regulatory Commission shall have determined that permitting such person to have access to Restricted Data will not endanger the common defense and security of the United States.

1.10 COMMUNICATIONS

It is requested that all communication pertaining to this application be sent to:

John D. Parkyn
Chairman of the Board
Private Fuel Storage L.L.C.
PO Box C4010
La Crosse, WI 54602-4010

PRIVATE FUEL STORAGE L.L.C.

Directors

June 1997

Ronald Cocherell
Manager Nuclear Fuel Services
Southern Nuclear Operating Company

Richard S. Phares
Manager Nuclear Assessment
Illinois Power

Robert W. Keaten
Vice President - Engineering
GPU Nuclear

Stephen Quinn
Vice President of Nuclear Power
Consolidated Edison

John D. Parkyn
Vice President
Genoa FuelTech, Inc.

Paul D. Myers
Manager of Nuclear Fuel Management
Southern California Edison

Doug H. Malin
Nuclear Fuel Manager
Indiana Michigan Power

Edward L. Watzl
President - NSP Generation
Northern States Power

Principal Officer of the Company

John D. Parkyn
Chairman of the Board

Chapter 1 References

Gaukler, P.A., Shaw Pittman. Private Fuel Storage – Docket No. 72-22 – ASLB No. 97-732-02. Letter to the E.M. Julian, U.S. Nuclear Regulatory Commission, December 3, 1999.

Parkyn, J.D., Private Fuel Storage Limited Liability Company. Supplemental Response to RAIs. Letter to Director, Office of Nuclear Material Safety and Safeguards, U.S. Nuclear Regulatory Commission, Docket No. 72-22, September 15, 1998.

APPENDIX C

COMMITMENT LETTERS

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SAFETY ANALYSIS REPORT COMMITMENT LETTERS ISSUED SUBSEQUENT TO THE FINAL RAI RESPONSES

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SAFETY ANALYSIS REPORT COMMITMENT LETTERS ISSUED SUBSEQUENT TO THE FINAL RAI RESPONSES

NOTE: Commitment Resolution Letters #20 and #21 are not included above since they only provided additional information in response to the commitments made in Commitment Resolution Letter #18.

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ENVIRONMENTAL REPORT COMMITMENT LETTERS
ISSUED SUBSEQUENT TO THE FINAL RAI RESPONSES

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Private Fuel Storage, L.L.C.

P.O. Box C4010, La Crosse, WI 54602-4010
Phone 303-741-7009 Fax: 303-741-7806
John L. Donnell, P.E., Project Director

Mark Delligatti, Senior Project Manager
Spent Fuel Project Office
Office of Nuclear Material Safety and Safeguards
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

March 17, 1999

COMMITMENT RESOLUTION
PRIVATE FUEL STORAGE FACILITY
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE L.L.C.

Reference: PFS Letter, Parkyn to Director, Office of Material Safety and Safeguards, Responses to Request for Additional Information, dated February 10, 1999

In accordance with our March 16/17, 1999 telephone calls, Private Fuel Storage submits the following resolution to NRC/CNWRA comments regarding recent PFS Safety RAI responses (Reference).

1. **RAI 4-1 (second round), Thermal Analysis**

   NRC Comment – The PFSF SAR Section 4.2.1.5.2 and 4.2.2.5.2 identifies a design ambient temperature of 110 F, and RAI 4-1 stated: 'Justify the use of the referenced cask systems at a site where the ambient or off-normal conditions appear to be an unanalyzed temperature condition'. The PFS RAI response did not adequately address the issue.

   PFS Resolution - The 110 F referenced is a short-term maximum recorded local event. PFS believes that the response to the RAI addressed this issue, but will provide additional clarification to demonstrate that PFSF site ambient temperatures are enveloped by the Cask vendor's design parameters. PFS plans to submit this written explanation by March 24, 1999.

2. RAI 2-5 (first round), Seismic Analysis

NRC Comment – Some of the equations provided by the Geomatrix report (Sections 6 and 7) in support of the response to RAI 2-5 are not clear with respect to some of the variables used. Also, equations used in conjunction with the Yucca Mountain studies are not clear as to their applicability to the PFSF site.

PFS Resolution – Geomatrix will provide a written explanation of each of the equations (and variables) used. Geomatrix will also provide a written discussion on the applicability of the Yucca Mountain studies to the PFSF site. PFS plans to submit this information by March 24, 1999.

3. RAI 8-2 (second round), Onsite Explosion Analysis

NRC Comment – The PFS SAR (Chapter 8) has adequately evaluated the effects of offsite explosions, but not onsite explosions. The PFSF site will have vehicles and a backup generator using diesel fuel with diesel fuel storage tanks, and building heating systems using propane storage tanks that should be considered.

PFS Resolution – PFS will prepare an assessment of onsite explosions based on the methodology from Reg Guide 1.91 and submit the results. PFS plans to submit this written response by March 24, 1999.

4. RAI's 2-1 & 2-2 (second round), Geotechnical Program

NRC Comment – The PFS field geotechnical investigation program is limited in the number of borings and tests provided. PFS should perform an evaluation of the Standard Penetration Test blow counts (N-values), individually, rather than averaged over 5-ft intervals, to demonstrate that the variability in N-values across the site is not significant. Atterberg Limits, shear strength, and compressibility should also be addressed.

PFS Resolution – PFS will review the variability of N-values across the site and provide a written report of the results. PFS plans to submit this information by March 24, 1999. PFS will also review the available soil samples in the laboratory and perform additional analysis on these samples, providing profiles for Atterberg limits and correlations for shear strength and compressibility. PFS plans to submit this information by March 31, 1999.

If you have any questions regarding this response, please contact me at 303-741-7009.

John C Donnell

John Donnell
Project Director

cc:
John Parkyn
Jay Silberg
Sherwin Turk
Asadul Chowdhury
Murray Wade
Scott Northard
Denise Chancellor
Richard E. Condit
John Paul Kennedy
Joro Walker

Private Fuel Storage, L.L.C.

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Washington, D.C. 20555

March 19, 1999

COMMITMENT RESOLUTION LETTER # 2
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: PFS Letter, Parkyn to Director, Office of Material Safety and Safeguards, Responses to Request for Additional Information, dated February 10, 1999

In accordance with our March 18, 1999 telephone call, Private Fuel Storage submits the following resolution to NRC/CNWRA comments regarding recent PFS Safety RAI responses (Reference).

RAI 2-3 (second round), Flooding Analysis

NRC Comment – The PMF flow rate of 53,000 cfs appears low. "Back of the Envelope" calculations based on other methods indicated that it may be on the order of 150,000 cfs. PFS needs to provide further justification for their PMF approach and results . Two significant parameters that effect the PMF results are the time of concentration and soil conditions (eg, saturated vs non-saturated). PFS needs to justify the models and values used in the PMF calculation.

Also, the PMF analysis (Calculation G(B)-12) needs to consider the effect of the access road and flood diversion berm. Cross Sections A-2 and A-3 which show the water surface profiles should be revised to include the storage pad evaluation.

PFS Resolution - PFS has performed sensitivity studies on the effects of soil conditions and will provide results to show that the effects are minimal. The time of concentration analysis was based on the Hathaway model which considers the drag effect of vegetation on the flow rate. Justification for the use of the Hathaway model will be provided. Additional studies have been performed showing the effect of the berm; these will be finalized and submitted.

PFS plans to submit these additional explanations and studies by March 25, 1999.

If you have any questions regarding this response, please contact me at 303-741-7009.

John L Donnell

John Donnell
Project Director

cc:
John Parkyn
Jay Silberg
Sherwin Turk
Asadul Chowdhury
Murray Wade
Scott Northard
Denise Chancellor
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Private Fuel Storage, L.L.C.

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Mark Delligatti, Senior Project Manager
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Office of Nuclear Material Safety and Safeguards
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

April 2, 1999

COMMITMENT RESOLUTION LETTER # 3
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

References:
1. PFS Letter. Parkyn to Director, Office of Material Safety and Safeguards. Responses to Request for Additional Information, dated February 10. 1999
2. Geomatrix Consultants, Inc., Fault Evaluation Study and Seismic Hazard Assessment, Private Fuel Storage Facility, February 1999
3. Geomatrix Consultants, Inc., Development of Design Ground Motions for the Private Fuel Storage Facility, March 1999
4. Benson. A.K., and Baer, J.L., 1989, Gravity Survey for Skull Valley and Ripple Valley, Tooele County, Utah (from Utah State Technical Report for the Superconducting Super Collider, Appendix E, pp 1-18)

In accordance with our April 1. 1999 telephone call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA comments regarding recent PFS Safety RAI responses (Reference 1).

RAI's 2-5 and 2-7 (first round), Seismic Program

NRC Comment – The PFSF SAR (Appendix 2A) discusses a gravity survey that was performed by Dr. James Baer, BYU. The NRC requested that PFS provide data from this gravity survey. Also, gravity data that was used to support the Geomatrix Report (Reference 2) was requested.

PFS Resolution – The noted Baer gravity data (Reference 4) consist of gravity profiles in the northern part of Skull Valley. Baer's gravity data were not incorporated in the data set for the gravity map in the Geomatrix report (Reference 2, App. E). The profiles show similar gradients to the ones on the data set that were used in App. E. A copy of the applicable pages from Baer's report and the Bouguer Gravity Map (Plate E-1 of Reference 2) will be provided by April 8, 1999. The digitized format of the Plate E-1 data is being expeditiously pursued with the owner of the data and will be provided as soon as possible.

NRC Comment – The NRC requested more information concerning near field effects and directivity and how they are included in the probabilistic seismic hazards analysis.

PFS Resolution – The Geomatrix Report (Reference 3), that is being submitted with the PFS Request for Exemption to 10CFR72.102(f)(1) on April 2, 1999, addresses near field and directivity effects from faults in the vicinity of the PFSF. The ground motion attenuation models incorporate these effects.

NRC Comment – The recent PFS response to RAIs 2-5 and 2-7 only included seismic accelerations based on the probabilistic hazards analysis approach. The staff suggested that PFS also submit PGA values using the deterministic approach that includes the effects of the recently identified faults in the vicinity of the PFSF.

PFS Resolution – PFS plans to submit the results of this deterministic calculation by April 8, 1999.

If you have any questions regarding this response, please contact me at 303-741-7009.

John L Donnell

John Donnell
Project Director

Copy to:

John Parkyn	Scott Northard
Jay Silberg	Denise Chancellor
Sherwin Turk	Richard E. Condit
Asadul Chowdhury	John Paul Kennedy
Murray Wade	Joro Walker

Private Fuel Storage, L.L.C.

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Mr. Mark Delligatti
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Office of Nuclear Material Safety and Safeguards
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

April 14, 1999

COMMITMENT RESOLUTION LETTER #4
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference:
1. PFS Letter, Parkyn to Director, Office of Material Safety and Safeguards, Responses to Request for Additional Information, dated February 10, 1999
2. PFS Letter, Donnell to Delligatti, Submittal of Commitment Resolution Information, dated March 24, 1999
3. PFS Letter, Donnell to Delligatti, Submittal of Commitment Resolution Information, dated March 31, 1999

In accordance with our April 8, 1999 telephone call, Private Fuel Storage submits the following resolution to NRC/CNWRA comments regarding recent PFS Safety RAI responses (Reference 1) and additional comment resolution responses (Reference 2 and 3).

RAI's 2-5 and 2-7 (first round), Seismic Program

NRC Comment – The PFS field geotechnical investigation program is limited in the number of borings and tests provided and relies heavily on the Standard Penetration Test blow counts (N-values) to extrapolate results of laboratory tests to deeper soils within the profile. PFS should further justify that the subsurface conditions at the site are uniform. PFS should explain how the N-values were used in developing the SAR. In addition, PFS should provide profiles of shear strength and compressibility of the soils within the depth interval of 10 ft to ~25 ft. PFS should consider performing additional field tests to determine in situ shear strengths, such as field vane tests or cone penetrometer tests, to determine the extent and thickness of the lower blow count soils on site.

PFS Response – PFS will provide a report identifying how the N-values were used in developing the SAR. PFS will perform additional field work to develop profiles of strength and compressibility of the soils within the depth interval of 10 ft to ~25 ft. This program will include performing cone penetrometer tests (CPT) to develop continuous profiles of the strength of the soils in the upper layer (from the surface down to ~ 25 ft) within the pad emplacement area. Dilatometer tests will be performed using cone penetrometer technology to develop profiles of the compressibility of these in situ soils.

Phase 1 of this program will include performing 36 CPTs, located on a grid pattern of ~300 ft within the entire emplacement area. This pattern will provide nine CPTs in each of the four quadrants occupied by the cask storage pads shown in the attached Figure 1. Several of these CPTs will be located in close proximity to the locations of the borings that were drilled in the emplacement area to permit using the existing boring information in interpretation of the CPT data. The results of the Phase 1 CPTs will include continuous profiles of sleeve stress and tip resistance, which will be used to identify the extent and thickness of the lower blow count soils within the upper layer. These data will also be interpreted to provide profiles of soil stratigraphy, blow count, and strength.

In Phase 2, the cone penetrometer technology will be used to perform dilatometer tests (DMT) to develop profiles of the compressibility of the in situ soils at the locations identified in Phase 1 where the softer soils exist. These data are expected to provide sufficient information on the strength and compressibility of the in situ soils over the entire emplacement area.

PFS will provide a report identifying how the N-values were used in developing the SAR by April 23, 1999. It is anticipated that all of the field activities can be performed during the first two weeks of May and that PFS will be able to report to the NRC the results of this field program by May 31, 1999.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

J L Cooper for

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

Enclosure

cc:
John Parkyn-1/1
Jay Silberg-1/1
Sherwin Turk-1/1
Asadul Chowdhury-1/1
Murray Wade-1/1
Scott Northard-1/1
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Mr. Mark Delligatti
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April 16. 1999

COMMITMENT RESOLUTION LETTER #5
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: 1. PFS Letter. Parkyn to Director. Office of Material Safety and Safeguards, Responses to Request for Additional Information. dated February 10, 1999

In accordance with our April 13. 1999 telephone call, Private Fuel Storage submits the following resolution to NRC/CNWRA comments regarding recent PFS Safety RAI responses (Reference 1).

Emergency Plan (second round), Section 10 Offsite Assistance, Support and Resources

NRC Comment – Provide a general description of the contamination control equipment and supplies which will be maintained at the local hospital to facilitate handling injured personnel who might be radiologically contaminated.

PFS Response – PFS does not currently have any specific arrangements in place with local hospitals to maintain any specific contamination control equipment, but would plan to have such an arrangement in place by the time it begins to receive spent fuel at the site. The type of equipment typically required for treating potentially contaminated personnel with injuries, and which PFS expects would be available for such treatment, is listed below:

- Operating table with side splash shields to contain contaminated liquid and built in drainage capability
- Poly bottles with HEPA filtered vents to collect the water from the operating table

- Wash water for irrigating/decontamination of wounds and other contaminated areas
- Portable "friskers" for detection of Alpha, Beta or Gamma contamination
- Disposable clothing, wipes, surgical sponges, and other surgical instruments typically found in an operating room
- Disposable floor covering and radiological boundary tape for establishing contamination control boundaries
- Poly bags for collecting contaminated material for disposal

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

John L. Donnell

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

cc:
John Parkyn
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May 10, 1999

COMMITMENT RESOLUTION LETTER #6
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In accordance with our May 7, 1999 meeting held in the offices of the Nuclear Regulatory Commission in Rockville, Maryland, Private Fuel Storage submits the following resolution to NRC/CNWRA comments regarding the PFSF flooding analysis (second round safety RAI 2-3).

NRC Comments

- PFS needs to revise the flooding analysis for basin A to use a PMF flow of 85,000 cfs (CN=96 and TOC=11 hrs). Provide a drawing that defines the limits of the floodway for a flow of 85,000 cfs. Provide a drawing showing the profile of the access road and flood diversion berm and show the corresponding water elevations.

- PFS needs to revise the flooding analysis for basin B to calculate the PMF flow using CN=96 (current TOC=4.26 hrs is acceptable). Provide a drawing that defines the limits of the floodway for this new flow. Provide a drawing showing the profile of the rail line and flood diversion berm and show the corresponding water elevations.

- Provide a discussion concerning the design of the culverts or trestle that will be used under the rail line.

- Provide a discussion explaining that there will be no cross-flow between basin A and basin B.

- Provide a statement discussing the planned "freeboard" (berm height above maximum expected water level) for the flood diversion berms.

- Provide a discussion on erosion protection for the flood diversion berms.
- Add a cross-section to the flooding analysis that shows the elevation of the cask storage pads relative to the water elevation at this location.
- Provide a discussion and/or analysis that demonstrates that a breach of the access road or rail line during a PMF event will not increase downstream flood levels.
- Provide a definitive statement in the analysis that PMF flood levels will not contact the cask storage pads.

PFS Response

PFS will revise the flooding analysis to address the issues listed above. SAR figures will be provided that define the limits of the floodway for the new PMF flows and that show the profile of the rail line, access road, and flood diversion berms with the corresponding water elevations.

PFS plans to submit this revised analysis and supporting figures by May 14, 1999. If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

John L Donnell

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

cc:
John Parkyn
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June 24, 1999

COMMITMENT RESOLUTION LETTER #7
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: 1. PFS Letter, Donnell to Delligatti, Submittal of Commitment Resolution #4 Information, dated May 28, 1999.

In accordance with our June 22 and 23, 1999 telephone calls, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA comments regarding the Private Fuel Storage Facility (PFSF) geotechnical program and the potential for impact of aircraft and air-delivered ordnance at the PFSF.

GEOTECHNICAL PROGRAM

The following requests for clarification and additional analyses of the cone penetration testing (CPT) data were generated during the 6/22/99 teleconference:

NRC Comments

1. The cone penetrometer testing (CPT) data that PFS submitted on May 28, 1999 (Reference 1) indicates that the soil classifications based on the CPT data include sandy silts, silty sands, and sands in the upper 25 to 30-ft layer, where PFS previously has reported that this layer consists of uniform silt, silty clay, and clayey silt. PFS should develop a map showing the areas where the sandier soils exist.
2. PFS should demonstrate that the revised profiles in the areas identified in Item 1 have an adequate factor of safety against a bearing capacity failure.
3. PFS should use the CPT data to calculate settlements based on Equation 6-16 or 6-17 of Lunne, Robertson, and Powell (1997), which were developed by Meyerhof and Schmertmann, respectively. These analyses need to include discussion of differential settlements and their relationship to the structural design of the cask storage pads.

4. PFS should explain why the shear wave velocities reported in Appendix C of the CPT report (Reference 1) increase with depth to about 10 to 15 ft and then level off for the remainder of the upper 25 to 30 ft in the profile.
5. PFS should explain why the plots of "PHI" in Appendix G of the CPT (Reference 1) report do not agree with the values of "PHI" shown in the tables of dilatometer data in Appendix H of that report.

Reference: Lunne, T., Robertson, P. K., and Powell, J. J. M., CPT in Geotechnical Practice, Blackie Academic and Professional, 1997.

PFS Response

PFS will provide a report that responds to Items 1-5 identified above by June 30, 1999.

AIRCRAFT CRASHES & AIR-DELIVERED ORDINANCE AT THE PFSF

The NRC Staff requested the following clarifications and additional information during the 6/23/99 teleconference concerning the potential hazard posed to the PFSF by aircraft crashes and the use of air-delivered ordnance in the general region of Skull Valley, Utah.

NRC Comments

1. In accordance with section 3.5.1.6 of NUREG-0800, PFS should produce calculations showing the cumulative crash probability of all the aircraft (military and civilian) flying in Skull Valley, Utah near the PFSF site.

2. PFS should demonstrate that that the structures, systems, and components important to safety at the PFSF can withstand the potential impact of light aircraft by virtue of being designed to withstand the impact of design basis tornado missiles. Alternatively, PFS can perform an analysis to demonstrate that the probability of light aircraft impacting the PFSF is low enough to be considered not credible.

3. PFS should produce documentation from the U.S. Air Force indicating that virtually all military aircraft that transit Skull Valley en route to using the Utah Test and Training Range (UTTR) are F-16s. The documentation should state specifically that the other types of aircraft listed by the Federal Aviation Administration as potentially using the UTTR do not fly through Skull Valley.

4. PFS should address, with respect to the potential hazard from aircraft crashes associated with air operations at Michael Army Airfield, the issue raised in the Lawrence Livermore National Laboratory Report, "Aircraft Crash Assessment of U.S. Nuclear Reactor Power Plant Sites Using the NRC Methodology," UCRL-JC-

128664 (February 20, 1998), that aircraft crashes associated with near-airport operations can occur up to 30 miles away from the airport.

5. PFS should demonstrate, using documentation from the U.S. Air Force regarding the history of military air operations on and around the UTTR, that it is not credible that hung ordnance would strike the PFSF.

6. PFS should demonstrate, using documentation from the U.S. Air Force regarding the history of military air operations on and around the UTTR, that it is not credible that cruise missiles fired or tested on the UTTR would strike the PFSF.

7. PFS should demonstrate, using documentation from the U.S. Air Force regarding the history of military air operations on and around the UTTR, that it is not credible that air-to-air or air-to-ground munitions fired or dropped on the UTTR would strike the PFSF.

PFS Response

PFS will provide a report that responds to Items 1-7 identified above by June 30, 1999.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

J L Cooper

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

cc: Mark Delligatti
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July 9, 1999

SUBMITTAL OF COMMITMENT RESOLUTION LETTER #8 INFORMATION
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In accordance with our July 7, 1999 telephone call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA comments regarding the seismic analysis for the Private Fuel Storage Facility (PFSF).

NRC Comment

Revision 3 of the Safety Analysis Report (SAR) revised the methodology for calculating the design earthquake from a deterministic approach to a probabilistic risk-informed approach and removed the previous discussion on the original deterministic analysis. PFS should resubmit this original deterministic analysis so that it is available for comparison with the new probabilistic approach.

PFS Response

The original deterministic analysis is currently discussed is SAR Section 8.2.1.1 and the actual analysis, performed by Geomatrix Consultants, Inc., is now listed in SAR Section 8.4 as Reference No. 28. This analysis is included as Attachment A to this response.

NRC Comment

PFS should provide an explanation of the basis for using a 1000-year return period for determining design basis ground motion as discussed in SAR Section 2.6.2.3.

PFS Response

An explanation of the basis for using a 1000-year return period for determining design basis ground motion is provided as Attachment B to this response.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

John L. Donnell

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

Attachments

Copies to:
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Asadul Chowdhury
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July 14, 1999

COMMITMENT RESOLUTION LETTER #9
DOCKET NO. 72-22/TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In our phone call this afternoon, the NRC requested two items to clarify materials previously sent to the NRC: a map showing airways in the vicinity of the Private Fuel Storage Facility (PFSF), and the source of information regarding the flight path of F-16s traversing Skull Valley.

Response 1: PFS is providing a map showing the airways in the vicinity of the site to enable one to visualize where the airways, described in General Cole's June 3, 1999 "Risk Assessment of Credible Aircraft or Missile Accidents Impacting [the PFSF)]," are located vis-à-vis the PFSF site. The map is from a 1996 Quantitative Risk Assessment for the Tooele Chemical Agent Disposal Facility (denominated as the "Plant" on the map, but which is not the PFSF) prepared by Science Applications International Corporation for the U.S. Army. We have added to the left side of the map the location of the PFSF site and airway IR 420.

Response 2: PFS is providing a revised page 10 to General Cole's June 3, 1999 report which adds a footnote at the end of the second sentence of the last paragraph on the page providing the source of the F-16 flight path distance information (five miles from the PFSF) and other information in the sentence, and revised pages 30 and 31, which include the new footnote 15A (which is the same as footnote 17).

We trust this letter and the attachments provide the NRC with the information requested in our telephone call this afternoon. If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

John L. Donnell

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

Attachments

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July 22, 1999

COMMITMENT RESOLUTION LETTER #10
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: 1. PFS Letter, Parkyn to Delligatti, Request for Exemption to 10 CFR 72.102(f)(1) Seismic Design Requirement, dated April 2, 1999.

In accordance with our July 21, 1999 telephone call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA comments regarding the Private Fuel Storage Facility (PFSF) seismic analysis and the aircraft crash hazard assessment at the PFSF.

SEISMIC ANALYSIS

The following requests for additional information regarding the PFSF seismic analysis were made during the 7/21/99 teleconference:

NRC Comments

1. For the original deterministic analysis (1997), PFS should provide horizontal and vertical Peak Ground Acceleration (PGA) at the 50th percentile.
2. For the updated deterministic analysis (1999), PFS should provide horizontal and vertical PGA at the 50th percentile.
3. For the probabilistic analysis (1999), PFS should provide horizontal and vertical PGA for the 2000-yr return period.

PFS Response

1. PGA values at the 50th percentile for the original deterministic analysis (1997) are provided in Attachment 1.

2. PGA values at the 50th percentile for the updated deterministic analysis (1999) are provided in Attachment 1.

3. For the probabilistic analysis (1999), PGA values for the 2000-yr return period were provided in Table 1 of the Geomatrix report entitled "Development of Design Ground Motions for the Private Fuel Storage Facility" transmitted in Reference 1. These values are repeated in Attachment 1.

AIRCRAFT CRASH HAZARD ASSESSMENT AT THE PFSF

The NRC raised the following issues concerning the aircraft crash hazard assessment performed by PFS for the Private Fuel Storage Facility during the 7/21/99 teleconference:

NRC Comments

1. Although the current practice is for F-16s to fly down the East side of Skull Valley, five miles from the PFSF site, the F-16s are not physically or legally constrained to follow this route. PFS should seek to determine whether a standing military order or procedure exists concerning avoidance of nuclear facilities that would apply here and would require the F-16s flying down Skull Valley to avoid the PFSF. Alternatively, PFS should show that the F-16 flights do not pose a significant hazard to the PFSF, assuming their paths through Skull Valley are not constrained.

2. PFS has stated that only 70 percent of the total aircraft sorties on the Utah Test and Training Range (UTTR) are flown by F-16s (June 30 Submission, p. 16). Furthermore, PFS has stated that there were a total of 8,711 sorties flown on the UTTR in 1998 (including the 3,871 F-16 flights down Skull Valley) (Cole Report dated June 3, 1999, p. 20). PFS should show that the aircraft sorties flown on the UTTR in addition to the F-16 flights down Skull Valley would not pose a significant crash hazard to the PFSF.

3. PFS's response to RAI 8-2 indicates that aircraft returning to Hill Air Force Base from the UTTR may use the Stansbury and Moser recovery routes, which may take them over or near the PFSF site. PFS should show that aircraft flying those routes would not pose a significant crash hazard to the PFSF.

4. In order to address an issue raised by the State of Utah, PFS should show that air crashes involving military helicopter flights would not pose a significant hazard to the PFSF.

5. PFS should provide a reference that indicates that the crash rates for large military transport aircraft are similar to those for civilian commercial aircraft.

6. PFS should provide a source for the Attachment E provided with the submission of June 30, 1999.

PFS Response

PFS is currently reviewing items 1-5 and will provide an additional commitment letter on July 26.1999 which will outline our plan of action and provide a completion date for these items.

6. The source for Attachment E provided with the submission of June 30. 1999 is Item 2 to the January 20. 1999 Response by the Department of the Air Force. Headquarters Ogden Air Logistics Center (AFMC). Hill Air force Base. to the December 18, 1998 Freedom of Information Act Request filed by General Cole.

If you have any questions regarding this response. please contact me at 303-741-7009.

Sincerely.

John L. Donnell

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

Attachments

Copy to: Mark Delligatti
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July 26, 999

COMMITMENT RESOLUTION LETTER #11
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: 1. July 21, 1999 telephone call between Private Fuel Storage (PFS), Stone and Webster (S&W) and the NRC/CNWRA

2. PFS Letter, Donnell to U.S. Nuclear Regulatory Commission, Commitment Resolution Letter #10, dated July 22, 1999

In Reference 1 the NRC/CNWRA asked several questions concerning aircraft crash hazard assessment at the PFSF. These questions were documented as items 1 through 6 in Reference 2 and a response was provided to item 6. PFS committed to provide a completion date for the remaining 5 items by July 26,1999. PFS has reviewed items 1 through 5 in more detail and has determined that additional information is necessary from the U.S. Air Force in order to respond to these items. The requests for additional information have been made and the information should be received no later than August 9, 1999, which will enable PFS to provide a response by August 13, 1999.

PFS will make every effort to obtain the necessary information as soon as possible and therefore respond prior to the dates listed above. If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

JLCooper for

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

cc:
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July 28, 1999

COMMITMENT RESOLUTION LETTER #12
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

This letter is issued to correct a typographical error in COMMITMENT RESOLUTION LETTER #12 dated July 27, 1999. The error is on page 2, item 2. & 3. In the 5th line the symbol ϕ did not print. The 5th line has been corrected to read "......stress parameters (c and ϕ), based on the results.....". The remainder of the letter is unchanged.

In accordance with our July 23, 1999 telephone call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA comments regarding the Private Fuel Storage Facility (PFSF) Geotechnical Program:

NRC Comments

1. The source of the earthquake forces that are used in Calculation 05996.02-G(C)-13, Rev 0, "Allowable Bearing Capacity of the Canister Transfer Building Supported on a Mat Foundation," is not clear. S&W explained that the earthquake forces are developed in Calculation 05996.02-SC-09, Rev 0. The CNWRA will locate the calculation and review it.

   Nevertheless, it was noted by the CNWRA that calculation 05996.02-G(C)-13 is missing one of the required loading combinations. The missing 100/40/40 combination is the one that includes 40% vertical earthquake down, 100% horizontal earthquake in the north-south direction, and 40% horizontal earthquake in the east-west direction. S&W explained this case was deemed to be not controlling, but agreed to revise the calculation to include it.

2. Foundation Configuration 2 on Page 14 of Calculation 05996.02-G(B)-4, Rev 3, "Stability Analyses of Storage Pads," identifies the pad as being constructed on top of compacted crushed stone or gravel. The calculation should indicate the thickness of the cohesionless layer. S&W explained that all of the pads are to be constructed on in situ material (cohesive soil) and the reference to cohesionless soil will be deleted from the calculation.

3. PFS should revise the determination of the sliding resistance of the foundations to utilize a shear strength for the soils that is dependent on the normal stress, which is decreasing during portions of the earthquake.

4. PFS should use a consistent coefficient of friction between the cask and the pad in determining the overturning stability of the pad as is used in the tipover analysis of the cask.

5. PFS should update SAR Figure 2.6-5, "Foundation Profile A-A'," to incorporate the results of the recently performed cone penetration testing, which indicates that there are three layers within the upper ~25-ft thick layer at the site.

PFS Response

1. Calculation 05996.02-G(C)-13 will be revised to include the specified load combination.

2.&3. Calculations 05996.02-G(B)-4 and 05996.02-SC-09 will be revised to address these two items. In these revised analyses, the driving forces will be determined based on the revised design earthquake (peak horizontal ground acceleration = 0.4g and peak vertical ground acceleration = 0.39g, SAR Section 3.2.10.1.1). The soil strengths will be defined based on total stress parameters (c and ϕ), based on the results of the triaxial tests that have been performed to date.

4. PFS will review the cask and storage pad analyses and will provide any necessary clarification to ensure that the coefficient of friction between the casks and the pad is incorporated in a consistent manner.

5. PFS will incorporate the results of the cone penetration testing on SAR Figure 2.6-5.

PFS will provide calculations and necessary responses to the above items by August 6, 1999. If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

JL Cooper for

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

cc:
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John Parkyn
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July 30, 1999

COMMITMENT RESOLUTION LETTER #13
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: 1. PFS Letter, Parkyn to Delligatti, Proprietary Responses to EIS RAIs, dated February 18, 1999.

In a telephone call between the NRC, Stone and Webster (S&W), and Private Fuel Storage (PFS) on July 16, 1999 the NRC asked several questions regarding the PFS Field Investigation Evaluation Report and the PFS Business Plan. The NRC questions and the PFS resolutions are provided below.

FIELD INVESTIGATION EVALUATION REPORT

NRC Comment

The PFS Field Investigation Evaluation Report was submitted to the NRC (Reference 1) as a Proprietary Attachment to EIS RAI 6-2. The NRC has reviewed this report and believes that Sections 3 and 4 do not contain proprietary information.

PFS Response

PFS agrees that Sections 3 and 4 of this report can be considered non-proprietary. A non-proprietary version of the Field Investigation Evaluation Report dated August 7, 1996 is enclosed.

PFS BUSINESS PLAN

NRC Comment

The PFS Business Plan has been submitted to the NRC as a proprietary document. The Business Plan contains information on the site selection process that the NRC believes should be in the PFSF Environmental Report (ER). PFS should review the Business Plan and incorporate as much information as possible on the site selection process into the ER.

PFS Response

Additional information on the site selection process will be included in License Application Amendment #4. which will be issued during the first week of August 1999.

If you have any questions regarding this response. please contact me at 303-741-7009.

Sincerely,

JL Cooper for

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

Enclosure

cc:
Mark Delligatti
John Parkyn
Jay Silberg
Sherwin Turk
Asadul Chowdhury
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August 6, 1999

COMMITMENT RESOLUTION LETTER #14
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference 1. PFS letter, Donnell to Delligatti, Request for Exemption to 10 CFR 72.102(f)(l) Seismic Design Requirement, dated April 2, 1999

2. PFS letter, Donnell to U.S. NRC, Commitment Resolution Letter #12, dated July 28, 1999

In accordance with our August 4, 1999 telephone call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA comments regarding the Private Fuel Storage Facility (PFSF) design earthquake and geotechnical program:

NRC Comments

1. PFS should consider using a design earthquake that is based on a Probabilistic Seismic Hazard Analysis (PSHA) with a return frequency of 2000 years. Alternatively, PFS could submit additional regulatory and technical basis information to justify the use of a 1000-year return period.

2. In order to demonstrate an adequate safety factor to resist sliding, the cask storage pad and canister transfer building stability analyses should use values of c and ϕ appropriate for the soils at the founding level of the structures. The slip failure plane should be investigated and discussed in the analyses also.

3. The cask storage pad design analysis should consider the variability of the in situ soils.

4. PFS should address post earthquake inspection procedures for equipment important to safety.

PFS Response

1. In order to include additional conservatism in the Private Fuel Storage Facility (PFSF) design, PFS will revise the design earthquake to utilize the PSHA approach with a return frequency of 2000 years. A license amendment reflecting this change will be prepared and submitted by August 20, 1999. Additionally, the seismic exemption request submitted in Reference 1 will be revised to state that for additional conservatism PFS has chosen to use a return frequency of 2000 years. A revised seismic exemption request will also be submitted by August 20, 1999.

2.&3. PFS will revise and submit the calculations listed below by August 20, 1999. These calculations will address the foundation sliding concerns as well as the concerns in items 1 through 5 of Reference 2. Please note that our previous response date provided in Reference 2 must be revised to August 20, 1999 due to the change in the design earthquake as discussed in item 1 above.

- Geomatrix Calculation, Soil and Foundation Parameters for Dynamic Soil Structure Interaction Analyses
- Geomatrix Calculation, Development of Acceleration Time Histories for the Design Earthquake
- S&W Calculation, Stability Analyses of Storage Pads
- S&W Calculation, Stability Analyses of the Canister Transfer Building Supported on a Mat Foundation
- S&W Calculation , Development of Soil Impedance Functions for the Canister Transfer Building
- S&W Calculation, Seismic analysis of the Canister Transfer Building
- Holtec calculation, multi-cask response at the PFS ISFSI from 2000 year seismic event

4. The PFSF Emergency Plan (EP) and the Technical Specifications provided in Appendix A of the License Application discuss a seismic event affecting the PFSF site.

EP Section 2.4.1, item 5 states:

"A seismic event affecting the PFSF site does not constitute an emergency condition. The cask storage systems, Canister Transfer Building, canister transfer cranes, and canister downloader are capable of withstanding earthquake conditions, and a design basis ground motion will not cause a canister drop, cask toppling event, or loss of safety functions. Only earthquakes exceeding the design basis ground motion are classified as an emergency condition. Information on the

magnitude of a seismic event is obtained from the National Earthquake Information Center in Golden. Colorado."
EP Section 2.4.2, item 4 states:

"A seismic event exceeding the design basis warrants the Alert classification. Information on the magnitude of a seismic event is obtained from the National Earthquake Information Center in Golden, Colorado. The storage pads and storage cask systems are designed to withstand the design basis ground motion, and analyses demonstrate the storage casks will retain their stability and not tip over during a seismic event of this magnitude. In addition, the Canister Transfer Building, canister transfer cranes, and canister downloader are designed to withstand the design basis ground motion. Therefore, a seismic event of this magnitude will not cause toppling of storage or transfer casks and there will be no loss of canister confinement nor loss of safety functions. However. an earthquake that exceeds the design basis ground motion has the potential for degradation of the level of safety, and the Alert classification is appropriate to mobilize personnel to investigate effects of the event. This <u>is not</u> considered a credible event."

EP Section 3.2, item B states:

"Cask Tipover Accident: This is not a credible event, since it would require an event such as an earthquake or tornado of a magnitude well beyond the design basis. The concern with this event would be the potential loss of heat removal capacity, since the natural convection design exists when the cask is oriented vertically. Mitigation of this event is provided by the design of the storage casks, which provides 30 hours (assuming a complete blockage of all air ducts) to restore natural convection cooling before temperatures exceed design criteria. A crane with the capacity to upright a toppled storage cask would be temporarily procured and the toppled storage cask placed back on the storage pad in its vertical orientation within 30 hours. A visual inspection and radiological survey of the cask would then be performed to verify its integrity."

Technical Specification, item 2.6, "Action", states:

"The center-to-center and distance to edge of pad spacing shall be measured upon initial storage cask placement. After a seismic event of magnitude greater than 5.0 Richter at the PFSF, as determined by the National Earthquake Information Center, Golden, CO., verify spacing specified above. If required, restore center-to-center and distance to edge of pad spacing."

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

JS Cooper for

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

cc:
Mark Delligatti
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August 6, 1999

COMMITMENT RESOLUTION LETTER #15
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference

1. PFS letter, Parkyn to Director, Office of Nuclear Material Safety and Safeguards, Response to Request for Additional Information, dated February 10, 1999

2. PFS letter, Donnell to Delligatti, Commitment Resolution Letter #5, dated April 16, 1999

In accordance with our August 5, 1999 telephone call, Private Fuel Storage (PFS) submits the following resolution to NRC comments regarding the Private Fuel Storage Facility (PFSF) Emergency Plan (EP):

NRC Comments

The RAI responses provided in Reference 1 adequately address the NRC questions/comments. However the NRC believes that the responses to the following RAIs need to be incorporated into the EP:

RAI EP-2	RAI EP-12
RAI EP-4	RAI EP-13
RAI EP-7	RAI EP-21
RAI EP-9	RAI EP-22
RAI EP-10	RAI EP-23
RAI EP-11	RAI EP-24

Additionally, the response provided in Reference 2 regarding a general description of the contamination control equipment and supplies which will be maintained at the local hospital to facilitate handling injured personnel who might be radiologically contaminated should also be included in the EP.

PFS Response

PFS will incorporate the above requested information into the EP by August 27. 1999.

If you have any questions regarding this response. please contact me at 303-741-7009.

Sincerely

JL Cooper for

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

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John D. Parkyn, Chairman of the Board

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August 10, 1999

COMMITMENT RESOLUTION LETTER #16
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In accordance with our August 6, 1999 telephone call, Private Fuel Storage (PFS) submits the following resolution to NRC comments regarding the Private Fuel Storage Facility (PFSF) License Amendment scheduled for submittal August 20, 1999:

NRC Comments

1. PFS should identify in the Safety Analysis Report (SAR) what organization is responsible for construction as required by NUREG–1567.

2. PFS should include design details and analysis results for the seismic struts that are utilized during the canister transfer operation. An analysis should also be provided evaluating the loads on the canister transfer cell walls from these struts.

3. PFS should discuss possible uplift resulting from a seismic event for the overhead bridge crane and semi-gantry crane in the Canister Transfer Building.

PFS Response

PFS will incorporate the above requested information into the License Amendment and calculation submittal scheduled for August 20, 1999.

If you have any questions regarding this response, please contact me at 303-741-7009.

John L. Donnell

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

cc:
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September 3. 1999

COMMITMENT RESOLUTION LETTER #17
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In the August 19, 1999 telephone call between Private Fuel Storage (PFS) and the NRC, the NRC asked additional questions regarding projected future growth in civilian and military air traffic. use of flight termination systems for cruise missiles. and information on a recent cruise missile incident. The NRC questions and the PFS response are provided below:

NRC Comments

1. PFS should provide any available information on the cruise missile incident of June 11, 1999.

2. PFS should include projected air traffic growth rates (civilian and military) in the aircraft crash hazard assessment for the Private Fuel Storage Facility (PFSF).

3. PFS should address the issue of apparently conflicting information from the Air Force concerning use of the cruise missile Flight Termination System on the UTTR.

PFS Response

PFS response to the above comments is enclosed. If you have any questions regarding this response, please contact me at 303-741-7009.

John L. Donnell

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

Enclosure

Copy to (with enclosure):

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October 13, 1999

COMMITMENT RESOLUTION LETTER #18
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In accordance with our October 7, 1999 conference call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA questions and comments regarding the aircraft crash hazard assessment for the Private Fuel Storage Facility (PFSF).

NRC Questions and Comments

1. The NRC raised questions about PFS's use of "normal" rather than "special operations" aircraft crash rates from DOE-STD 3014-96 for the F-16s flying down Skull Valley and requested further justification from PFS for its use of the normal crash rate.

2. The NRC questioned PFS's approach of using a time-weighted average area for the cask storage area when calculating the air crash impact hazard to the PFSF instead of evaluating the hazard on an annual basis.

3. The NRC questioned why PFS analyzed the air crash impact hazards for the cask storage area and the canister transfer building independently and requested PFS to provide further justification for such independent treatment or for PFS to alternatively treat the cask storage area and the canister transfer building together.

4. In light of the above questions, the NRC also questioned the overall likelihood of an air craft crash impacting the PFSF, and the NRC requested PFS, should it be unable to show the lack of any credible hazard from aircraft crashes, to identify the type and quantity of live ordnance carried by F-16s flying down Skull Valley and assess the potential consequences of an F-16 carrying live ordnance crashing at or nearby the PFSF. Also, to further support the Air Force's statement of no inadvertent release of ordnance, the NRC requested PFS to show, if possible, how many flights on the UTTR have taken place without an inadvertent release of ordnance.

5. The NRC raised questions about the speeds at which crashing aircraft would impact the spent fuel storage casks and requested PFS to clarify and further address, if possible, this issue.

PFS Response

PFS is currently reviewing the foregoing items and will provide an additional letter responding to the NRC's questions and comments on October 22, 1999. PFS submitted Freedom of Information Act Requests to the U.S. Air Force on Friday October 8, 1999, requesting certain information to be used in preparing the responses. It is possible that PFS may not receive this information in time to be included in its October 22, 1999 response. In such an event, PFS would supplement its October 22, 1999 response as necessary upon receipt of the information from the U.S. Air Force.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

JL Cooper for

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

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October 15, 1999

SUBMITTAL OF COMMITMENT RESOLUTION
LETTER #19 INFORMATION
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In accordance with our October 4, 1999 conference call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA questions and comments regarding the cask storage pad sliding analysis for the Private Fuel Storage Facility (PFSF).

NRC Questions and Comments

- The NRC questioned the use of soil cohesion at the concrete-soil interface to resist the sliding force and how it would be achieved during pad construction.
- The NRC suggested a positive locking mechanism, such as a key, be considered to resist sliding.

PFS Response

The PFS response to the above questions/comments is enclosed. If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

John L. Donnell

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

Enclosure

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November 19, 1999

COMMITMENT RESOLUTION LETTER #22
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

A meeting was held on November16, 1999 in Salt Lake City, UT between Private Fuel Storage (PFS), Stone and Webster (S&W), and the NRC/CNWRA to discuss several geotechnical issues, as well as questions regarding the aircraft and ordinance crash hazard assessment for the Private Fuel Storage Facility (PFSF). After considerable discussion on both topics, it was determined that PFS would take the action outlined below to resolve outstanding issues.

GEOTECHNICAL

PFS will issue an amendment to the License Application by December 10, 1999 that will provide the following information:

- PFS will revise the SAR to expand the explanation of how the results of the cone penetration tests (CPT) were interpreted by ConeTec, Inc to determine the soil behavior types (SBT) that are presented in their report (ConeTec, 1999). The SBTs, which are functions of the tip resistance values and friction ratios, were determined based on a soil behavior type classification chart (presented on page 9 of their report) that was developed based primarily on saturated uncemented soils. The SBTs reported by ConeTec for the site soils are biased towards the sandier soil types because these soils are partially saturated and cemented. Both of these factors result in higher tip resistance values and lower friction ratios than would be the case for saturated uncemented soils. PFS will demonstrate that by applying the results from the other tests performed specifically for the purpose of classifying the soil types (Atterberg limit testing, direct shear tests, and triaxial shear tests) the CPT data can be correlated to the previously reported soil classifications.

- PFS will revise the SAR to provide 13 additional foundation profiles, similar to Figure 2.6-5, showing the results of the soil borings and CPTs. These profiles will use the measured Q_t from the CPTs to further distinguish layering within the upper 25 to 30-ft depth of the profiles and will use the classifications determined based on Atterberg tests of the soil samples obtained in the borings to identify soil types.

- PFS will revise the SAR to include a discussion on the use of cement to stabilize the layer of eolian silt existing at the ground surface at the site such that it is suitable for use as base material beneath and between the cask storage pads. The discussion will be similar to that provided in PFS letter, Donnell to U.S. NRC, Submittal of Commitment Resolution Letter #19 Information, dated October 15, 1999, but it will address the following key issues in greater detail:

    1. Revision of Figure 4.2-7 to show the design with the cask storage pads founded on and within the soil-cement.
    2. Provide a description of the standard, industry-accepted field and laboratory testing program that will be utilized to design the soil-cement mix necessary to develop the required strength within the soil-cement and between the concrete pad and the soil-cement.
    3. Include a commitment to perform this standard, industry-accepted testing during detailed facility design.
    4. Provide technical justification including published references demonstrating that a bond exists between the concrete storage cask pad and the soil-cement that resists sliding of the pad during a seismic event.
    5. Provide a description of standard construction techniques that are used to obtain this bond.

References

ConeTec, 1999, Cone penetration testing report, Private Fuel Storage Facility, prepared for Stone and Webster Engineering Corp., Denver, CO, 2 volumes.

AIRCRAFT AND ORDINANCE CRASH HAZARD ASSESSMENT

PFS will consolidate all the previous submittals on this issue into one consolidated report that contains the latest information as well as the clarifications requested during the November 16, 1999 meeting. PFS will provide this report to the NRC Staff by November 24, 1999. Future modifications, if any, to PFS's analysis of aircraft crash hazards will be incorporated in an update to this consolidated report.

PFS has obtained access to the area planning guide for military training routes for North and South America, DoD Flight Information Publication AP/1B, 4 NOV 1999. By

November 24, PFS will provide and discuss material from the guide relevant to nuclear facilities as it may apply to the PFSF.

The discussion provided in SAR Chapter 2 on hazards from aircraft crashes will be updated by December 10, 1999 to include the latest information and to reference the new consolidated report on hazards from aircraft crashes.

PFS is still investigating further the use of Army multiple launch rocket systems on Dugway Proving Ground. We have submitted a Freedom of Information Act request to Dugway asking for information on that subject and is also pursuing information by other means. Upon receiving relevant information, PFS will provide it to the NRC Staff.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

John L Donnell

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

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January 7, 2000

COMMITMENT RESOLUTION LETTER #23
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
<u>PRIVATE FUEL STORAGE L.L.C.</u>

In accordance with our January 5. 2000 conference call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA questions and comments regarding geotechnical issues for the Private Fuel Storage Facility (PFSF).

NRC Questions and Comments

1. The discussion in Section 2.6 of the SAR needs to be revised such that it clearly reflects the proposed cask storage pad foundation design as shown in SAR Figure 4.2-7. The calculations that support the discussion in this section also need to be revised to clearly reflect the proposed design.

2. The NRC suggested that Tables 1. 2, and 3 on pages 50, 51, and 52 respectively of calculation 05996.02-G(B)-04. Rev 4 and the Table on page 34-1 of calculation 05996.02-G(B)-13. Rev 1 be updated and summarized in applicable sections of the SAR.

PFS Response

1. PFS will update Calculation 05996.02-G(B)-04 Revision 4 to incorporate the beneficial effects of using soil cement beneath and around the pads in the stability analyses of the cask storage pads and revise SAR Chapter 2 accordingly. This update will address the increased resistance provided by the presence of the soil cement to both sliding and overturning (dynamic bearing capacity) of the cask storage pads due to the design basis ground motion. It will also include the revised vertical soil bearing pressures and horizontal shear forces applicable for the 2,000-yr return period design ground motion. which were determined in Calculation 05996.02-G(PO17)-2, Rev 1.

2. PFS will add discussion of the overturning (dynamic bearing capacity) analyses of the Canister Transfer Building (summary presented on page 34-1 of Calculation 05996.02-G(B)-13, Rev 1) to SAR Chapter 2.

PFS will provide these analyses and updates in Revision 9 of the SAR, which is scheduled to be submitted January 24, 2000. If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

John L. Donnell

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

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January 14, 2000

COMMITMENT RESOLUTION LETTER # 24, SER OPEN ITEMS
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference:
1. December 10, 1999 telephone call between Private Fuel Storage (PFS), Stone and Webster (S&W) and the NRC
2. NRC Letter, Delligatti to Parkyn, "Safety Evaluation Report For Systems Not Directly Associated With Storage Casks (TAC No. L22462), dated December 15, 1999
3. January 7, 2000 telephone call between Private Fuel Storage (PFS), Stone and Webster (S&W) and the NRC/CNWRA

On December 10, 1999 a conference call was held between the NRC, Private Fuel Storage (PFS) and Stone and Webster (S&W). The purpose of the call was for the NRC to identify open items/outstanding issues remaining in their review of the Private Fuel Storage Facility (PFSF) Safety Analysis Report. The open items discussed during this call were those that are not listed as open items in the Safety Evaluation Report (SER) issued with the Reference 2 letter. A subsequent phone call was held, Reference 3, to discuss the open items in greater detail. The open items discussed in the phone calls, References 1 and 3, and the proposed resolutions are provided below as items 1 through 23. The open items identified within the SER, Reference 2, and the proposed resolutions are provided below as items 24 through 30. The scheduled resolution date for each open item is provided with the response.

The following open items are associated with Chapter 5, "Installation and Structural Evaluation", of the SER:

Canister Transfer Building (CTB)

1. It appears the CTB was analyzed for the worst-case loading condition only. PFS should provide more detailed information on the other load cases that are described in Section 3 of the PFSF SAR.

   RESPONSE - Because of the high seismic loads and relatively low or non-existent loads from other abnormal conditions (e.g. tornado and accident pressure loads), the seismic loads will govern the design. The SAR will be revised to provide a discussion of all the load cases considered in the design of the CTB and why these two load cases are considered to govern the design. As stated in SAR Section 4.7.1.5.1, "During the detailed design phase, all load cases as described in Chapter 3 and all areas will be addressed in detail."

   This information will be included in a License Amendment submitted by January 24, 2000.

2. PFS should verify that the worst-case load combination for all critical areas is seismic.

   RESPONSE - See response to Item 1 above.

3. Crane loads provided by the Vendor and used in the CTB design will increase due to the latest dynamic soil properties. Has this been included in the design (check NUREG 0612)?

   RESPONSE – See response to Item 14 below.

4. Provide design information and define critical elements of major CTB components, including cask loading/unloading bay, 3 cask transfer cells, 200-ton overhead bridge crane, 150-ton semi-gantry crane, crane runway, girders and supports, cask transporter bay, tornado missile barriers, LLW storage room, radiation shield walls and doors, equipment laydown areas, storage cask delivery and staging area, and mechanical/electrical equipment areas.

   RESPONSE – After discussion of the responses to Items 1, 2, 6 through 10, and 12, it was agreed that these responses and associated SAR updates adequately demonstrate the structural adequacy of the CTB. Regarding other details of the building, PFS will revise the SAR to include identification and discussion of radiation shield walls, tornado missile barriers, and fire rating of walls and doors. PFS will review the information currently provided in the SAR for the CTB mechanical and electrical systems to ensure that the industry codes and standards that will be used for design and construction are clearly identified. Additionally PFS will ensure that information provided for both cranes includes a discussion on potential liftoff during a seismic event and crane operability after a seismic event (see Item 13, 14, and 15 also).

This information will be included in a License Amendment submitted by January 24, 2000.

5. Provide details of the large sliding doors in the canister transfer cell walls. These are provided for shielding, and PFS must demonstrate that they will stay in place during all loading conditions.

   RESPONSE – The SAR will be revised to include a discussion of the design requirements (i.e., seismic, radiation shielding, and fire rating) and applicable industry codes and standards for these doors.

   This information will be included in a License Amendment submitted by January 24, 2000.

6. PFS should confirm that the seismic analysis has been updated to reflect the latest Probabilistic Seismic Hazard Analysis.

   RESPONSE - S&W Calculation entitled "Seismic analysis of Canister Transfer Building", (Calculation number 0599602-SC-5, Revision 1) was submitted to the NRC via letter, Donnell to U.S. NRC, dated September 9, 1999. This calculation reflects the latest Probabilistic Seismic Hazard Analysis (i.e., the latest soil properties and 2,000 year Return Period Design Ground Motions). Additionally the latest Probabilistic Seismic Hazard Analysis was included in SAR revision 5.

7. PFS should confirm that the seismic analysis has been updated to reflect the latest soil data/investigations that were used to develop the soil impedance functions in the model.

   RESPONSE - S&W Calculation entitled "Development of Soil Impedance Functions for Canister Transfer Building", (Calculation number 0599602-SC-4, Revision 1) was submitted to the NRC via letter, Donnell to U.S. NRC, dated September 9, 1999. This calculation develops impedance functions using the latest soil data/investigations. These impedance functions were then used as inputs for the seismic analysis (Calculation number 0599602-SC-5, Revision 1) of Canister Transfer Building. The latest soil properties were included in SAR revision 6.

8. The CTB was modeled using a 3-D lumped mass system. Does the lumped model account for local response of the roof and wall panels under seismic loading?

RESPONSE – Yes. Lumped mass points 5 and 6 of the model developed in the "Seismic Analysis of Canister Transfer Building", (Calculation number 0599602-SC-5, Revision 1) were included to account for local flexibility of the roof and wall panels, respectively. This is described in SAR Section 4.7.1.5.3 in the 5th paragraph.

9. Dynamic analysis was performed using the computer program "FRIDAY" to develop response spectra at critical locations. Results were integrated into the 3-D model using the ANSYS program. Why was the 3-D model not subjected to the earthquake time history input to develop stress levels automatically?

   RESPONSE – The ANSYS model does not properly account for dynamic soil-structure interaction effects, such as input of free field ground motions and radiational damping. The "FRIDAY" program has the capability to include soil impedance functions that do account for soil-structure interaction effects. The soil impedance functions are developed from the dynamic soil properties using the S&W program "REFUND". The input ground motion is applied at the free field.

10. In the ANSYS analysis, only 2 load cases were presented. Provide justification for ignoring other load cases with supporting details.

    RESPONSE – The two load cases selected for preliminary design were chosen because they produce the worse downward loading and the worst overturning loads. These two cases will envelope the design of the building. The SAR will be revised to provide a discussion of all the load cases considered in the design of the CTB and why these two load cases are considered to govern the design. All additional load cases will be documented in the final design calculations.

    This information will be included in a License Amendment submitted by January 24, 2000.

11. Results of the ANSYS analysis were used in design of reinforced concrete. Design of overall structure was provided, but not for local areas. Provide design details for all openings & wall/roof interfaces.

    RESPONSE – After the additional explanation provided during the discussion of Items 1, 2, 6 through 10, and 12, it was agreed that these responses and associated SAR updates adequately demonstrate the structural adequacy of the CTB. Design details for all local areas are not required at this time. As stated in SAR Section 4.7.1.5.1, "During the detailed design phase, all load cases as described in Chapter 3 and all areas will be addressed in detail".

Earthquake Duration

12. PFS must demonstrate that a 15-second earthquake is appropriate and conservative, considering standard engineering practice and the seismic event at the PFSF site. Conclusions in the SAR are based on a deterministic time history approach using a 15-second Italian earthquake as being representative of the site. Based on other recent seismic events, with 30 to 40 second earthquakes, confirm that the earthquake duration is justified. (Ref 10CFR72-102.a,b, f).

    RESPONSE –SAR Section 4.7.1.5.3 describes the development of the time histories used in the design of the CTB. A 3-D artificial earthquake of 30 seconds was developed to simulate the ground motion. The earthquake duration was reduced to 20 seconds for analysis of the Canister Transfer Building. The time histories meet the requirements of the Standard Review Plan (NUREG 0800) and ASCE 4-86. Since a linear elastic analysis was performed, an earthquake record longer than a 20-second duration would not have any additional effect on results. The "Seismic Analysis of Canister Transfer Building", (Calculation number 0599602-SC-5, Revision 1) describes the input of the time histories to the analysis of the Canister Transfer Building.

Crane Design

13. PFS must demonstrate the structural integrity and functionality of the overhead bridge and semi-gantry cranes under appropriate seismic loadings and uplift conditions. The cranes are Important to Safety and must be demonstrated to be capable of performing their intended functions under all loading conditions.

    RESPONSE – In Safety RAI No. 1, question 4-2, the NRC requested that PFS "Provide the detailed design analyses for the overhead and semi-gantry cranes that demonstrate they meet the criteria specified in ASME NOG-1." In response to this question PFS prepared a crane specification, obtained and evaluated bids from various vendors, and awarded the crane design, fabrication and testing to EDERER, Inc. The Crane analysis, drawings, and reports that were prepared and submitted to the NRC (PFS Letter, Parkyn to U.S. NRC, Response to Request for Additional Information, dated February 10, 1999) with the RAI response are listed below:

    200/25 TON BRIDGE CRANE

    Attachment 1 includes the following design drawings and documents for the Private Fuel Storage Facility 200/25 ton Overhead Bridge Crane:

    - Appendix B Supplement To Generic Topical Licensing Report Edr-1, Rev. 0, Facility Specific Crane Data, 200 Ton Bridge Crane

- Appendix C Supplement To Generic Topical Licensing Report Edr-1, Rev. 1, Summary Of Regulatory Positions, 200 Ton Bridge Crane
- Seismic Qualification Analysis, December 1998 (200 Ton Overhead Bridge Crane)
- Technical Description Of Hoist And Traverse Motion Electrical Controls System (150 & 200 Ton Cranes), Ederer Document Ea-37547, Rev. B
- Technical Description Of Radio Controls Systems (150 & 200 Ton Cranes) Ederer Document Ea-37548, Rev. A
- Ederer Drawing B-36951, Rev. A, Reeving Diagram Sixteen Parts (Main Hoist)
- Ederer Drawing B-36952, Rev. A, Reeving Diagram Eight Parts (Aux Hoist)
- Ederer Drawing B-37061, Rev. A, Main Hoist Block & Hook Dim (200 Ton Crane)
- Ederer Drawing B-37062, Rev. A, Aux Hoist Block & Hook (200 &150 Ton Cranes)
- Ederer Drawing C-36975, Rev. A, Sister Hook 200 Ton (200 & 150 Ton Cranes)
- Ederer Drawing Pa-2189, Rev. C, Clearance Dwg. 200/25 Ton Bridge Crane
- Ederer Drawing D-36976, Rev. A, Bridge Arrangement 200/25 Ton Capacity
- Ederer Drawing B-36977, Rev. A, Trolley Arrangement 200/25 Ton Capacity

<u>150/25 SEMI-GANTRY CRANE</u>

Attachment 2 includes the following design drawings and documents for the Private Fuel Storage Facility 150/25 ton Semi-Gantry Crane:

- Appendix B Supplement To Generic Topical Licensing Report Edr-1, Rev. 1, Facility Specific Crane Data, 150 Ton Semi-Gantry Crane
- Appendix C Supplement To Generic Topical Licensing Report Edr-1, Rev. 1, Summary Of Regulatory Positions, 150 Ton Semi-Gantry Crane
- Seismic Qualification Analysis, December 1998 (150 Ton Semi-Gantry Crane)

- Ederer Drawing B-37063, Rev. A, Main Hoist Block & Hook Dim (150 Ton Crane)

- Ederer Drawing B-36953, Rev. A, Reeving Diagram Sixteen Parts (Main Hoist)

- Ederer Drawing B-36954, Rev. A, Reeving Diagram Eight Parts (Aux Hoist)

- Ederer Drawing Pa-2190, Rev. D, Clearance Dwg. 150/25 Ton Semi-Gantry Crane

- Ederer Drawing D-36978, Rev. A, Bridge Arrangement 150/25 Ton Semi-Gantry

- Ederer Drawing B-36979, Rev. B, Trolley Arrangement 150/25 Ton Capacity

The information provided above clearly demonstrates that the crane design complies with ASME NOG-1, meets the single-failure-proof requirements of NUREG 0554, and is seismically qualified. The design specification for the "Overhead Bridge Crane and Semi-Gantry Crane", specification No. 0599602-M001, Revision 1, dated September 16, 1998, Section 3.5.8.1, General Seismic Requirements, states the following:

> "The Seller shall qualify the canister transfer building cranes and associated equipment to the specified seismic environment utilizing the dynamic analysis method of seismic qualification in accordance with ASME NOG-1 and the requirements of this specification. It is not a design requirement that the crane be operable during an earthquake nor that it be operable after an earthquake, although the latter is desirable. The following is mandatory:
>
> a) The crane bridge (gantry) and trolley are provided with suitable restraints so that they do not leave their rails during an earthquake.
> b) No part of the crane shall become detached and fall during an earthquake.
> c) The crane load shall not lower in an uncontrolled manner during or as the result of an earthquake."

The failure of a crane during canister transfer operations is discussed in SAR Section 8.1.1.3. This section states that with a canister loaded into a transfer cask, a loss of electrical power will delay the transfer operation but will not challenge the integrity of the canister or safe storage of the spent fuel in the canister. There are no safety concerns associated with storage of a canister in its transfer cask until electrical power is restored and the canister transfer operation can resume. The transfer casks are designed to provide adequate shielding and decay heat removal from the canisters. Therefore the canister is in an analyzed condition at all times during the transfer operation. Additionally, the crane design specification requires that the crane design include the ability to manually release the hoist, emergency, bridge, gantry, and trolley brakes to allow for controlled lowering and positioning of the load in the event of an emergency.

The design specification for the "Overhead Bridge Crane and Semi-Gantry Crane", specification No. 0599602-M001, Revision 1, dated September 16, 1998, Section 3.5.3.17, requires that:

> "Bridge and trolley seismic uplift restraints shall be provided if required by the seismic dynamic analysis in Section 3.5.8, Seismic Requirements. The restraint arrangements shall be such that the trolley may be located anywhere along the bridge, and the bridge may be located anywhere along the runway."

As described in SAR Section 4.7.2.5.5, the seismic analysis performed by the crane vendor indicated no uplift from a seismic event on either the bridge crane or the semi-gantry crane, therefore uplift restraints are not required. This will be confirmed as part the "Final Detailed Engineering" phase of the crane design. The cranes are designed with lateral restraints that consist of sidebars mounted next to the crane rails. The sidebars prevent any lateral movement of the bridge wheels and therefore, prevent the wheels from leaving the rails.

14. The crane analysis performed by the crane vendor is based on a deterministic seismic approach. PFS should confirm the analysis has been updated to use the latest PSHA.

 <u>RESPONSE</u> – As discussed in SAR Section 4.7.2.5.3 "Seismic Analysis", the analyses were performed for both cranes by Anatech Corporation to qualify the crane designs for the original PFSF deterministic design earthquake (0.67g horizontal, 0.69g vertical). Although the seismic accelerations in the new design basis are lower, the revised soil properties resulted in increased accelerations at higher elevations in the building. Therefore the cranes were re-evaluated by Ederer for their seismic stability based on the current PFSF design basis ground motion of 0.53g horizontal and 0.53g vertical and resulting response spectra curves. The response spectra curves for this design basis ground motion are shown in Calculation 05996.02-SC-5 and include the effects of properties of the soil underlying the Canister Transfer Building. The Ederer evaluation and resulting minor modifications to both crane designs are also discussed in SAR Section 4.7.2.5.3. Since the modifications are considered minor and the key elements of the analysis discussed in SAR Section 4.7.2.5.3 remain unchanged (i.e., analysis methods, load cases, design allowables, models, properties and mass distribution, and response sprectra), PFS believes that the evaluation presented in conjunction with the design documentation discussed in Item 13 above adequately demonstrate that the cranes will be able to perform their intended function under all loading conditions. PFS intends to formally update the seismic analysis for both

cranes as part the "Final Detailed Engineering" phase of the crane design and fabrication.

15. PFS should demonstrate that critical crane components such as wheel restraints will perform satisfactorily and prevent uplift off the rail and allow crane to be operated following a seismic event). Reference 10CFR72.24 d & i and 10CFR72.122 b, c, d, and f-l.

    RESPONSE – Performance of critical components of both cranes have been demonstrated and provided. Refer to the response to Items 13 and 14 above.

The following open items are associated with Chapter 6, "Thermal Evaluation", of the SER:

Chapter 6 - Open Items

16. PFS should provide the size of diesel storage tanks.

    RESPONSE – The size of the diesel storage tanks is currently shown in Section 3.3.3 of the Environmental Report.

    SAR Chapter 4 will be revised to include this information and a License Amendment submitted by January 24, 2000.

17. PFS should justify why a simultaneous explosion of both propane tanks is not a credible accident scenario.

    RESPONSE – Additional discussion/justification as to why simultaneous explosion of both propane tanks is not a credible accident scenario will be added to SAR Chapter 8.

    This information will be provided in a License Amendment submitted by January 24, 2000.

18. PFS should provide information regarding spacing between the two propane tanks.

    RESPONSE – SAR Chapter 4 will be revised to describe the spacing between the propane tanks as well as any other features required to ensure that simultaneous explosion of both propane tanks is not a credible accident scenario.

    This information will be provided in a License Amendment submitted by January 24, 2000.

19. PFS should provide an evaluation of the effects of a fire on the doors on CTB. Credit is taken for the cask transporter to be outside the transfer cell during canister transfer operations, however no discussion is provided on the fire rating of the cell doors and walls. PFS needs to identify which doors and walls in the CTB are fire rated and discuss the fire rating of each.

    RESPONSE –The SAR will be revised to identify all fire rated doors and walls and discuss the fire rating of each.

    This information will be included in a License Amendment submitted by January 24, 2000.

20. Lightning often causes fires. PFS needs to discuss the effects of lightning strikes on the CTB and associated Structures, Systems, and Components and provide a description of the lightning protection system in the vicinity of CTB.

    RESPONSE –The SAR will be revised to discuss the CTB lightning protection system and the effect of lightning strikes on the CTB and associated Structures, Systems, and Components. The discussion will include the type of system to be utilized and the codes and standards that the system will comply with.

    This information will be included in a License Amendment submitted by January 24, 2000.

21. PFS should identify the type of sprinkler system to be installed in CTB and evaluate its effectiveness in controlling and extinguishing fires.

    RESPONSE –The SAR will be revised to include additional discussion on the type of sprinkler system (including applicable codes and standards for design and construction) to be installed in the CTB and its effectiveness in controlling and extinguishing fires.

    This information will be included in a License Amendment submitted by January 24, 2000.

22. PFS should provide an evaluation of the potential consequences of a main line locomotive fire (6,000 gal of fuel) on the spent fuel storage casks located on the cask storage pads.

    RESPONSE – PFS will evaluate the requested locomotive fire and determine the consequences as well as the impacts, if any, on the facility design.

This information will be included in a License Amendment submitted in March 2000.

23. PFS should show that the design of the fire detection and suppression system is consistent with current industry standards.

RESPONSE – The SAR will be updated to provide additional discussion on the types of fire detection and suppression systems to be utilized in the CTB. Specific sections of applicable codes will be referenced and the Reference section will be revised to indicate that the latest code editions in affect at the time of design will be used.

This information will be included in a License Amendment submitted by January 24, 2000.

The following open items are those that are identified in the SER submitted with Reference 2.

24. (SER open item 1-1) The dry cask storage systems proposed for the Facility are currently under NRC review for use under the general license provisions of 10 CFR Part 72, Subpart K. Review of the cask systems for site-specific use at the Facility will be conducted when one of the cask systems is approved for use under the general license. Before a license for the Facility is issued, the applicant should demonstrate that the cask system is acceptable for use at the Facility under the site-specific license provisions of 10 CFR Part 72. Further, the applicant should ensure that cask information in the Facility SAR is consistent with the Final Safety Analysis Report for the specific dry cask storage system. The final SER will include consideration of the cask system.

RESPONSE – The PFS SAR will be reconciled with the specific dry cask storage system(s) Final Safety Analysis Report when it is issued. Any revisions required to the PFSF SAR will be made at that time and a License Amendment submitted to the NRC.

25. (SER open item 1-2) The SAR should be updated to incorporate all information that was used as the basis for demonstrating compliance with 10 CFR Part 72 including information and commitments provided in the applicant's responses to the NRC's requests for additional information (RAIs).

RESPONSE – With the issuance of License Amendments 4 through 8, PFS believes that all information provided to the NRC in Commitment letters and RAI responses that was used as the basis for demonstrating compliance with 10 CFR Part 72, has been incorporated into the PFS License Application. Commitments provided subsequent to License Amendment 8 will be incorporated in future amendments as required.

26. (SER open item 2-1) As discussed in Section 2.1.2 of the SER, the staff has determined that additional information is needed to assess the potential hazards from military aircraft flying in the vicinity of the Facility.

    RESPONSE – With the issuance of License Amendments 4 through 8, PFS believes that all information requested by the NRC has been incorporated in the License Application. Additional information requests or open items identified subsequent to License Amendment 8 will be incorporated in future amendments as required.

27. (SER open item 2-2) As discussed in Section 2.1.3.2 of the SER, the staff has determined that additional information regarding the site meteorological data is needed to assure appropriate use of the information in future cask-specific analyses.

    RESPONSE – Any open items regarding the site meteorological data resulting from completion of the NRC review of the cask systems for site-specific use will be resolved after receipt. Any necessary SAR revisions will be made at that time and a License Amendment submitted to the NRC.

28. (SER open item 2-3) As discussed in Section 2.1.6.2 of the SER, the staff has determined that additional information is needed to assess the affects of ground vibrations on the Facility. The applicant has requested an exemption to 10 CFR 72.102(f)(1) and proposes to use a PSHA approach with a 1,000-year return period, instead of the DSHA approach. The staff agrees with the PSHA approach, but it should use a 2,000-year return period instead of the applicant-proposed 1,000-year return period.

    RESPONSE – Amendment 5 of the PFSF SAR incorporated the PSHA approach using a 2,000-year return period.

29. (SER open item 2-4) As discussed in Section 2.1.6.4 of the SER, the staff has determined that additional information regarding soil classification (e.g., detailed soil profiles) is needed to assess stability of subsurface materials.

    RESPONSE – With the issuance of License Amendments 4 through 8, PFS believes that all information requested by the NRC regarding soil classification (e.g., detailed soil profiles) needed to assess stability of subsurface materials has been incorporated in the License Application.

    The questions asked in the recent January 5, 2000 phone call will be addressed in a License Amendment submitted by January 24, 2000.

30. (SER open item 2-5) As discussed in Section 2.1.6.4 of the SER, the staff has determined additional information regarding stability of the cask storage pad and Canister Transfer Building is needed to assess stability of subsurface materials. Additional information that is required includes analyses that use cask-specific sliding resistance values and address overturning and sliding of the storage pad and Canister Transfer Building under a design basis earthquake.

    RESPONSE – With the issuance of License Amendments 4 through 8, PFS believes that all information requested by the NRC regarding stability of the cask storage pads and Canister Transfer Building has been incorporated in the License Application.

    The questions asked in the recent January 5, 2000 phone call will be addressed in a License Amendment submitted by January 24, 2000.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

JH Cooper for

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

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January 18, 2000

COMMITMENT RESOLUTION LETTER #25
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

A meeting was held on January 10, 2000 between Private Fuel Storage (PFS) and the NRC/CNWRA to discuss several questions regarding the aircraft crash hazard assessment for the Private Fuel Storage Facility (PFSF). The following issues were discussed in the conference.

AIRCRAFT CRASH HAZARD ASSESSMENT

Presentation of the Analysis in One Document

NRC Comment:

In the meeting between PFS and the NRC/CNWRA in Salt Lake City on November 16, 1999, the issue of PFS's multiple submissions of additional information concerning aircraft crash hazards was discussed. At that meeting, PFS committed to combining its submissions into a single document that would fully would set forth PFS's position and analysis on aircraft crash hazard issues. Subsequent to that meeting, PFS combined all its information and analyses in one Report, dated November 24, 1999. The Report, however, did not contain cross-references to earlier PFS submissions.

PFS Response:

As noted in the transmission letter for the Report, PFS took a fresh look at all of the available data and submissions to present one comprehensive position and results. This single report is referenced in the PFS SAR and constitutes PFS's sole position on aircraft crash hazards. PFS reiterated this point at the January 10 meeting. Thus, cross-references to the earlier PFS submissions are not necessary nor relevant. PFS will update its November 24 Report and SAR as necessary to resolve the issues raised in the meeting set out below, so that its analysis will remain in a single comprehensive report.

F-16s Transiting Skull Valley

NRC Comment:

In Tab H to its November 24, 1999 Report, PFS determined that 95 percent of the F-16 crashes that would occur in Skull Valley would occur due to engine failure and would allow a pilot time to attempt to direct a crashing plane away from the PFSF. PFS should provide additional support for its 95 percent determination. This support may take the form of further confirmatory analysis of the existing data (possible approaches include normalization or testing for trends), the gathering of additional data from the Air Force, and/or statements from Air Force officials

PFS Response:

PFS will undertake additional analyses and/or obtain additional supportive data and information from the Air Force through the FOIA process.

NRC Comment:

PFS also determined in its November 24 Report that for no more than 5 percent of the F-16s that would crash in Skull Valley due to engine failure, which would allow a pilot time to attempt to direct a crashing plane away from the PFSF, would the pilot fail to do so. PFS should provide additional information to support this determination. Topics discussed at the meeting of the type of additional information that might be provided by PFS included: a description of a pilot's situational awareness of his location and surroundings, the fact that pilots do as a matter of general course what they are trained to do, the fact that F-16s transit Skull Valley under visual flight rules with a clear air mass, the applicability of PFS's analysis to night missions (by virtue of the same emergency procedures being employed by F-16 pilots at night and the lighting of the PFSF), and the F-16 performance (zoom) charts referenced in the PFS November 24 Report.

PFS Response:

PFS will provide additional information to support its determination.

The Moser Recovery Route

NRC Comment:

In its impact probability calculation for the F-16s returning to Hill AFB via the Moser Recovery Route, PFS similarly determined that for no more than 5 percent of the F-16s crashing due to engine failure (in which the pilot would remain in control of the plane) would the pilot fail to direct the plane away from the site. PFS should provide additional information to justify the application of this factor to the probability calculation for the Moser Recovery Route. Topics discussed at meeting of the type of additional information that might be provided by PFS included: the additional response time provided by the fact that F-16s on the Moser Recovery Route are flying at an altitude of 15,000 ft MSL; the likelihood that the bad weather necessitating use of the Moser Recovery Route is local to Hill AFB; and a pilot's awareness of his location and surroundings, supplemented by information and direction provided by air traffic control.

PFS Response:

PFS will provide additional information to support the application of this factor to the Moser Recovery calculation.

Effective Area for Impact Probability Calculation

NRC Comment:

When PFS calculated the effective area for the PFSF in order to determine the probability of an aircraft impact at the facility, it calculated separately the effective area for the cask storage area and the effective area for the canister transfer building and then added them together. It was unclear that this approach is appropriate, in that the cask storage area and canister transfer building are not separated by a great distance.

PFS Response:

To clarify its approach, PFS explained at the January 10 meeting that this formulation was reasonable, conservative, and consistent with both DOE Standard 3014 and standard probability text (e.g., *Introduction to Probability*, John Freund, Dover Publications Inc. (1993)), in that 1) the probability of a crashing aircraft hitting either area is, as a theoretical matter, equal to the probability of hitting one area, plus the probability of hitting the second area, minus the probability of hitting both areas and 2) PFS's analysis treats any impact with the canister transfer building or the cask storage area as an impact, regardless of the severity of the effects of the impact. From the meeting, PFS understands that no further information or clarification is required to justify its methodology of adding the effective areas of the cask storage area and the canister transfer building.

Military Airway IR-420

NRC Comment:

With respect to PFS's probability calculation for IR-420, PFS used destroyed aircraft as the basis for calculating the crash rate for large military cargo planes. PFS should more fully justify its use of destroyed aircraft in this calculation.

PFS Response:

PFS will explain further why it used destroyed aircraft as the basis for the calculation and why such usage is proper.

NRC Comment:

PFS should explain or justify its application of the factor of 3/21 on page 53 of its Report.

PFS Response:

PFS will explain the use of the 3/21 factor and will ensure that accidents are appropriately accounted for through additional supportive data and information to be obtained from the Air Force through the FOIA process.

NRC Comment:

In footnote 60 on page 53 of the Report, PFS refers to a multi-engined aircraft "'losing' an engine." PFS should clarify what this phrase means.

PFS Response:

PFS will clarify the footnote to include that "losing an engine" means failure of the engine and not its falling off of the aircraft.

NRC Comment:

In its Report, PFS uses the NUREG-0800 commercial aircraft crash rate per mile of 4×10^{-10} for large cargo aircraft instead of the crash rate that it calculates in its probability calculation for IR-420.

PFS Response:

PFS will use the calculated rate instead of the NUREG-0800 rate.

The Utah Test and Training Range (UTTR)

NRC Comment:

In its November 24 analysis, PFS concluded that aircraft conducting air combat training on the UTTR more than 10 miles from the PFSF would not pose a hazard to the facility. PFS should further justify its use of the 10 mile limit by including additional discussion. Topics discussed at meeting of the type of additional information that might be provided by PFS included the fact that high stress maneuvers inside restricted areas on the range are conducted toward the center of the restricted area and the fact that an aircraft that experienced an incident leading to a crash while at high altitude would not glide a long distance off range and strike the PFSF.

PFS Response:

PFS will provide additional discussion to support its use of the 10 mile limit.

NRC Comment:

In the section of its Report discussing air-to-ground combat training, PFS states that aircraft conducting such training over 20 miles from the PFSF would not pose a hazard to the facility. PFS should clarify why that conclusion does not contradict its conclusion discussed above that aircraft conducting air-to-air combat training over 10 miles away do not pose a hazard to the facility.

PFS Response:

PFS's referred to aircraft conducting air-to-ground combat training 20 miles from the PFSF because that is the location closest to the PFSF at which such training occurs. PFS will clarify that it is not applying a different standard to air-to-ground than to air-to-air combat training.

NRC Comment:

When calculating the probability that an aircraft experiencing an incident on the UTTR that would lead to a crash would hit the PFSF, PFS used a factor Ap to account for the potential area in which the aircraft could hit the ground. PFS described the area as a circle with a 10-mile radius. It is inappropriate to consider a 10-mile circle around the PFSF when aircraft on the UTTR are only located to the west of the facility. PFS should clarify how this area is applied in its analysis.

PFS Response:

PFS will clarify that, as discussed in the meeting, the center of the circle representing the potential area in which a crashing aircraft could hit the ground is not located at the PFSF but rather at the point at which the incident leading to the crash occurs.

NRC Comment:

In its probability analysis for the UTTR, PFS only divides elevations below 10,000 ft. AGL into altitude bands and does not do so for elevations above 10,000 ft. AGL.

PFS Response:

PFS will further clarify why it took that approach.

General Aviation

NRC Comment:

With respect to PFS's probability calculation for General Aviation, PFS states at page 71 of its November 24 Report, that 73.1 percent of all General Aviation aircraft in the United States were single-engine piston types and approximately 75 percent of those weigh less than 3,500 lbs. PFS should provide the basis for these two percentages. PFS should also review an apparent conflict in its data regarding the number of General Aviation aircraft in the United States given at page 69 of its November 24 Report.

PFS Response:

PFS will provide the bases for its data and will resolve any conflicts between the data and its Report.

NRC Comment:

When calculating the probability that a general aviation aircraft would impact the PFSF and cause a radioactive release, PFS states that crashes of aircraft weighing less than 3,500 lbs. would not penetrate a spent fuel storage cask, in that the aircraft crash would be bounded by the cask design basis tornado missile. In footnote 74 on p. 71 of its November 24 Report, PFS states that this conclusion applies to the canister transfer building despite the fact that its design basis tornado missile impacts at a lower velocity than the design basis tornado missile for the casks. PFS justifies this conclusion on the grounds that any spent fuel canister in the canister transfer building would be inside a storage or transportation cask all but 8 percent of the time, when it would be inside a transfer cask. PFS concludes that the potential for an aircraft impact to occur while the canister is inside the transfer cask is "negligible" in view of the relatively small area of the facility constituted by the canister transfer building and the overly conservative, bounding nature of the general aviation probability calculation. PFS should further support this conclusion with additional information. Topics discussed at meeting of the type of additional information that might be provided by PFS included the nature of the transfer operation and the robust nature of the canister transfer building and overhead bridge crane.

PFS Response:

PFS will provide further information to support its conclusion.

Federal Airways

NRC Comment:

With respect to probability calculations for commercial airways J-56 and V-257, PFS did not state whether its calculations included private business jet traffic.

PFS Response:

PFS will clarify that its airway calculations includes private business jet traffic.

Ordnance

NRC Comment:

In its November 24 Report, PFS determined that for 95% of the F-16s transiting Skull Valley the pilots would remain in control of the aircraft in the event of a crash and have time to jettison unarmed ordnance or dummy ordnance that they may be carrying and attempt to take measures to avoid the PFSF. PFS calculated the probability that either the aircraft or the jettisoned live (but unarmed) or dummy ordnance would strike the PFSF. For the remaining 5% of the F-16s transiting Skull Valley, PFS assumed that ordnance was not jettisoned and made a single calculation of the probability of the aircraft impacting the PFSF. PFS should take additional steps to assess whether live, but unarmed, non-jettisoned ordnance carried on such crashing aircraft might adversely impact the PFSF in cases in which the aircraft itself did not directly impact the facility. Methods discussed at the meeting concerning how PFS might assess this possible potential included further verification that live but unarmed ordnance will not explode on impact and/or ensuring that in the unlikely event on an explosion, the effective impact area for this event is bounded by the effective area calculation for the PFSF.

PFS Response:

PFS will resolve this issue by further verifying the unlikely potential for unarmed ordnance to explode on impact through additional supportive data and information to be obtained from the Air Force through the FOIA process and/or ensuring that the effect of any such explosion is bounded by the effective area calculation for the PFSF.

Schedule of Responses

PFS will provide its response to the above NRC comments (except those for which PFS has had to file Freedom of Information Act requests with the U.S. Air Force in order to obtain some of the data necessary for PFS to respond to the NRC's comments) by Monday, January 24, 2000. PFS will provide its responses to those NRC comments for which FOIAs have been filed promptly upon receiving and analyzing the requested data.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

John L. Donnell

John L. Donnell
Project Director
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Enclosure

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November 12, 1999

EIS COMMITMENT RESOLUTION LETTER #1
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

References: 1. U.S.NRC letter, Flanders to Parkyn, Request For Additional Information For The Environmental Impact Statement (TAC No. L22462), dated August 19, 1999
2. PFS letter, Parkyn to U.S. NRC, Responses To Second Round EIS Request For Additional Information. dated October 19, 1999

During our November 5, 1999 conference call, the NRC/ORNL requested clarification and additional information regarding several RAI responses provided by Private Fuel Storage (PFS) in Reference 2. The NRC/ORNL questions and comments are documented below along with the PFS response.

NRC Questions and Comments

EIS RAI No. 2, Question 2-5
The NRC asked for further clarification on the sources of private water referred to in this response, amount of water available, and potential impacts from trucking this water.

RESPONSE: PFS will provide additional information explaining the conclusion that the required quantity of water is available from private water sources. PFS will also verify that previously provided information on air pollutant emissions, truck traffic, and noise considerations include the impacts from trucking these quantities of water. PFS will provide the requested information by November 19, 1999 in a separate letter.

EIS RAI No. 2, Question 2-7
The NRC asked for additional clarification on how the 60-ft width permanently affected by the Low Corridor heavy haul road alternative was determined. NRC also requested

that PFS provide a comparison, between the rail line and the heavy haul road, of the potential for initiating wildfires.

RESPONSE: The width of land permanently affected by the alternative Low Corridor heavy haul road was determined to be approximately 60-ft. The asphalt portion of the road is about 34-ft wide, consisting of two 12-ft wide lanes and a 5-ft wide paved shoulder on each side of the road. The alternative road would require numerous 4-ft diameter culverts that would pass under the road so that it does not interfere with natural drainage. Allowing for 2-ft of cover over the top of the culverts, much of the road would be approximately 6-ft above grade. A 2-to-1 (horizontal to vertical) side slope of the embankment along the sides of the road was used, resulting in 12-ft horizontal on each side of the road. This results in a total average permanently affected width of 34 + 24 = 58-ft, which was rounded off to 60-ft. This is consistent with the methodology used to assess the average width of the Low Corridor rail line, in which a 40-ft width was determined to be permanently affected.

PFS has performed a search on the National Transportation Safety Board web site for information regarding the probabilities of wildfires associated with rail and heavy haul transport. While the search revealed numerous documents that discuss fires, these fires were determined to be associated with severe accidents such as collisions rather than grass or brush fires initiated by a passing vehicle. Since insufficient data was found on which to base a quantitative assessment of the differences in probabilities of wildfires initiated by rail versus heavy haul transportation, the following is a qualitative assessment.

PFS will own (lease) and maintain the rail equipment used for delivery of Spent Nuclear Fuel to the storage facility. This equipment will utilize the latest design innovations (train monitoring, breaking systems. etc.) to reduce the risk of wildfires due to rail transport. It is inherent in the design of rail equipment that sparks can be produced by the steel wheels of railroad trains in contact with the steel rails. Unlike cars and trucks, the axles on a train do not have differentials that permit the two wheels on one axle to rotate at different rates around curves. When a train moves around a curve, one of the wheels on the same axle slides along the rail to some extent, and this has a tendency to generate sparks. Sparks can also be generated when the locomotive wheels slip while pulling a train uphill. There will be very few curves (no sharp curves) and no steep grades along the Low Corridor rail line. Nevertheless, the possibility exists of sparks being produced by rail transport. In the case of a heavy haul truck, sparks are unlikely to be produced since only the rubber tires contact the asphalt surface under normal driving conditions. Sparks can be produced by vehicle transport if a metal part of the undercarriage, such as exhaust piping, becomes partially disconnected and drags along the asphalt. However, with routine preventive maintenance on the heavy haul vehicles, such an occurrence would be unlikely. If a driver were to toss a lighted cigarette out the window of the vehicle, it is possible that a wildfire could start. This could occur whether the vehicle is a heavy haul truck or train. with similar likelihood of starting a fire. Since trains can produce sparks during normal operating conditions due to the metal wheels contacting the metal rails, a condition that does not exist with the heavy haul option, it is considered that rail transport would have a slightly higher probability of creating sparks that could lead to wildfires than heavy haul transport. However, as noted above. the Low Corridor rail line with its minimum number of curves, no steep grades. and use of the latest equipment design innovations will minimize the risk of sparks which could lead to wildfires.

EIS RAI No. 2, Question 4-8
NRC requested clarification on some of the air emission data provided. What are the units for stack height, diameter, and for temperature in the spreadsheet provided by Utah DEQ?

RESPONSE: PFS contacted Deborah McMurtrie, Environmental Scientist with the State of Utah DEQ, who had provided the data on the stacks that was included in the response to this question. Ms. McMurtrie indicated that the stack heights and diameters are in units of feet, and temperatures are in units of degrees Fahrenheit.

EIS RAI No. 2, Question 4-14
The NRC requested clarification on the dose rates used in this response. They appear to differ from the dose rates presented in SAR Table 4.2-2 and 4.2-5.

RESPONSE: The dose rates specified in PFSF SAR Tables 4.2-2 (HI-STORM storage cask) and 4.2-5 (TranStor storage cask) were from previous revisions of the vendor Topical Safety Analysis Reports (TSARs), and are based on the assumption that the storage cask contains design basis fuel. These tables (and corresponding tables in PFSF SAR Chapter 7) will be revised in an upcoming amendment to the PFSF License Application to agree with the latest vendor TSAR dose rate information for the vendors' design basis fuel.

The dose rates on contact with storage casks presented in the response to EIS RAI No. 2, Question 4-14 were calculated in Stone & Webster Calculation No. 05996.02-UR (D)-008, Revision 0, Dose Rate Calculations at PFSF Locations Potentially Accessible to Wildlife and Estimates of Annual Doses to Individual Animals. The calculation uses the most recently updated contact dose rates in the vendors' TSARs. However, the calculation does not assume that the storage casks contain the vendor's design basis fuel, since this fuel is too "hot" to be transported to the PFSF in the respective vendor shipping casks. It was assumed in the calculation that the storage casks contain either spent PWR fuel having 40 GWd/MTU burnup and 10 years cooling time (as discussed in PFSF SAR Section 7.3.3.5), or spent BWR fuel having 35 GWd/MTU burnup and 10 years cooling time. Gamma and neutron dose rates were scaled accordingly. As stated in the response to EIS RAI No. 2, Question 4-14, dose rates presented in this response are total dose rates which include not only the dose rates from contact with a storage cask, but also include the contribution from nearby casks. The calculation assumed that nearby casks are the same model (HI-STORM or TranStor) as the contact cask, and are loaded with average or typical PFSF spent fuel, represented by PWR fuel having 35,000 MWd/MTU burnup and 20 years cooling time (as discussed in PFSF SAR Section 7.4).

The details of this methodology are explained in S&W Calculation No. 05996.02-UR(D)-008, Rev. 0, which is attached to this response.

EIS RAI No. 2, Question 4-15
The NRC asked if PFS had received the requested information on protected species from the Wyoming Natural Diversity Database or the U.S. Fish & Wildlife Service?

RESPONSE: PFS has not received the requested information to date. PFS continues to pursue this information and will provide it to the NRC as soon as it is received.

EIS RAI No. 2, Question 4-29
PFS should provide a brief description of the equipment used to take the photographs and explain the technique used to size the facilities. The NRC requested clarification on Figure 9. They need an explanation as to where this photograph was taken from (description and distance from the site) and why this vantage point was chosen. PFS should provide an additional artist's sketch using a photograph taken with a non-telephoto lens similar to the other photographs provided. Figure 9 appears to show light poles along the access road however the access road does not appear to be lighted in the corresponding night time photograph (Figure 10).

RESPONSE: All background photos except Figure 15 (16 night) were taken with an Olympus Stylus Zoom 115 DLX, 38-115 mm, compact camera using the 38-mm lens setting (not magnified) with the exception of Figure 9. Figure 9 was taken in a previous year (10/7/98) as part of a unrelated field survey in the Stansbury Mountains. The Skull Valley Band of Goshute granted permission for a PFS survey team to cross the eastern side of the reservation into the Stansbury Mountains. The photo used for Figure 9 was extracted from a multi-picture view of the Skull Valley taken for reference during this survey. The shot was taken from the mouth of Antelope Canyon located at the approximate center of the SW ¼ of Section 30, T4S R7W on the western flank of Deseret Peak. Because this photo was taken for reference purposes, the shot used the full zoom feature of the Olympus camera (115-mm setting) to bring up the detail at the location of the PFS siting area. For this reason the two views of the site area (Figure 9 and Figure 11) give a different perspective of distance. However, the artist did compensate for this effect in his work on Figure 9.

The photo background used for Figure 15 (16) was taken with a Minolta SLR Zoom, 35-80 mm camera owned by the artist using the 35-mm setting (not magnified). This figure(s) was originally created for the first RAI submittal of Artist Sketches (February

18, 1999, Question 14-1) to provide the closest and clearest viewing perspective of the site area from Skull Valley road. For this reason, the photo background was reused for this submittal. It should be noted that the viewpoint for Figure 13 (PFS Facility from the Pony Express Store) is also physically located adjacent to the Skull Valley Road but at a ½ mile greater distance to the site area and subject to a more obscured viewing perspective.

To create the artist sketches, an initial photo survey was conducted based on the locations identified in the NRC question (4-29) by PFS with a skilled graphic artist. The proposed view using the initial backgrounds was then discussed in a meeting with the NRC in Salt Lake City on September 1, 1999 to verify their acceptability. A second photographic survey was then conducted to verify the appropriateness of the final background views and to also finalize in the field with the artist what would be seen from the vantage point contained within each view. The in-field verification process following the two photo survey efforts directly supported the subsequent artistry used in adding overlays to the background photos to complete the final sketches.

The artistry itself is based on the engineering drawings of the storage area, Intermodal Transfer Point, Low rail siding area, and rail equipment; topographic maps of the respective areas to identify physical reference points and features; and the aforementioned field surveys to visually review the area and generate additional reference points. In addition, the artist for this submission was the same person who took the aerial photos using the Minolta SLR camera and generated the first submission of artist sketches submitted to the NRC. The unique visual perspective from the air allowed the artist to identify key ground reference points that were used in locating and scaling the various scenic additions to the photo backgrounds. These same reference points were used in the generation of this second submission.

To supplement the use of physical mapped or observed field references to locate and size PFS features at the storage area, a photo scale created from the first series of artist sketches was reused. This scale was created by photographing buildings of a known size and spacing in downtown Salt Lake City at a known distance. The same Minolta SLR camera and settings used in the aerial photography were used for the reference photo. A graphic scale was then created to provide relative dimensions from known reference points originating from topographic maps or observable on the background photos (e.g.- seismic test lines across the site north/south, east/west). This provided a means of scaling the PFS features to fit a known area in a specific view. Ultimately all of the above reference information coupled with the skill of the artist contributed to the completion of the overlays which were then added to the photo backgrounds.

Figure 9 and 10, the day and night view of the PFS site from Deseret Peak includes all of the NRC desired requested features (buildings, berms, batch plant, access road, rail road, and light poles) and the 35 foot high power poles along the access road. There are no plans to illuminate the access road from Skull Valley road to the PFS siting area. Therefore, in the night view of the site area (Figure 10), the access road will not be visible.

New artists sketches of the facility from Deseret Peak have been provided and are attached as Figure 17 and 18. These photographs were taken with the same camera as the other photographs (Olympus Stylus Zoom 115 DLX, 38-115 mm, compact camera using the 38-mm lens setting, not magnified). Descriptions of the Figures are provided below:

> FIGURE 17: PFS Facility from West of Deseret Peak – Looking west from the highest accessible point near Deseret Peak (at the entrance to Antelope Canyon located at the approximate center of the SW ¼ of Section 30, T4S R7W) in the Stansbury Mountains 6 miles east of the PFS site, one can see the general layout of the facility and buildings. At this distance, little detail of the facility can be discerned.
>
> FIGURE 18: PFS Facility from West of Deseret Peak at Night – From the same vantage-point as Figure 17, this is how the PFS Facility will appear with nighttime illumination.

As requested we have included a CDROM of Figures 1 though 18 in .jpg format.

<u>EIS RAI No. 2, Question 5-1</u>
The NRC requested clarification on several items in the cost benefit analysis. They requested a year by year breakdown in constant dollars of the cost components used by PFS (hard copy and electronic version). The basis for assumptions used in the analysis needs to be provided along with documented sources for data used. PFS needs to explain the differences between this analysis and the analysis presented in the business plan.

RESPONSE: PFS will provide the requested information by November 19, 1999 in a separate letter.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

John L. Donnell

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Private Fuel Storage L.L.C.

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November 19, 1999

EIS COMMITMENT RESOLUTION LETTER #2
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

References:
1. U.S.NRC letter, Flanders to Parkyn, Request For Additional Information For The Environmental Impact Statement (TAC No. L22462), dated August 19, 1999
2. PFS letter, Parkyn to U.S. NRC, Responses To Second Round EIS Request For Additional Information, dated October 19, 1999
3. PFS letter, Donnell to U.S. NRC, EIS Commitment Resolution Letter #1 dated November 12, 1999

During our November 5, 1999 conference call, the NRC/ORNL requested clarification and additional information regarding several RAI responses provided by Private Fuel Storage (PFS) in Reference 2. Responses were provided to most of these requests in Reference 3, which indicated that an additional letter would be forthcoming to provide information associated with remaining requests, specifically EIS RAI No. 2, Questions 2-5 and 5-1. The purpose of this letter is to provide the remaining information requested by the NRC/ORNL. Some of the information requested is proprietary and is being provided under separate cover. The NRC/ORNL questions and comments are documented below along with the PFS response.

NRC Questions and Comments

EIS RAI No. 2, Question 2-5

The NRC asked for further clarification on the sources of private water referred to in this response, amount of water available, and potential impacts from transporting this water.

RESPONSE: PFS provided one reputable contractor in Tooele County pertinent information on water needs contained in EIS RAI No. 2, Question 4-4, for construction of the PFSF, the ITP, and the Low Corridor, and asked if existing water sources in Northern

Skull Valley could supply these needs. This contractor, who has an extensive work history on large construction projects in the Utah West Desert similar to the PFS project, submitted a reply to PFS's request. The contractor states in the letter that sufficient quantity and quality of water is available in the North end of the Stansbury Mountain range to supply the needs for construction of the PFSF, and the Low Corridor rail line or the Intermodal Transfer Point (ITP), in the time period identified. The letter response itself is proprietary, and is being submitted under separate cover. While this contractor is not the only one in the area with relevant work experience, the specific statements illustrate that, based on historical experience, sufficient water exists in the area to meet projected demands.

Information currently in the PFSF Environmental Report (ER) regarding truck traffic associated with PFSF construction does not include the impacts from trucking the quantities of water identified in EIS RAI No. 2, Question 4-4. In addition to water trucks, the number of truck trips for PFSF construction phases 1, 2 and 3 is estimated to increase by 14.7%, based on the latest projected earth work and imported material needs. The duration of construction phases 2 and 3 was previously reported as 10 years each. The correct duration for these phases is 5 years each. This changes the frequency of truck trips associated with PFSF construction during these phases. The projected traffic along the Skull Valley Road during PFSF construction, shown in Table 4.1-3 of the ER, needs to be revised to account for the increased number of truck trips due to water trucks, the latest earth work and imported material projections, and the shortened intervals (5 versus 10 years) of PFSF construction phases 2 and 3.

The response to EIS RAI No. 2, Question 4-20, included a table that provided a breakdown of traffic on the Skull Valley Road projected during construction and operation of the PFSF. This table was based on the existing ER Table 4.1-3, which did not account for the projected increase in truck trips discussed above. In Attachment 1, the table has been updated to account for water trucks, the 14.7% increase in truck trips for PFSF construction phases 1, 2 and 3 to accommodate the latest projected earth work and imported material needs, and for the 5 year duration of construction phases 2 and 3. The number of water truck trips incorporated into the Attachment 1 table is based on the assumption that each water truck has a capacity of 7,500 gallons. Traffic projections in ER Table 4.1-3 will be updated based on Attachment 1. ER Table 4.1-3 also identifies the sound level associated with the peak traffic volumes during the various construction phases of the PFSF, and during operation. These peak sound levels will also be revised to account for the increased Skull Valley Road traffic reflected in Attachment 1. Preliminary assessment indicates that the sound levels from traffic on the Skull Valley Road will increase only slightly (less than 2 decibels) over those currently identified in ER Table 4.1-3.

Information in the PFSF Environmental Report (ER) regarding air pollutant emissions for construction operations does not include the impacts from trucking the quantities of water identified in EIS RAI No. 2, Question 4-4, nor for the number of trucks needed for the latest earth work and imported material projections. PFS is in the process of revising applicable air pollutant emission calculations to include the effects of the revised truck traffic reflected in Attachment 1. The results of these calculations will be used to revise ER Table 4.1-4 ("Estimated Construction Related Pollutant Emissions" for construction of the PFSF), Table 4.3-1 ("Estimated Construction Related Pollutant Emissions for Intermodal Transfer Building"), and Table 4.3-2 ("Estimated Construction Related Pollutant Emissions for Low Corridor Rail Line"). ER Table 4.1-4 assesses peak emissions associated with PFSF construction which occurs during construction phase 1, and is not affected by the change in construction phase 2 and 3 durations to 5 years. Based on preliminary results of the air pollutant emission calculations, these tables have been revised and are included in Attachment 2.

The sections of the ER that discuss construction traffic, noise, and air pollutant emissions will be updated in an upcoming amendment of the License Application, planned for submittal to the NRC on December 10, 1999.

<u>EIS RAI No. 2, Question 5-1</u>

The NRC requested PFS to provide a year by year breakdown in constant dollars of the costs and benefits of building the PFSF versus the no-action alternative in order to allow an independent evaluation of the discounted cash flow. Explain why PFS chose to use $8 million in annual post-shutdown costs at shutdown reactors when the PFS response to the first RAI indicated these costs would range from $3 million to $8 million per year. Explain the difference in assumptions for the 38,000 MTU cases selected versus the others. Explain how PFS selected the non-PFS utility reactors used in the analysis. Provide both proprietary (if necessary) and non-proprietary versions of the requested information, as well as a basis for protecting the proprietary information.

RESPONSE: Annual constant dollar benefits and costs are provided for each of the 6 cases analyzed (Cases 1, 5, 7, 9, 11, and 13 from PFS' response to EIS RAI No. 2, Question 5-1). The annualized avoided costs (benefits) for a 2002 PFSF versus "repository only" cases are provided in Energy Resources International, Inc. (ERI) Report ERI-2025-9901 (enclosed). The annualized PFS operating costs are provided as separate Excel spreadsheets for each of the 6 cases analyzed, included as Attachment 3. The information in these spreadsheets was extracted from PFS Financial Plan files. The Excel spreadsheets are also provided in electronic format in the attached diskette (Attachment 4). The electronic file of the enclosed ERI Report is proprietary to ERI and is being submitted under separate cover, accompanied by an affidavit that sets forth the basis for protecting this information.

The basis for shipment rates to the DOE repository is provided in section 2.4.2 of the enclosed ERI report.

The "expected" cases assume approximately 16,000 MTU of spent fuel stored at PFS for the 2010 repository opening, and 20,000 MTU for the 2015 repository opening. These cases were determined by selecting reactors which were PFS member reactors, shutdown reactors, reactors with limited on-site storage capacity, and reactors that began operating in the mid-1980s or earlier. These reactors were assumed to have the greatest need for interim storage capacity either now or during the PFSF term of operation, and would likely consider storage at the PFSF. Further discussion of the assumptions for these and other cases are covered in sections 2.5 and 3.2 of the enclosed ERI report.

The explanation of the selection of $8 million per year as the post-shutdown spent fuel pool operating cost is contained in section 2.3.2 of the enclosed ERI report. Recent cost data indicate that post-shutdown spent fuel pool operating costs now range from $8 to $15 million per year, so the selection of $8 million per year is conservative.

The explanation for the assumptions used in the 38,000 MTU cases is provided in section 2.5 of the enclosed ERI report and the results are discussed in section 3.2 of this report. The explanation of how non-PFS reactors were selected is also discussed in these sections.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

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Enclosure

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December 7, 1999

EIS COMMITMENT RESOLUTION LETTER #3
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

References: 1. U.S.NRC letter, Flanders to Parkyn, Request For Additional Information For The Environmental Impact Statement (TAC No. L22462), dated August 19, 1999

2. PFS letter, Parkyn to U.S. NRC, Responses To Second Round EIS Request For Additional Information, dated October 19, 1999

3. PFS letter, Parkyn to U.S. NRC, Proprietary Response To Second Round EIS Request For Additional Information, dated October 19, 1999

During the November 30, 1999 phone call, between the NRC and Stone and Webster (S&W), the NRC requested clarification/additional information regarding two RAI responses provided by Private Fuel Storage (PFS) in Reference 2. The NRC requests/questions are documented below along with the PFS response.

NRC Requests/Questions

EIS RAI No. 2, Question 4-7

The NRC has reviewed the meteorological data provided with this RAI response. The wind speed data for the dates of 1/13/97 through 1/15/97, 1/30/97, 2/3/97, and 4/14/98 seems unusually constant in both speed and wind direction. Is this data correct or has there been a malfunction of the recording equipment?

RESPONSE: PFS has evaluated the persistent wind speeds for the periods 1/13-15/97, 1/30/97, 2/3/97, and 4/14/98. These conditions were noted when the data were originally reviewed, especially the 1/13-15/97 period, and the data were examined for behavior that might indicate some sort of instrument problem. In these cases, the winds for the periods in question (0.3 mph) are indicative of calm conditions caused by high pressure areas over the site region. The barometric pressures measured at the site and at other locations where concurrent data are available such as Dugway, Muskrat Springs, and Salt Lake

City indicate high pressure for these time frames. Readings for the four periods were generally on the order of 1020, 1030, 1020, and 1010 millibars (adjusted to sea level), respectively. Available wind speed data from these other locations also indicate generally light or calm winds during these periods, for the most part, keeping in mind that site specific conditions can be somewhat different from these other locations.

PFS also examined surface maps for the 1/13-15/97 period and found persistent high pressure over the site that would lead to such calm conditions. Given the recent installation and calibration of the instruments and subsequent semi-annual calibrations indicating no problems. PFS believes that these calm winds reflect actual conditions and there is no reason to discredit the data.

EIS RAI No. 2, Question 4-21

The NRC stated that the proprietary response provided to this RAI (Reference 3) was adequate. However, the non-proprietary response provided (Reference 2) did not include enough information. PFS is requested to provide a qualitative assessment of doses at the Wyoming site and resubmit the non-proprietary response to question 4-21.

RESPONSE: A revised non-proprietary response to EIS RAI No. 2, question 4-21 is enclosed. This response supercedes the previous response provided in Reference 2.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely

JL Cooper for

John L. Donnell
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