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CHAPTER 1

INTRODUCTION AND GENERAL DESCRIPTION OF FACILITY

1.1 INTRODUCTION

Electric utilities operating nuclear power plants in the United States are rapidly reaching their maximum capacity for onsite storage of spent fuel. The Nuclear Waste Policy Act (NWPA) of 1982 mandated that the Department of Energy (DOE) was responsible for the permanent disposal of spent nuclear fuel from the nation's commercial nuclear power plants. The NWPA obligated DOE, beginning not later than January 31, 1998, to dispose of the spent fuel. In a December 17, 1996 letter to all utilities, DOE stated that it would not meet the 1998 deadline. As a result, utilities have had to plan for alternate means of interim storage for their spent fuel beyond 1998.

One such alternate means of spent fuel storage includes dry cask storage. Using this concept, a consortium of utilities have joined in a cooperative agreement through the Private Fuel Storage L.L.C. (PFSLLC) with the Skull Valley Band of Goshute Indians (Band) to undertake the development, licensing, construction, and operation of an Independent Spent Fuel Storage Installation (ISFSI) called the Private Fuel Storage Facility (PFSF). The PFSF will be built on the Skull Valley Indian Reservation and will provide timely, centralized, cost-effective spent fuel storage capacity to meet the needs of the utilities and provide long-term, stable financial income, employment, and training opportunities for Band members and the surrounding community. Preservation of the site and surrounding environment has resulted in the adoption of a "Start Clean / Stay Clean" philosophy that will permit utilization of the land and all buildings constructed in this project for other traditional industrial uses after the facility is decommissioned.

The PFSF will utilize the dry cask storage technology. Dry cask storage safely stores spent nuclear fuel inside of sealed canisters rather than in a spent fuel pool. The storage system technology is compatible with the long-term plans of the DOE interim storage facility and permanent repository. The PFSF is designed to store spent fuel for up to 40 years by which time it is anticipated that all of the spent fuel will be transferred offsite and the facility ready for decommissioning. The initial request for a license is for a term of 20 years. Prior to the end of the initial license term an application for license renewal will be submitted.

The PFSF is located on the Skull Valley Indian Reservation in Tooele County, Utah, approximately 27 miles west-southwest of Tooele City[1] (see Figure 1.1-1). There are no major towns within 10 miles of the PFSF site. The Skull Valley Band of Goshute Indian village is approximately 3.5 miles east-southeast of the site. This village has approximately 30 residents.

The reservation consists of approximately 18,000 acres, of which the PFSF site area is approximately 820 acres. Interstate Highway 80 and the Union Pacific Railroad main line are about 24 miles north of the site. Due to the proximity of the PFSF to the railroad mainline, the shipping cask will either be off-loaded at an intermodal transfer point 1.8 miles West of Timpie, Utah, and loaded onto a heavy haul tractor/trailer for transporting to the PFSF, or transported via a new railroad line connecting the PFSF directly to the Union Pacific mainline at Low Junction. The PFSF will be accessed by a new road from the Skull Valley Road as shown on Figure 1.1-2.

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 (Reference 1), prior to June 2002 in order to commence operation of the PFSF. Construction of the

[1] Tooele City is used to distinguish the City of Tooele from Tooele County.

1.4 SPENT FUEL TRANSPORTATION TO THE PFSF

Although transportation activities are not part of the 10 CFR 72 license application for the PFSF, the transportation process is briefly described herein to provide an understanding of the overall program. Spent fuel enroute to the PFSF will be transported in accordance with applicable U.S. Department of Transportation (DOT) regulations (49 CFR 173-Shippers General Requirements for Shipments and Packaging, Subpart A-"General" and Subpart I - "Radioactive Materials", 49 CFR 171-"General Information, Regulations, and Definitions", 49 CFR 172-"Hazardous Materials Tables and Hazardous Materials Communications Regulations", 49 CFR 174-"Carriage by Rail", 49 CFR 177-"Carriage by Public Highway"), and NRC regulations (10 CFR 71 - "Packaging and Transportation of Radioactive Material"). Holtec is supplying their Holtec International Storage, Transport, and Repository Cask System (HI-STAR 100) (Reference 7) and SNC is supplying their TranStor Shipping Cask System (TranStor) (Reference 8).

As a result of adherence to strict controls, utilities and carriers have a long history of safe spent fuel transportation. In more than 20 years of shipping fuel in the United States, no accident has caused a release of radioactive material. Moreover, no deaths or serious injuries to the public or to transportation industry personnel have ever occurred as the result of the radioactive nature of any radioactive material shipment (Reference 9).

Currently there is no direct rail line to the PFSF. Therefore the PFSF will be designed to employ two transport vehicle modes to ship the cask from the railroad mainline to the site. The preferred mode is to ship the shipping cask the final 32 miles by rail on a new rail line. The alternate mode is to transfer the shipping cask from the rail car to a heavy haul transport tractor/trailer at an intermodal transfer point located 1.8 miles West of Timpie and haul the shipping cask the final 26 miles by road to the PFSF. The PFSF

is expected to receive 100 to 200 shipments of loaded spent fuel canisters annually. The PFSF will accept delivery and perform receipt inspection of the spent fuel shipping casks at the PFSF.

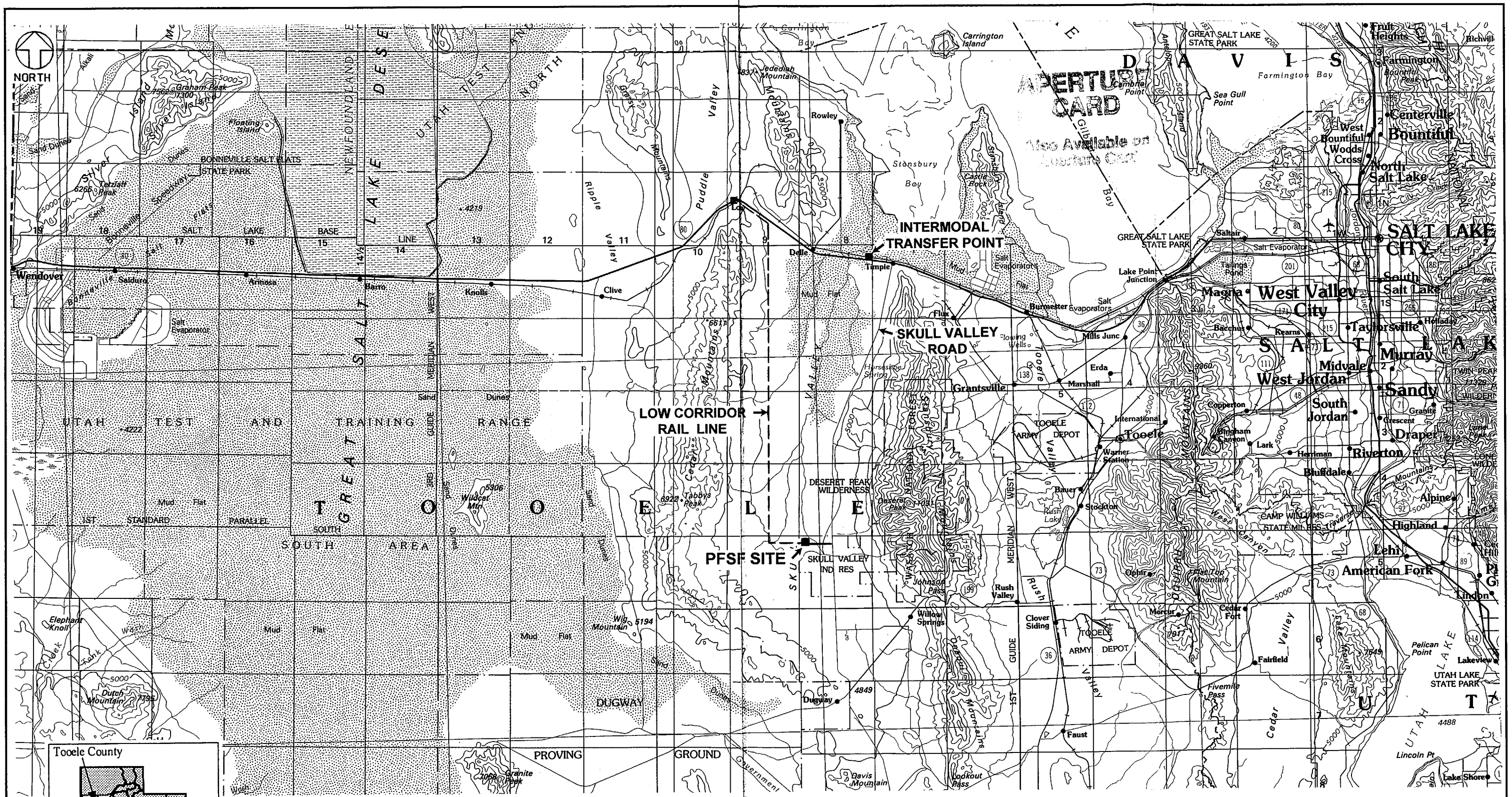

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Figure 1.1-1

PFSF LOCATION

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

Revision 3

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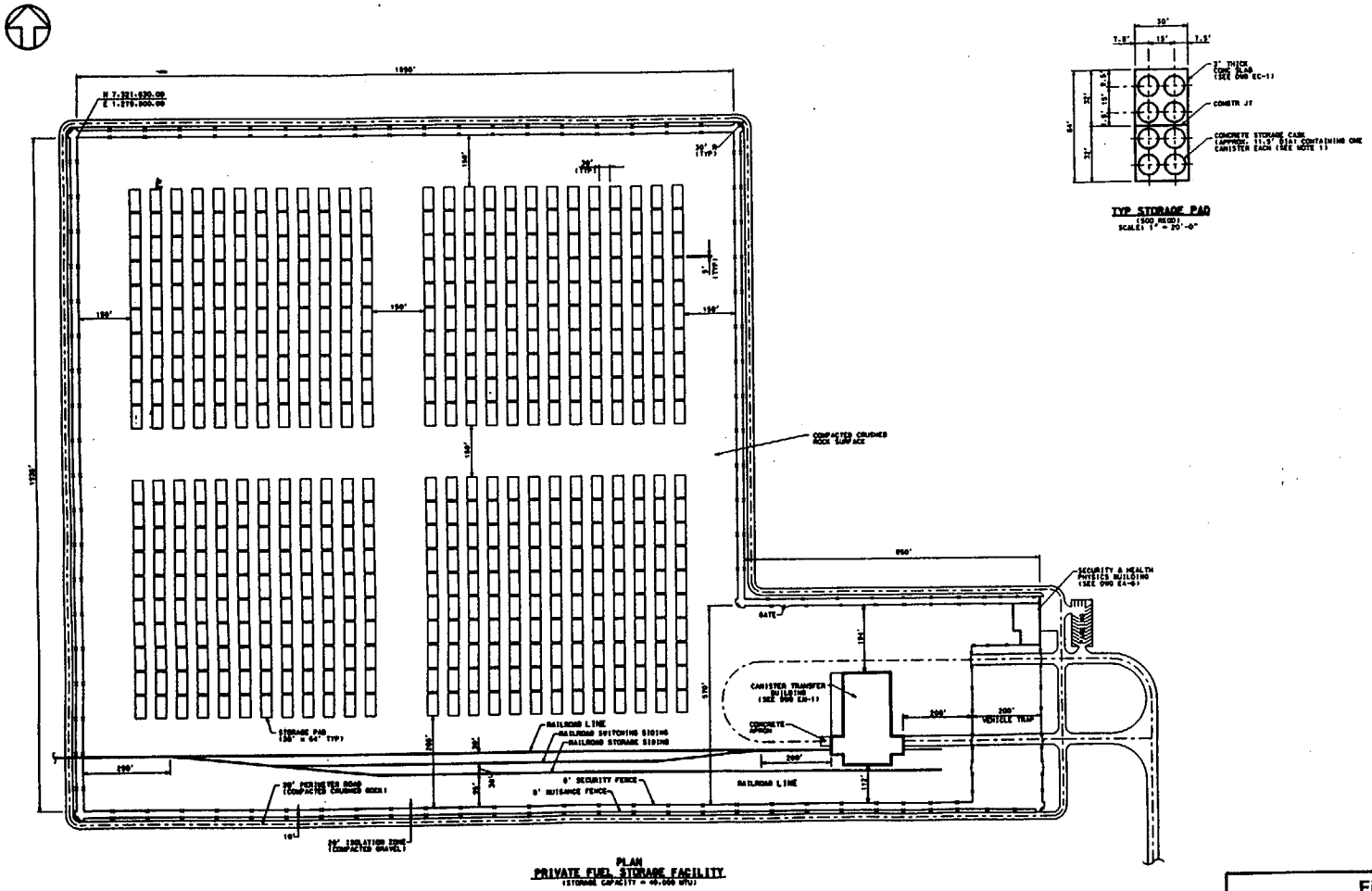

Figure 1.2-1

PFSF GENERAL ARRANGEMENT

PRIVATE FUEL STORAGE FACILITY
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2.4.1.1 Site and Structures

The land surface at the site is approximately 4,465 ft elevation and is nearly flat, sloping gently to the north. A few shallow, dry washes and former beach or lake bottom features provide slight relief to the site area. Desert shrubs and grasses form a thin vegetative cover.

No streams that would even be considered intermittent cross the facility area. The closest stream with a significant channel crosses the northeast corner of Section 6 and the center of Section 5, about 1,500 ft from the northeast corner of the facility (Figure 2.4-1). The channel is up to 3 ft deep and 6 to 8 ft wide in some areas. It carried no water during the observation period between June 1996 and February 1997.

The facility and its structures are described in Chapter 1. The PFSF storage area is approximately 99 acres. The storage area is graded for surface drainage with slopes from south to north of approximately 0.5%. Site elevations vary from approximately 4,460 ft at the north to 4,470 ft at the south. A stormwater detention basin is located on the north side of the storage area to collect runoff from the storage area.

2.4.1.2 Hydrosphere

Watersheds contributing runoff to the areas of the access road and the 3-mile-long rail road adjacent to the PFSF site are shown in Figure 2.4-1. Watershed runoff contributing

to the access road area is primarily from the east mountain range of the valley, and is designated as Basin A in Figure 2.4-1. Basin A is a watershed comprising an area of approximately 270 square miles. Watershed runoff contributing to the 3-mile-long rail road adjacent to the PFSF site is from the west mountain range on the west of the valley, and is designated as Basin B in Figure 2.4-1. Basin B is a watershed comprising an area of approximately 64 square miles (40,960 acres). The PFSF is separated from Basin A and Basin B by an earthen berm proposed for construction to keep out runoff from the two basins. The topography and approximate sheet flow directions in the site vicinity are also shown in Figure 2.4-1.

A major portion of Basin A runoff originates in the east upland extending from the lower Stansbury Mountains to the Lookout mountain in the south. Runoff is drained into the valley by a number of intermittent streams (see Figure 2.4-1). The flow from the mountain front, after crossing the alluvial fans at the foothills of the mountains is quickly lost to the pervious sublayer and evapotranspiration to become an intermittent stream. Stream flow would be produced only by very heavy rainfall or during snowmelt conditions.

Basin B runoff is primarily from the upland in the Lower Cedar Mountains on the west of PFSF. The runoff converges to the 3-mile-long rail road through many small streams. Similar to Basin A, these small streams are normally dried, stream flow can only be seen after a heavy thunder storm.

During site visits conducted between June 1996 and February 1997, several hydrologic observations were made. No perennial streams were observed to cross Skull Valley road from the uplands to the east, nor were any perennial streams observed west of the site to the base of the Cedar Mountains. There are no upstream or downstream flow control structures whose failure could conceivably affect the site or its access road. The only

structures located in the area are very small reservoirs in the foothills used as stock ponds or for collection of water for irrigation purposes.

Hood and Waddell (1968) indicate that the groundwater table in Skull Valley in the vicinity of the site ranges from elevation 4,300 to 4,350 ft. The groundwater table at the site is at approximate elevation 4,350 ft, a depth of approximately 125 ft based on a monitoring well near the Canister Transfer Building (CTB), installed in 1998. This value is consistent with the upper bound of the range of depths to the water table reported by Geosphere Midwest (Appendix 2B), who report seismic refraction results indicate that the water table may be located at depths of between 90 and 136 ft (Seismic Lines 1, 2, & 3) below existing grade. Because of the great depth to the groundwater table, it is very unlikely that the groundwater regime could have any influence on the stability of structures at the site.

2.4.2 Floods

There is no evidence the site area has experienced flooding in the past. Storm-induced runoff will provide sheet flow toward the site which will easily be controlled by construction of short diversion berms near the southern portions of the PFSF.

Analyses of the probable maximum precipitation were performed to determine a PMF for stormwater drainage Basins A (SWEC, 1999a) and B (SWEC, 1999c). The analyses demonstrated that the site would not be in the flood plain caused by any flood event.

2.4.2.1 Flood History

The PFSF site is located in an area of western Utah with a semi-arid climate, receiving average annual precipitation of 7 to 12 inches (Hood and Waddell, 1968). There are no

perennial water courses within 4 miles of the site. The nearest streams are high gradient streams that drain the slopes of the Stansbury Mountains through steep-walled canyons. This flow is quickly lost to the unconsolidated sediments comprising the alluvial apron at the foot of the mountains and becomes part of the groundwater system. No perennial surface flow makes its way across Skull Valley road which runs north-south, approximately 1.5 miles east of the PFSF site.

There is no evidence of past flooding in the site area and only minor development of drainage channels created by infrequent thunderstorms (<1 to 2 ft deep). There is no evidence of flash-flooding in the area, such as flood deposits, nor are there channels that could affect the site if they were subject to a flash flood.

The only conceivable scenario for floods would involve a return to climatic conditions of the Late Pleistocene causing a significant rise (~300 ft) in the level of Great Salt Lake. Those conditions generally require millennia to develop, therefore, this scenario is dismissed.

2.4.2.2 Flood Design Considerations

Hypothetical Probable Maximum Precipitation (PMP) events were analyzed to determine maximum flooding elevation at the PFSF site due to flood flows from Basin A and Basin B. The analyses included the general storm and the local storm events, as discussed below in Section 2.4.2.3. Determination of Probable Maximum Flood (PMF) was based on the procedures given in Hydrometeorological Report (HMR) 49 (U.S. Department of Commerce, 1977). In Basin A, the PMF is generated by a general storm, while in Basin B, it is generated by a local storm.

For an extremely conservative PMF (flow = 85,000 cfs for Basin A, 102,000 cfs for Basin B, detailed in Section 2.4.2.3), results of hydraulic analysis indicate that maximum water surface elevation is predicted to be 4,506.4 ft upstream of the access road and the PMF berm in the east floodway with runoff from Basin A, and 4,478.0 ft upstream of the rail spur and the PMF berm in the west floodway from Basin B. Both of the predicted flood elevations are below the designed top elevations of the PMF diversion berms of 4,507.5 ft and 4,480 ft, respectively. At cross sections downstream of the access road, rail spur and the PMF berms, the predicted flood elevations (SWEC 1999d) are below the designed site grade elevations of 4,463 ft to 4,475 ft. Consequently, all Structures, Systems, and Components (SSCs) classified as being Important to Safety are located above the PMF flood plains.

Figures 2.4-2, 2.4-3, 2.4-4 and 2.4-5 show a plan and profile (elevation) view of the PFSF site PMF berm, the PFSF rail line, the PFSF access road, and the PFSF access road PMF berm, respectively. In each of these figures, the maximum PMF water level is also shown in the profile view.

Stormwater runoff from Basin A and Basin B drains as a sheet flow toward the PFSF site. An earthen berm (PMF berm) and drainage ditch system will be constructed on the south and west sides of the PFSF storage site to divert the PMF stormwater flows around the site to the east floodway and the west floodway. Consequently, all Structures, Systems, and Components (SSCs) classified as being Important to Safety are protected from the sheet flows associated with the PMF from Basin A and Basin B by the PMF berm.

The PFSF site drainage systems (both offsite and onsite) are designed for the 100-yr. storm event. Offsite drainage system design due to Hickman Knolls runoff is conveyed around the south and west sides of the PFSF. This flow is then discharged at a

permissible velocity to the Skull Valley natural drainage system. Flows resulting from a storm event more severe than a 100-year event from Hickman Knolls are also diverted into the Skull Valley drainage system. Onsite drainage system design due to local runoff is conveyed by a surface flow system utilizing swales channeled to a stormwater collection and detention basin where it can evaporate and seep into the soil.

The PFSF access road and the rail road drainage systems are designed to safely convey the surface water under the roadway during a 100-yr. storm event. During a PMP, the excess runoff will overtop the access road embankment and the rail road embankments. The flood overflow will be contained with a north-south berm tied into Hickman Knolls to prevent flows from approaching the PFSF site. Downstream of the access road and the rail road, the PMF returns to the natural flow conditions. Access to the site by normal vehicular traffic, as well as emergency vehicles, will be provided at all times except during a storm which is more severe than a 100-yr. storm event.

2.4.2.3 Effects of Local Intense Precipitation

The PMP was estimated based on HMR 49. HMR 49 requires different evaluations for combinations of storm events. Although the PMF generated by a general storm is a major concern in the evaluation of maximum flow, the local thunder storm in the summer time, having the greatest potential rain fall intensity and short duration may produce a larger PMF. Consequently, the PMP for both general storm and local storm must be analyzed. The PMP events are used with the HEC-1 program (U.S. Army Corps of Engineers, 1990) to determine peak discharges. After these evaluations are completed, the largest peak discharge is selected as the PMF. The water surface profile corresponding to the PMF is determined with the HEC-RAS (U.S. Army Corps of Engineers, 1997) backwater program.

According to the procedures given in HMR 49, the total precipitation for the general PMP were estimated to be approximately 10.7 and 10.5 inches during a 24-hour duration for the month of August (SWEC, 1999a, 1999c) at the Basin A and Basin B, respectively.

The total precipitation during the local PMP were estimated to be approximately 14.1 and 12.7 inches during a 6-hr duration at the Basin A and Basin B, respectively.

The 1-hr local PMP induced by a short duration thunderstorm over a 1 square mile basin at the site was determined to be 10.1 and 9.8 inches at the Basin A and Basin B, respectively. Durational variation and areal reduction described in HMR 49 were employed to estimate the incremental PMP.

The total duration of the PMP was divided into six increments. The sequence position of hourly incremental PMP for the 6-hr thunderstorm was arranged in accordance with HMR No. 5 (U.S. Weather Bureau, 1947) as shown in the following table:

Increment	HMR No. 5 Sequence Position
Largest hourly amount	Third
2nd largest	Fourth
3rd largest	Second
4th largest	Fifth
5th largest	First
least	Last

The local PMP depths at 15-minute intervals for the first hour were determined by the procedures provided in HMR 49. The remaining 15-minute intervals were estimated

from the depth-duration curve as documented in the calculation (SWEC, 1999a, 1999c). The sequence of the four 15-minute incremental PMP has the greatest intensity in the first 15-minute interval. The second, the third, and the fourth largest were placed after.

The general PMP depths at 1st, 6th, 12th, 18th, 24th, 48th, and 72nd hours were determined by HMR 49. The remaining 3-hr increments can be found from the depth-duration relationship. Time distribution procedures given by the Corps of Engineers (U.S. Army, 1952) were used for all other storms except the local PMP. The total duration of the storm was divided into four increments. The increments were arranged in a sequence with the largest one at the middle and decreased progressively to either side of the greatest increment.

The HEC-1 computer program (US Army, COE, 1990) was used to determine the peak discharge. Soil type and its corresponding hydrological group, runoff curve number (CN), were estimated based on USDA's unpublished county soil report. The Utah Division of Water Resources suggested CN=70 for the type of soil surface condition in the Skull Valley. Consequently, CN=70 was assumed in the calculation.

The Kirpich formula (Chow, 1964) was used to estimate the time of concentration (Tc) for the overland flow on grassed surface. Based on the measured watershed length and the slope of the basin, the time of concentration at Basin A was calculated to be 11.24 hours. For Basin B, the time of concentration was estimated to be 4.17 hours, by using the regression equation developed by U.S. Corps of Engineer ("Time of Concentration vs Drainage Area for Mountain Watershed in Utah", no date, provided by Water Resource Division, State of Utah).

Based on the HEC-1 ,model computer output, the results of the PMF are summarized in the following:

	Basin A	Basin B
$Q_{Local\ PMF}$ =	40,237 cfs	68,500 cfs
$Q_{General\ PMF}$=	52,983 cfs	20,972 cfs

The larger peak discharge is selected as the PMF for the basin:

Q_{PMF}=	53,000 cfs	68,500 cfs

An extremely conservative case that is unlikely to happen in the Skull Valley drainage basin was also analyzed. In Basin A, assuming CN = 96 and Tc = 11 hours, the calculated PMF = 85,000 cfs. In Basin B, assuming CN=96, and Tc = 4.17 hours, the calculated PMF = 102,000 cfs. A CN = 96 is equivalent to an impervious surface or a saturated ground condition.

For the design of the access road and the railroad spur, the 100-year floods for Basin A and B were estimated (SWEC, 1999a, 1999c) from the FHWA method (FHWA, 1977) and USGS method (USGS, 1994). The results of the 100-year flood are summarized in the following

	Basin A	Basin B
$Q_{100\ (FHWA)}$ =	2,430 cfs	1,391 cfs
$Q_{100\ (USGS)}$ =	2,317 cfs	936 cfs

The larger peak flow is selected as the 100-year flood for the basin

Q_{100} =	2,430 cfs	1,400 cfs

As described above, the extremely conservative PMF flows were calculated to be 85,000 cfs and 102,000 cfs for Basin A and Basin B, respectively. The PMF would most likely flow to the north along the east and west fringe of the PFSF site to Great Salt Lake. The peak discharges of 85,000 cfs and 102,000 cfs were used for backwater computation. The computer program HEC-RAS (US Army, COE, 1997) was used to compute Basin A and Basin B maximum flood profile elevations at various cross section and flood plains at the vicinity of the site (SWEC, 1999a, 1999c, 1999d).

For Basin A, with existing natural topography the predicted maximum water elevation is 4,501.3 ft at the access road (SWEC, 1999a). The predicted flood levels at the southeast and northeast corners of the site are approximately 4,468.8 ft and 4,456.7 ft, respectively. The site elevation of 4,476 – 4,462 ft is above the calculated water elevations. Consequently, the site is not in Basin A PMF flood plain.

In addition, the PMF flood levels for Basin A and B after the construction of the access road and rail line embankment were calculated. The predicted maximum water elevation at the access road and rail road is 4,506.4 and 4,478.1 ft, respectively.

All Structures, Systems, and Components (SSCs) classified as being Important to Safety are protected from the sheet flow associated with Basin A and B PMF by an earthen berm to be built with top elevation at 4,507.5 and 4,480.0 ft, respectively.

2.4.3 Potential Maximum Flood on Streams and Rivers

Since there are no perennial streams or rivers in the vicinity of the PFSF, a PMF analysis is not required on streams and rivers. However, an ephemeral stream bed is present; therefore, PMF analyses were performed for this ephemeral stream as described above in Section 2.4.2.

2.4.4 Potential Dam Failures (Seismically Induced)

There are no flow control structures on any stream upgradient from the site; therefore, there is no potential for impact on the site from potential dam failures.

2.4.5 Probable Maximum Surge and Seiche Flooding

No surge or seiche flooding is possible, as there is no large water body near the site.

2.4.6 Probable Maximum Tsunami Flooding

The site is not located near a coastal area. As a result no tsunami sea waves are anticipated.

2.4.7 Ice Flooding

There are no water bodies near the site on which ice flooding conditions could arise.

2.4.8 Flooding Protection Requirements

All Structures, Systems, and Components (SSCs) classified as being Important to Safety are protected from flooding by diversion berms to deflect potential flows generated by PMF from both the east mountain range (Basin A) and the west mountain range (Basin B) watersheds.

2.4.9 Environmental Acceptance of Effluents

With the exception of the sanitary system, there are no liquid releases that result from the normal operation of the PFSF.

2.5 SUBSURFACE HYDROLOGY

2.5.1 Regional Characteristics

Skull Valley is a north-trending valley extending 50 miles from Lookout Pass in the Onaqui Mountains, to the southwest shore of the Great Salt Lake. It is one of many linear valleys of the Basin and Range bordered by relatively young fault-block mountains. These blocks are composed mainly of limestone and dolomite with a few beds of quartzite, sandstone, and shale, ranging in age from Early Cambrian to Tertiary. Primary permeability of these rocks is generally low; secondary permeability exists as joints, fractures, faults, and bedding plane separations.

A large portion of the precipitation that falls in the uplands runs off the steep hillsides as spring snowmelt in short, high-gradient streams, with little infiltration into the mountain blocks. Another portion drains eastward, becoming part of the hydrologic system of the adjacent Tooele and Rush valleys while some is discharged as springs in the foothills along the edges of the valley.

Another portion enters the valley-fill aquifers through an extensive recharge area consisting of alluvial fans at the base of the ranges. Hood and Waddell (1968) estimated the long-term average annual runoff from the uplands is about 32,000 acre-feet with only a small part of this actually flowing out of Skull Valley. They estimated the average annual groundwater discharge and recharge is between 30,000 to 50,000 acre-feet with evapotranspiration accounting for 80 to 90 percent of the total discharge.

The valley-fill deposits are unconsolidated and semi-consolidated rocks of Tertiary and Quaternary age. They consist of inter-stratified colluvium, alluvium, lacustrine, and fluvial deposits with minor basalt and ash, and some eolian material. These sediments

are derived almost entirely from the surrounding uplands and constitute the main groundwater reservoir.

In general, the coarser deposits are near the perimeter of the valley, grading into well-sorted sand and gravel, and interlayered with lacustrine silt and clay towards the center of the valley. Thick beds of clay exist in some areas and may create local, confined aquifers where they interfinger with sand and gravel along the alluvial fans.

The Salt Lake Group of Tertiary age comprises the majority of the valley fill ranging in thickness from 2,000 ft to over 6,000 ft (Arabasz et al., 1987). The overlying Quaternary rocks are mainly silt and clay, Pleistocene lake sediments, and reworked lake sediments. Sack (1993) has recently mapped and described the various Quaternary and Holocene surficial deposits in Skull Valley.

The Tertiary and older Quaternary deposits are slightly to highly permeable, depending upon grain size and degree of cementation. The deeper, more consolidated deposits contain some volcanic deposits that may reduce the permeability. The Tertiary and Quaternary deposits probably contain most of the groundwater of usable quality in storage in this part of Utah.

The younger Quaternary and Holocene sediments in the valley bottom have generally low permeability except for areas of windblown sand, and old beach and bar deposits. Precipitation on or surface runoff to the valley bottom remains ponded until it evaporates. The precipitation that is absorbed does not reach the water table in the southern and central parts of the valley because of the depth of the water table, the low permeability of the soil materials, and the low amount of precipitation. Most of this water is captured by plants and transpired; a small portion evaporates directly through

capillary action and contributes to the development of a high alkali content in the surface soils.

Groundwater flow is generally northward toward the Great Salt Lake. Hood and Waddell (1968) calculated that with a transmissivity of 2,675 sq ft/day, the annual volume of underflow out of the valley is about 800 acre-feet per year. Pumpage from wells for all purposes was estimated at 5,000 acre-feet per year in Skull Valley and is not believed to have changed significantly in the last 30 years.

Domestic water wells are developed almost exclusively in the unconsolidated alluvial fan deposits along the east side of Skull Valley. This same area serves as the main recharge area for the valley. Water quality is also the highest in this area. Discrete sand and gravel lenses are sufficiently interconnected so that water moves from bed to bed as a single hydrologic unit. Groundwater is commonly between 110 and 160 ft below ground in this area.

Farther out in the valley where lake clays have been deposited between granular layers, some degree of confinement occurs and, as a result, many irrigation and stock wells are under artesian conditions. These wells are commonly drilled to depths between 250 to 500 ft but maintain static water depth of 100 ft or less. Figure 2.5-1 indicates the location of all water wells within 5 miles (8km) of the PFSF. Some well records indicate artesian flow at the ground surface from wells just south of the Reservation. This information dates from the 1940's to 1960's (Arabasz et al., Appendix F, 1987).

Groundwater quality varies significantly in Skull Valley, dependent mainly on proximity to the bordering ranges. The alluvial apron along the base of the Stansbury Mountains contains the lowest total dissolved solids (TDS) in the valley, with concentrations from

100 to 800 mg/l. In the southernmost part of the valley, TDS concentrations range from 700 to about 900 mg/l with a few isolated wells above 1,000 mg/l TDS. A well south of the Reservation yielded a TDS concentration of greater than 2,500 mg/l (Arabasz et al., Appendix F, 1987). Sodium and chloride are the major ions found in these waters.

Toward the center part of the valley, away from the alluvial apron, unconsolidated lacustrine materials are interstratified with clastic material. Wells in this area tend to have lower yields and poorer quality water (TDS > 1,000 mg/l) and are used mainly for irrigation and stock watering. The north end of the valley has generally high TDS concentrations, in the range of 1,600 to 7,900 mg/l with sodium and chloride again being the main constituents (Arabasz et al., Appendix F, 1987).

2.5.2 Site Characteristics

Based on boring data obtained at the site, the uppermost soil layer consists of interbedded silt, silty clay, and clayey silt with a thickness of approximately 30 ft. This layer is underlain by very dense fine sand and silt that extends to a depth of approximately 45 ft. At this depth a carbonate soil (the Promontory soil) formed in subaerially-deposited alluvium was encountered in numerous borings across the site. Quaternary deposits are believed to extend to a depth of about 85 ft where Tertiary Salt Lake Formation was encountered in several borings. The groundwater table was encountered in borings at a depth of 125 feet. Hood and Waddell (1968) indicate that ground water flows from the south to the north in Skull Valley, toward Great Salt Lake. Based on their Plate 1, the hydraulic gradient is estimated to be approximately 9.5 x 10^{-4}.

Soil interpretations prepared by USDA (undated) indicate that the permeability of a silt soil in Skull Valley ranges from 0.2 to 0.6 inches/hr. The average groundwater rate of flow was estimated to be approximately 2.8×10^{-3} to 8.5×10^{-3} gallons/day/sq ft. The source of groundwater flow at the site is mainly derived from precipitation that falls at the higher elevations of the Stansbury Mountains. As a result of the low permeability deposits and high evapotranspiration at the site, rainfall at the site is unlikely to contribute to groundwater flow.

Water needs during construction (8,500 gallons/day) and operation (3,600 gallons/day) of the PFSF are modest. During operation, it will be similar to a light industrial facility with a 24-hour a day contingent of security personnel. Highest water demand is associated with the larger daytime work-force as well as operation of a concrete batching plant during initial construction. It is anticipated that surface storage tanks would be erected for potable water, emergency fire water, and for the concrete batching plant, as it is unlikely that water wells drilled into the main valley aquifer would yield adequate quantities of water for these purposes on demand. Several wells on the site may be required to meet the demand. Localized drawdown of the valley aquifer would occur in the vicinity of the wells, the extent of which can not be determined until the wells are drilled, developed, and pump-tested, but which would not extend beyond the site boundary.

Based on preliminary testing of the onsite monitoring well, it has been determined that operation of the PFSF water well will have no measurable off-site effects on existing groundwater quality or levels (SWEC, 1999b).

2.5.3 Contaminant Transport Analysis

The nature and form of the material stored (spent fuel assemblies in sealed metal canisters) and the method of storage (dry casks) preclude the possibility of a liquid contaminant spill. Discussion of potential contamination of groundwater is not applicable since the depth to groundwater at the site is substantially removed from any activity at the site finished grade.

2.6 GEOLOGY AND SEISMOLOGY

2.6.1 Basic Geologic and Seismic Information

Geological and seismological investigations for the PFSF site and area began in 1996 and included review of pertinent published and unpublished literature, consultation with geologists and seismologists familiar with the area, and reconnaissance level geologic mapping. In addition, a test boring program was conducted at the site at that time and along the 2.5-mile-long access road to characterize subsurface soil conditions. The drilling was performed by Earthcore, Inc. of Salt Lake City under the direct supervision of Stone & Webster Engineering Corporation (SWEC). Laboratory testing of soil for engineering properties was performed in SWEC's Soil Testing Laboratory in Boston, Massachusetts. Seismic reflection and refraction (both P- and S-wave) surveys were completed in 1996 in order to determine soil and rock stratigraphy, bedrock surface profiles, depth to the water table, seismic velocities of the underlying rock and soil, and engineering parameters of near-surface soils from shear wave velocities. This work was performed by Geosphere Midwest of Brooklyn Park, Minnesota, also under the direct supervision of Stone & Webster.

Professor Donald Currey of the University of Utah completed an evaluation of surficial linear features near the site at Stone & Webster's request. Drs. William Nash and Michael Perkins of the University of Utah analyzed volcanic ash samples from site borings and trench excavations for age correlation purposes.

Additional work was performed in 1998 in response to NRC Requests for Additional Information (RAI). This work included review of existing literature and data, and discussions with researchers knowledgeable with the structural and stratigraphic setting of Skull Valley. In addition, an extensive program of surface and subsurface

investigations was conducted under the direction of Geomatrix Consultants, Inc. and Stone and Webster. Their investigations included the following activities and are described in detail in Geomatrix Consultants, Inc. (1999a):

- photogeologic interpretation of aerial photos
- surficial and bedrock geologic mapping
- geomorphic mapping and interpretation
- geophysical investigations including interpretation of gravity data, conducting a magnetometer survey, reprocessing and interpretation of proprietary P-wave reflection data, and completing a high-resolution shear wave reflection survey (Bay Geophysical Associates, 1999).
- Trenching and test pit excavation and mapping
- Drilling and logging of boreholes
- Geochronologic analysis

As a result of these investigations and in response to recent changes to 10CFR50, Part 100 (100.23), as well as anticipated changes to Part 72 (SECY-98-126), probabilistic analyses were performed to evaluate the potential for fault displacement and ground shaking hazards at the PFSF. The design basis for vibratory ground motion was assessed at this time. The results of this effort are reported in Geomatrix Consultants, Inc. (1999a and b).

Stone and Webster also conducted a drilling program in 1998 in the area of the Canister Transfer Building (CTB) for engineering purposes. One of these borings reached a total depth of 225 ft. and a groundwater monitoring well was installed.

The site is situated in western Utah near the eastern boundary of the Basin and Range Physiographic Province with the Middle Rocky Mountain Province (Figure 2.6-1). This area is characterized by a series of roughly North-South trending, tilted fault block ranges

separated by down-faulted linear basins. The PFSF is located near the middle of the Skull Valley basin, at approximate elevation 4,465 ft, between the Stansbury Mountain range on the east and the Cedar Mountains on the west. The top 25 to 35 ft of surficial soils at the site are mainly lacustrine marly silts and clays. Below about 25 to 35 ft is a very dense fine sand with minor gravel and silt layers to at least 100 ft deep (Appendix 2A). The base of the late Pleistocene Bonneville alloformation is believed to be at a depth of about 45 ft. in the site area where the Promontory Soil was identified (Geomatrix Consultants, Inc., 1999a) and the soil blow-counts increase dramatically (Appendix 2A). The base of the Quaternary section is not well-constrained but the Tertiary "Walcott ash" is known in several borings at a depth of about 85 ft.

Bedrock was not encountered in the borings but is believed to occur at a depth of between 520 and 880 ft, based on seismic survey results (Appendix 2B). Bedrock outcroppings, about 1.5 miles south of the site at Hickman Knolls, have been mapped as the Fish Haven Dolomite of Late Ordovician age (Moore and Sorensen, 1979; Geomatrix Consultants, Inc., 1999a). As indicated in Section 2.4.1.2, the groundwater table is about 125 ft below grade at the site, based on the observation well and seismic refraction surveys.

The Stansbury fault, exposed along the base of the western escarpment of the Stansbury Mountains, is about 6 miles east of the site. The fault dips to the west and is projected beneath the PFSF site at a depth of 4.4 miles (55 degree dip assumed). The most recent events on the Stansbury fault displace late Pleistocene shorelines that are estimated to be about 18,000 years old. Arabasz et al. (1987) consider the Stansbury fault capable of a maximum magnitude 7.3 earthquake. Empirical relations by Wells and Coppersmith (1994) that relate surface rupture length to magnitude suggest the maximum earthquake magnitude on the Stansbury fault is 7.0 ± 0.28 (moment mag.). Helm (1995) has calculated that the next seismic event on the fault should be a Ms 6.8-6.9 ± 0.04 , based

on strain accumulation rates of previous events Geomatrix Consultants Inc. calculate an expected value (mean) of **M** 7.0 for the maximum magnitude on the Stansbury fault.

Two unnamed faults were identified in the PFSF area, and are informally named the East and West faults (Geomatrix Consultants Inc., 1999a). Late Pleistocene activity is indicated for both of these faults, based on geophysical and geomorphological studies. The East fault lies 0.9 km east of the site and the West is 2 km to the west. Mean maximum magnitudes for the East and West faults were calculated to be **M** 6.5 and **M** 6.4, respectively. The Stansbury, East, and West faults are the most important structures with respect to the assessment of seismic hazard in the PFSF vicinity. A transition zone or zone of distributive fault offset between the East and West faults was identified. The potential for surface fault displacement beneath the PFSF is evaluated in Geomatrix Consultants, Inc. (1999a).

The maximum "random" earthquake for this region has been defined by Pechmann and Arabasz (1995) as $M_L = 6.5$.

A probabilistic hazard analysis was performed to assess vibratory ground motion and fault displacement at the PFSF site. Peak accelerations for the design basis earthquake (1000-year return period) were calculated to be 0.40 g horizontal and 0.39 g vertical. Ground surface displacements associated with faults believed to exist beneath the site were determined to be less than 0.1 cm for the same return period.

The closest Quaternary igneous activity is 50 miles south of the PFSF site at Fumarole Butte on Crater Bench. Basaltic volcanic activity here is believed to be between 950,000 and 880,000 years old (Hecker, 1993) and does not present a threat to the integrity of the PFSF at that distance, if it were to become active.

2.6.1.1 Site Geomorphology

Site topography is shown on Figure 2.6-2. Topography in the site vicinity is shown on Figures 1.1-1 and 2.6-4. The PFSF site lies near the center of Skull Valley about midway between the Stansbury Mountains and the Cedar Mountains. Skull Valley is in a part of the Great Basin that was once occupied by Lake Bonneville, during the late Pleistocene (30,000 to 12,000 years B.P.).. As the climate became warmer in the latest Pleistocene and outlets for the lake were abandoned, the lake shrank in size and the water became saline. The gently north-sloping floor of Skull Valley is the former bottom of the lake and the unconsolidated deposits at the site are sediments laid down in and by Lake Bonneville. About 2 miles east of the site, the valley bottom meets the toe of an alluvial apron built up from a series of coalescing alluvial fans along the base of the Stansbury Mountains. The apron slopes at about 200 ft/mile in the vicinity of the Skull Valley Indian Reservation village. A wave-cut bench or terrace can be seen near the head of the apron representing the maximum level of Lake Bonneville about 15,000 years B.P. at elevation 5,240 ft. Also present here is a scarp and small graben in Quaternary deposits reflecting Quaternary movement on the Stansbury fault (Barnhard and Dodge, 1988; Geomatrix Consultants, Inc. 1999a).

The apron is only slightly dissected by streams originating in the steep bedrock terrain of the Stansbury Mountains. Stream and spring flow is rapidly absorbed into the coarse granular fan deposits resulting in very little water reaching the valley bottom as surface runoff in this area.

The valley floor is relatively smooth, being interrupted in only a few locations by bedrock outcrops, such as Hickman Knolls rising about 400 ft above the valley bottom near the site. Relief on the valley bottom is slight consisting of a few shallow (1 to 3 ft) north-trending dry washes and low (1 to 3 ft) linear soil ridges. The washes are marked by

more dense desert shrub vegetation, whereas the ridges tend to be grass covered. The washes carry water for very short periods during spring snowmelt and infrequent, local thunderstorms. A few shallow depressions appear to pond water at times until it evaporates. This network of shallow washes eventually leads offsite to the north where it joins the central valley drainage system leading to the Great Salt Lake. Perennial surface water is found about 10 miles north of the site in a large mudflat fed mainly by springs along the toes of alluvial fans emanating from the Stansbury Mountains.

Other features recognized on the valley bottom near the site include beach ridges and shoreline deposits associated with Lake Bonneville and eolian dune deposits in various forms, mainly parabolic or shrub-coppice dunes (Sack, 1993). Sack (1993) interpreted linear and curvilinear tonal features from air photos as possible fault traces in soil near the site. Dr. Donald Currey of the University of Utah examined these features in the field and believes they are not of tectonic origin, but are beach ridges developed during the transgression of Lake Bonneville. His report on these features is included as Appendix 2C. Subsequent trenching and augering of these features have affirmed Currey's original interpretation (Geomatrix Consultants, Inc., 1999a).

There is no evidence of flash-flooding near the site area nor any deposits indicative of mudflows or recent landslides. The great depth to bedrock and the very dense condition of most subsurface soils preclude the development of collapse or uplift features associated with karst terrains or tectonic depressions. There is no history of mineral extraction or injection in the area and little likelihood of future development. Withdrawal of water in the area is widely scattered and consists of a few domestic supply wells, irrigation wells, and stock-watering wells. There is no potential for subsidence from water withdrawal because of the distance from these sources and the present depth to water at the site (greater than 100 ft).

In summary, the geomorphology of the site is typical of a semi-arid to arid desert setting. The adjacent ranges are affected by mass-wasting processes and stream erosion that deliver their load of sediments to a complex of alluvial fans at the edge of the ranges. Most of the sediment load is dropped here as the water infiltrates or evaporates. The central part of the valley is relatively unaffected by fluvial processes. Mechanical and chemical weathering of rock and soil proceeds very slowly in this flat dry environment. Essentially, the only geomorphic processes to affect the site are microprocesses wherein soil moisture from occasional precipitation is drawn upward by capillary action and evaporates near the ground surface. This results in a gradual buildup of calcium carbonate, alkali, and sulfate in the near-surface soils. Soils at the site are described in the County soil report (USDA, unpublished report) as being calcareous and saline.

2.6.1.2 Geologic History of Site and Region

2.6.1.2.1 Bedrock

The Skull Valley PFSF site lies above a sediment-filled, structural basin that is bounded on the east and west by uplifted range blocks, the Stansbury-Onaqui Mountains and the Cedar Mountains, respectively. This pattern is repeated throughout western Utah and Nevada and elsewhere and is so characteristic that the name Basin and Range is applied to the physiographic area containing this structural arrangement (Figure 2.6-1). The eastern border of this province is generally drawn along the North-South trending Wasatch Front about 55 miles east of the site. The western boundary of the Front is known to be a major, active normal fault, the Wasatch fault, along which the Front has been uplifted and the Salt Lake basin is down-dropped. This major structural element is believed to have persisted since at least Late Precambrian time. The Uinta arch, which includes the present Uinta Mountains east of the Wasatch Front, is an east-west trending, anticlinal structure with a similarly long history of uplift. It intersects the Wasatch line at

right angles and is believed to have influenced sedimentation patterns, as well as provided a stable buttress during tectonic episodes. Evidence of the Uinta arch has been traced as far west as central Nevada (Roberts et al., 1965) and is postulated to have affected sedimentation patterns in the rocks of the Stansbury Mountains and patterns of faulting and mineralization (Zoback, 1983; Helm, 1995; Stokes, 1986). The regional bedrock geology is depicted on Figure 2.6-3. Structural geologic cross sections were drawn north and south of the PFSF site across Skull Valley. These profiles are included in Geomatrix Consultants, Inc. (1999a) as Figures 2-1 and 2-2 with their locations shown on Figure 1-1.

The Wasatch line may have its origin during the Late Precambrian breakup of the North American craton as the resulting rift margin. Clastic deposition off the craton margin eventually became carbonate shelf deposition as the shoreline migrated eastward. The site of this deposition is believed to be the Cordilleran geosyncline, receiving some of the greatest thicknesses of Precambrian and Paleozoic sediments found anywhere (Stewart, 1976). Shallow marine deposition persisted throughout much of the Paleozoic, except along the Uinta arch where periodic uplift caused erosion of previously deposited sediments or non-deposition. Orogenic events during the Mid to Late Paleozoic, such as the Antler orogeny, greatly affected the edge of the continent, then located in Nevada, and detrital sedimentation became predominant.

During the Triassic and Jurassic Periods, shallow marine deposition alternated with long episodes of subaerial erosion and deposition, mainly east of the Wasatch line. No sedimentary rocks from this period are known in the site vicinity (Moore and Sorensen, 1979). Orogenic events continued to affect the Cordilleran geosyncline, compressing and uplifting these sediments progressively from west to east.

A distinct orogenic event has been identified in western Utah, occurring during the Late Cretaceous. It is called the Sevier orogeny and is characterized by extensive tectonic shortening of the geosyncline and eastward folding and thrusting of sediments along numerous low angle thrust faults. Tooker (1983), Tooker and Roberts (1971), and Christie-Blick (1983) discuss evidence for these thrust sheets in several mountain ranges near the PFSF site, including the Stansbury, Cedar, Sheeprock, and Oquirrh Mountains. The Uinta arch and the Wasatch line are believed to have influenced the lateral extent and placement of some of these thrust sheets.

The Laramide orogeny closely followed the Sevier orogeny beginning in the Late Cretaceous and ending in early Eocene time. Its effects were felt mostly east of the Wasatch line; uplift of the Uinta Mountains by block faulting is one of the results of this event. The Laramide orogeny ended in the mid-Eocene with a major change in the regional stress regime. Extensional tectonics replaced compressional and were accompanied by widespread igneous activity. Normal faulting was the common expression of the extension.

The timing of development of basin-and-range tectonics varies considerably in the western U.S. and several theories of origin have been proposed. Stewart (1978) provides a summary of much of the evidence, based mainly on age relationships to volcanism associated with the extension of the crust. He concludes that extensional faulting of the Great Basin, of which the Skull Valley area is a part, probably started about 17 million years ago (m.y.a.). Moore and McKee (1983) conclude basin-and-range faulting is no older than 12 to 13 m.y. in the Stansbury Mountains, based on ages of basalt flows there. Evidence of Quaternary movement is indicated by the fault scarps offsetting Quaternary deposits along the west side of the Stansbury Mountains, as well as numerous other areas in the Great Basin. Seismicity along some of these faults, in situ stress

measurements, and earthquake focal plane solutions (summarized in Smith, 1978) indicate extensional activity continues today.

2.6.1.2.2 Site Area Structural Geology and Geologic History

The Stansbury Mountains are a north-trending range in the eastern Great Basin. They are about 30 miles long and 10 miles wide with a maximum elevation of 11,031 ft at Deseret Peak, just northeast of the PFSF. A smaller range, the Onaqui Mountains, is an extension of the Stansbury range to the south. A generalized profile of the range includes a steep, curvilinear western slope dissected by a series of young canyons. The eastern slope into Tooele and Rush Valleys is much less rugged, perhaps reflecting a typical tilted fault block origin.

The overall structure of the range is a doubly-plunging anticline uplifted along the steep westerly-dipping Stansbury fault (Tooker and Roberts, 1971). The stratigraphy and general geology of the Stansbury Mountains has been studied most completely by Rigby (1958). He divided the history of the range into two episodes of uplift and folding, each followed by long periods of erosion. These activities took place with the rocks at or near the present location of the range.

Tooker and Roberts (1971), Tooker (1983), and Roberts et al. (1965) reinterpreted much of Rigby's work to include a sequence of at least four thrust slices derived mainly from a site an unknown distance to the west. Most of the folding and faulting internal to the mountain block is believed to have occurred prior to or during the deformation that carried these rocks to their present location during the Late Cretaceous. Additional details of these older events and cross sections through the area are found in Geomatrix Consultants, Inc. (1999a), Section 2. Regardless of the provenance of the rocks in the Stansbury Mountains, it is evident the range itself is the result of normal faulting

associated with extensional tectonics developed since the beginning of the Miocene, about 24 m.y.a. This process continues today and is a major focus of seismic safety considerations for the PFSF, as well as other facilities in the region.

The Stansbury Mountains are but one of numerous mountain ranges in the Great Basin with similar origins and characteristics. The ranges are oriented roughly North-South, are commonly 9 to 12 miles wide, and are separated by valleys or basins filled with alluvium and colluvium derived from the ranges. The thickness of sediment in the valleys ranges from 1,000 ft to as much as 12,000 ft. Elevation of the ranges (and subsidence of adjacent basins) occurs by movement along major faults on one or both sides of the blocks. It is generally believed that the faulting is distributed along several range-front faults, many of which are buried beneath the valley-fill deposits. Many of the mountain blocks show significant tilt; in the eastern Great Basin, most blocks are tilted to the east (Stewart, 1978).

Latest movement is known to be Quaternary or younger on many of the range front faults. Offset of Quaternary sediments or Holocene alluvial fans is well documented in numerous studies, particularly along the Wasatch fault. The Stansbury fault has been considered to be active at least since the work of Rigby (1958). More recent analyses suggest the fault may be segmented with movement on the southern segment occurring less than 18,000 years B.P. (latest Pleistocene) (Helm, 1995; Geomatrix Consultants, Inc., 1999a). Detailed discussion of the Stansbury fault and the seismic implications are found in Section 2.6.2.3, and Sections 5 and 6 of Geomatrix Consultants, Inc. (1999a).

Other Tertiary normal faults in Skull Valley have been proposed by various authors (Cook et al., 1989; Helm, 1995; Zoback, 1983). Recent work for the PFSF has identified two additional west-dipping normal faults in the vicinity of the PFSF, based mainly on geophysical data and subtle geomorphic expression (Bay Geophysical

Associates, 1999; Geomatrix Consultants, Inc., 1999a). These faults are informally named the "East" and "West" faults and are discussed in detail in Geomatrix Consultants, Inc. (1999a), Sections 2, 5, and 6. As shown on the cross sections, Figures 2-1 and 2-2 in Geomatrix Consultants, Inc. (1999a), the East fault is interpreted to form the east margin of the Tertiary basin that underlies Skull Valley whereas the West fault lies within the basin, west of the PFSF location. The East and West faults are interpreted to merge together about 9 miles southeast of the site. The PFSF appears to be located in the stepover zone between the East and West faults where the slip is transferred from the East to the West fault. The west boundary of the Tertiary Skull Valley basin is believed to be the East Cedar Mountains fault.

Interpretation of the high resolution seismic reflection survey (Bay Geophysical Associates, 1999) performed across the PFSF site indicates the East fault displaces a subsurface reflector believed to be the unconformity at the base of the Bonneville alloformation on the Promontory soil. The Bonneville sediments are 30,000 years or younger. Therefore, the East fault is considered to be capable as defined in 10CFR50 Appendix A. The West fault is also considered to be capable based on apparent changes in elevation along the late Pleistocene Stansbury gravel bar, southwest of the PFSF. The evidence for the fault and an analysis of its slip rate are discussed in Section 2 and 5 of Geomatrix Consultants, Inc. (1999a).

The zone of distributed faulting, where slip is transferred from the East fault to the West fault, was also interpreted from the high resolution reflection survey. Small normal faults, both east-and west-dipping, were imaged; some were interpreted to offset the base of the Bonneville alloformation whereas others clearly do not (Bay Geophysical Associates, 1999). Displacement on individual faults within the zone of distributed faulting is small (Geomatrix Consultants, Inc., 1999a, Table 5-1). A drilling program conducted across this

zone confirmed the presence and nature of this zone of faulting, as shown on Figure 5-4 in Geomatrix Consultants, Inc. (1999a).

2.6.1.2.3 Surficial (Basin-fill deposits)

Unconsolidated and semi-consolidated rocks of Tertiary and Quaternary age are believed to underlie Skull Valley. The total thickness is unknown but has been inferred to be 6,000 to 7,000 ft near the Cedar Mountains on the west side of the valley (Johnson and Cook, 1957) to 6,000 to 8,000 ft near I-80 (Baer and Benson, 1987). These materials are believed to include complex interlayered conglomerates, alluvium, lake deposits, and a few volcanic ash deposits, some of which must date from the beginning of Basin and Range faulting (12 to 13 m.y.a.) (Moore and McKee, 1983). The oldest of these deposits are included as the Salt Lake Group of Miocene-Pliocene age. This formation outcrops in the southern part of Rush Valley and is in the subsurface of Tooele Valley, east of the Stansbury Mountains (Everitt and Kaliser, 1980). The Salt Lake Group is not mapped at the surface on the Quaternary geologic map of Sack (1993) (Figure 2.6-4). Davis Knolls, about 5 miles southeast of Dugway, is identified on the State Geologic Map (Hintze, 1980) as part of the Salt Lake Formation. Moore and Sorensen (1979) mapped this area as early Tertiary conglomerate, tuffaceous sandstone, and fresh-water limestone. Several borings at the PFSF site encountered volcanic ash at depths between 65 and 90 ft. An ash at a depth of 85 ft was correlated with the Walcott tuff, known to be 6.6 million years old (Appendix 2E).

Everitt and Kaliser (1980) believe there is an unconformity at the top of the Salt Lake Group prior to deposition of the largely unconsolidated sand, silt, gravel, and clay of Quaternary age (<2 m.y.). This appears to be the case in Skull Valley as evidenced by the distinct reflector at a depth of about 85 ft in the high resolution shear wave reflection survey, reflector Q/T from Bay Geophysical Associates (1999) and Geomatrix

Consultants, Inc. (1999a). Fossils are rare from these deposits; as a result, the boundary between the Tertiary and Quaternary is somewhat arbitrarily drawn on the bases of degree of consolidation, relative amount of volcanic material, and degree of deformation evident.

The surficial geology of Skull Valley is predominantly unconsolidated material of Quaternary age deposited in Lake Bonneville (~ 30,000 to 12,000 years B.P.). Pre-Lake Bonneville lacustrine deposits have been found in other valleys in the region indicating numerous lakes occupied the Salt Lake basin prior to Lake Bonneville. These deposits date from at least 600,000 B.P. to 30,000 B.P. (Lund et al., 1990). Pre-Lake Bonneville sediments were encountered in borings, test pits, and trenches in the site vicinity, as discussed in Geomatrix Consultants, Inc. (1999a).

Gilbert (cited in Sack, 1993) believed that the extensive pre-Bonneville alluvial fans were an indication of a long period of hot, dry climate prior to the transgression of Lake Bonneville. Most investigators believe that the Bonneville lake cycle began between 30,000 and 25,000 years B.P., coinciding with the final glacial maximum in the Rocky Mountains (Scott, 1988; Oviatt et al., 1992). Lake levels continued to rise until about 21,000 to 20,000 years B.P. when the level remained somewhat stable for an extended period of time. The Stansbury shoreline developed at this time and has been identified throughout the Bonneville Basin (Oviatt et al., 1990), near elevation 4,468 ft. Sack (1993) has mapped the Stansbury shoreline through the southern part of Section 6, T5S-R8W near the PFSF, based on aerial photographs (Figure 2.6-4). Geomatrix Consultants, Inc. (1999a) mapped numerous additional Stansbury shoreline features in the area as shown on their Figure 1-3.

Continued filling of the basin after 20,000 years B.P. caused the lake to rise to its maximum elevation of about 5,240 ft approximately 15,300 years B.P. At that time, an

outlet for the lake into the Snake River drainage was reached. The Bonneville shoreline was created at this time and can be seen as a bench on the alluvial fan east of the PFSF. At about 14,500 years B.P., unconsolidated deposits in the lake outlet channel were rapidly eroded. The lake dropped more than 300 ft in a matter of a few weeks, and the Bonneville flood resulted. The outlet stabilized at about elevation 4,740 ft, and the Provo level developed (Malde, 1968). Sack (1993) and Geomatrix Consultants, Inc. (1999a) have also mapped this shoreline east of the PFSF site on the alluvial fan (Figure 2.6-4). This shoreline also was identified at an elevation of approximately 4830 ft on Hickman Knolls by Geomatrix Consultants (1999a).

Climatic change beginning about 14,000 years B.P. caused the gradual shrinkage of Lake Bonneville to at least the lowest level of present Great Salt Lake by about 12,000 years B.P. (Currey, 1990). A brief transgression of the lake occurred between about 10,900 and 10,300 years B.P. to about elevation 4,250 ft (Currey, 1990). This level is known as the Gilbert level of the Great Salt Lake and has been mapped about 11 miles north of the PFSF site (Sack, 1993). Since that time the lake has receded and fluctuates within about 20 ft elevation of its historic average (Lund et al., 1990). Only once in the past 10,000 years has the level of the lake been as high as 4,220 ft (Atwood and Mabey, 1995). The PFSF site is at approximate elevation 4,450 ft, well above any probable historic level of the Great Salt Lake.

2.6.1.3 Site Geology

The site geology was investigated in 1996 by a subsurface drilling program totaling 24 borings to a maximum depth of 100 ft and a seismic refraction and reflection program. Logs of these borings are included in Appendix 2A and the results of the seismic surveys are found in Appendix 2B.

Additional investigations were undertaken in 1998 and included surficial and bedrock mapping, excavation and mapping of numerous test pits and trenches, drilling of more than 40 additional boreholes to a maximum depth of 225 ft, and completion of 6 kilometers of high-resolution seismic shear-wave reflection lines. A summary of these efforts is included below. Additional detail and discussion are found in the original reports of this work (Geomatrix Consultants, Inc., 1999a; Bay Geophysical Associates, 1999).

The PFSF site is situated near the center of Skull Valley where Quaternary lacustrine and geomorphic features dominate the topography. The stratigraphy beneath the site consists of approximately 500 to 800 ft of Quaternary and Tertiary basin fill overlying Paleozoic bedrock. The nature of the deepest Tertiary deposits is unknown at this time but is believed to include sediments of the Salt Lake Formation, mainly sand, silt, marl and tuff in varying states of consolidation. The Salt Lake Formation extends up to a depth of about 85 ft in the central part of the PFSF. A volcanic ash at that level has been correlated with the Walcott tuff, know to be late Miocene in age (approximately 6 m.y.; Appendix 2E). This boundary was also identified as a prominent reflector on the high-resolution shear wave profiles (Bay Geophysical Associates, 1999; Geomatrix Consultants, Inc., 1999a, Plate 4).

There is evidence for four major lake cycles in the Bonneville basin during the past 700,000 years (Machette and Scott, 1988; Oviatt et al., 1997). Evidence for the three oldest is not well preserved regionally and was found only sporadically in the PFSF vicinity. The most recent cycle, the Lake Bonneville cycle, occurred between about 30,000 and 12,000 years ago and is well documented in Skull Valley. Several transgressions and recessions of the lake occurred during this time, each leaving an identifiable characteristic in the geomorpholoy of the valley or in the stratigraphic record. This evidence is presented in detail in Geomatrix Consultants, Inc. (1999a, Section 3.2). Near-surface Pleistocene deposits at the PFSF consist mainly of fine sand, silt, clay and

marl. In general, the finer grained materials, such as silt, clay and marl were deposited during the deeper water portions of the lake cycle and the sand represents shallower, near-shore beach or deltaic fan environments. The engineering properties of those materials are discussed in Section 2.6.1.6. Locally, Holocene eolian and fluvial activities have reworked the surface soils to some extent (Sack, 1993). Eastward from the PFSF, along the proposed access road to Skull Valley Road, the influence of the proximity to the range-front alluvial fans is apparent as an increase in gravel content at shallow depths (Appendix 2A).

Bedrock is not exposed at the PFSF site but is found about 1.5 mile to the south at Hickman Knolls, and about 1.5 mile northeast in a series of low hills informally called Castle Rock Knoll. Hickman Knolls is inferred to consist of Fish Haven Dolomite of Ordovician age (Moore and Sorensen, 1979; Geomatrix Consultants, Inc.,1999a). At this location the formation is a medium to dark gray dolomite and limestone breccia. Bedding is massive to indistinct and breccia pebbles are angular to sub-round and appear to be the same composition as the enclosing matrix. Bedding strikes northerly to northeasterly and dips to the east at moderate to steep angles. Bedrock fracturing consists mainly of two sets of high angle fractures, one trends east-west and the other north-south. These fractures tend to coincide with more silicified zones that form prominent scarps on the Knolls that are strongly expressed in the morphology and are associated with many of the aerial-photo lineaments (See Plate 1, Geomatrix Consultants, Inc., 1999a).

Several faults and ductile shear zones were identified at Hickman Knolls during the recent investigations. Geomatrix Consultants, Inc. (1999a) presents evidence that indicates the faults developed prior to the dolomitization process and the shear zones are likely penecontemporaneous with the process of brecciation. No large, through-going faults are believed to exist on Hickman Knolls.

There has been some enlargement of a few joints resulting from dissolution and a few small caves or openings (1 to 4 ft deep) can be seen on some of the steeper rock faces. Karst conditions do not exist at Hickman Knolls nor are they likely to develop because of the near-desert environment and the depth to groundwater (greater than 100 ft). The outcrop mapped northeast of the PFSF site (Castle Rock Knoll) has been identified as Deseret Limestone of Mississippian age (Moore and Sorensen, 1979).

Faults in the PFSF vicinity important to the understanding of site geology include the East, West, "F", and smaller faults in the "zone of distributed faulting." These faults are discussed briefly in Section 2.6.1.2.2 and in detail in Geomatrix Consultants, Inc. (1999a), Sections 2, 5, and 6.

2.6.1.4 Geologic Map of Site Area

Figure 2.6-4 is the geologic map of the PFSF site, reproduced from Sack (1993). Areas of bedrock outcrop are indicated in addition to the surficial deposits. Figure 1-3 in Geomatrix Consultants, Inc. (1999a) is a photo-geologic map of the site vicinity showing additional details and interpretations of geologic features. Scarps in soil near the site identified on Sack's map have been investigated by Dr. Donald Currey for this project (Appendix 2C). Both Currey and Geomatrix Consultants, Inc. (1999a) concluded the features were related to lacustrine processes of Lake Bonneville and are not of tectonic origin.

2.6.1.5 Facility Plot Plan and Geologic Investigations

Figure 2.6-2 is a plot plan showing the locations of the major structures of the PFSF, the locations of the 1996 geotechnical borings and geophysical survey lines, and the location of Foundation Profile A-A'. Plate 1 of Geomatrix Consultants, Inc. (1999a), indicates the locations of both the 1996 and 1998 investigations exclusive of the geotechnical borings for the Canister Transfer Building. Results of the borings, laboratory tests, and geophysical surveys are found in Appendices 2A and 2B,-Geomatrix Consultants, Inc. (1999a), and Bay Geophysical Associates (1999).

2.6.1.6 Relationship of Major Foundations to Subsurface Materials

Figure 2.6-5 presents Foundation Profile A-A', which shows the locations of the proposed structures in relationship to the subsurface materials encountered in the borings. As indicated, the generalized subsurface profile consists of three layers. The uppermost layer extends to a depth of between 25 and 35 ft below existing grade and is mainly interlayered silt, silty clay, and clayey silt. Standard Penetration Test (SPT) N-values for this layer are mostly between 8 and 20 blows per ft, with an average value of 16 blows per ft and a median value of 14 blows per ft, indicating that these are "stiff" or "medium dense" materials. The proposed structures will be constructed on strip and spread footings and the casks will be placed on mat foundations founded in this layer.

A distinct change in material occurs at about 25 to 35 ft, where refusal (N>100 blows per 6 inches) conditions are often encountered. The following 25 to 30 ft consists of very dense, dry, fine sand. Thin layers of fine gravel and coarse sand also are evident. A few clayey zones were encountered, but they had no apparent effect on the blow counts. The borings that were drilled to a depth of 100 ft or more (Borings A-1, D-4, CTB-5, and CTB-8) indicate that this layer is mainly underlain by very dense silt, silty

sand, and sandy silt with occasional layers of clayey silt. Several layers of volcanic ash were also encountered in these borings.

Seismic refraction results (Appendix 2B) in the vicinity of the Storage Facility (see Figure 2.6-2, Seismic Lines 1 & 2) indicate the compression wave (P-wave) velocity changes from approximately 2,780 ft/sec to approximately 5,525 ft/sec at depths of between 90 and 131 ft below grade, which may represent the water table. A groundwater monitoring well was installed in 1998 in the vicinity of the CTB. Initial readings from this well indicate the groundwater table is about 125 ft below ground surface, approximate elevation 4,350 ft.

As shown in Figure 2.6-2, Borings AR-1 through AR-5 were drilled along the proposed corridor for the access road, which extends easterly from the area in the vicinity of the proposed Operations & Maintenance (O&M) Building and the Administration Building to Skull Valley Road. These borings indicate that the near-surface soils are similar to the uppermost layer described above; i.e., silt, silty clay, and clayey silt. Sands were encountered at depths of 5 and 10 ft in Boring AR-1 and from a depth of 5 ft to 20 ft in Boring AR-2. These sands are likely the subsurface continuation of the surficial beach ridges identified by Currey (Appendix 2C) and shown on Figure 1-3 of Geomatrix Consultants, Inc. (1999a). Silty or sandy gravels were encountered at a depth of 30 ft in Boring AR-3, 20 ft in Boring AR-4, and 6 ft in Boring AR-5.

These borings did not encounter bedrock. Interpretation of the seismic reflection survey data indicates that the depth to bedrock is between 520 ft and 820 ft below the surface at the site in the vicinity of the storage pads and that it drops off towards the east, dipping from an estimated depth of 740 ft at Station 700 on Seismic Line 3 (shown on Figure 2.6-2) to approximately 1020 ft at the eastern end of this seismic line. This is consistent with the interpretation that the Tertiary half-graben basin beneath Skull

Valley is tilted down to the east along the East fault (Geomatrix Consultants, Inc., 1999a).

Refer to Sections 2.6.1.11 and 2.6.2.1 for a discussion of the engineering characteristics of these soils.

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 to minimize potential problems in the future.

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, Method D. 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, Method D. 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 the monitoring well (CTB-5) at a depth of 125 ft (elevation 4,350) in the vicinity of the CTB. 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). 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 and are summarized in Table 2.6-1. 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).

2.6.1.11 Static and Dynamic Soil and Rock Properties at the Site

Geotechnical laboratory tests were performed on samples obtained in the borings of the upper layer of silt, silty clay, and clayey silt that is shown in Figure 2.6-5. The results of these tests indicate:

- Water content: 28% $< \omega <$ 47%, ω_{avg} 36%,
- Liquid Limit: 29% $< LL <$ 61%, LL_{avg} 42%

- Plastic Limit: 20% < PL < 44%, PL_{avg} 29%
- Plasticity Index: 5% < PI < 23%, PI_{avg} 13%
- Specific gravity: 2.72
- Saturation: 51%
- Initial void ratio: 1.9
- Unit weight: Dry 59 pcf
  Moist 80 pcf
  Saturated 100 to 105 pcf
- Consolidation parameters:
  Maximum past pressure: 6 ksf
  Virgin compression ratio, CR: 0.294
  Recompression ratio, RR: 0.014
  Rate of secondary compression, as shown by the dashed curve in Figure 2.6-6.

Effective-stress strength parameters for drained analyses are estimated to be $\phi = 30°$ and c = 0, based on the plasticity index of this material.

Total-stress strength parameters for undrained analyses (e.g., earthquake loadings) are estimated to be $\phi = 0°$ and c = 2.2 ksf, based on unconsolidated-undrained triaxial tests, and Poisson's ratio = 0.433.

The recommended coefficients of earth pressure for this material are as follows:

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

The recommended coefficient of friction between concrete placed on the in situ soils is 0.58 for long-term loadings, and a cohesion of 2.2 ksf (i.e., the undrained shear strength) should be used to resist sliding for short-term (e.g., earthquake) loadings.

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 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:

$$RF = [(B+1) / 2B]^2$$

where B is the effective width of the footing.

The recommended value of the coefficient of vertical subgrade reaction of the in situ soils for use in design of the storage pads is 20 kips/ft^3.

The recommended value of the coefficient of horizontal subgrade reaction of the in situ soils for use in the design of drilled caissons is 20 • z / B kips/ft^3, where z is the depth below finished grade and B is the effective width of the caisson.

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 shall 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 Motions, 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 Motions.

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 Bearing Capacity and Settlement Analyses—Cask Storage Pads

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 4 ksf. However, loading the storage pads to this value may result in undesirable settlements.

Analyses were performed to estimate the settlement of the 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). The actual bearing pressure for this case was about 1.9 ksf, and the estimated total settlement of the pad is 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 crushed rock surface material will be feathered to meet the edges of the raised pads for transporter access.

The allowable bearing pressure for the storage pads for dynamic loads from the design basis ground motion was determined based on the assumption that the horizontal inertia of the casks due to the earthquake may exceed the frictional resistance available between the cask and the top of the pad. If this were to occur, the horizontal force that would be imparted to the pad from the casks would equal $\mu \cdot F_v$, where:

μ = the coefficient of friction between the steel bottom of the cask and the top of the concrete storage pad.

F_v = weight of the casks + the vertical inertia force of the casks as result of the design basis ground motion.

These analyses were performed for various values of μ and the following values of gross allowable bearing pressures were determined:

Coefficient of friction (μ)	Gross Allowable ($q_{allowable}$, ksf)
0.00	8.98
0.10	8.07
0.20	7.23
0.30	6.47
0.40	5.79
0.50	5.18
0.60	4.64
≥0.67	4.30

It is expected that the minimum coefficient of friction between the cask and the storage pad (steel to concrete) will be 0.3.

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

2.6.1.12.2 Allowable Bearing Capacity—Other Structures

Other structures at the PFSF shall be founded on strip and spread footings. These include the Canister Transfer Building, Administration Building, Operating and Maintenance Building, and Security and Health Physics Building. 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 three. Bearing capacity analyses were also performed using total-stress strength parameters, which are applicable for earthquake loads. The former analyses yielded the minimum allowable bearing pressures.

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 0.40 g for horizontal ground motion and 0.39 g for the vertical ground motion were determined as the design bases of the PFSF for a 1000-yr return period (Geomatrix Consultants, Inc, 1999b).

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

The dynamic foundation parameters in support of the soil-structure interaction analyses were derived from the results of a one-dimensional site response analysis. Figures 2.6-13 and 2.6-14 present the strain-compatible shear-wave velocity and damping ratio profiles.

Strain-compatible soil properties of the upper layer of silt, silty clay, and clayey silt, developed based on the weighted average of the values within 30 ft below the foundation, include:

- Shear-wave velocity 515 ft/sec
- Shear-wave damping 11%
- Compressional-wave velocity 1500 ft/sec

- Shear modulus 668 ksf
- Young's modulus 1915 ksf
- Poisson's ratio 0.433

Table 2.6-2 presents the equivalent dynamic soil parameters for the storage pad, developed based on Newmark and Rosenblueth (1971). Table 2.6-3 presents dynamic soil parameters for the design of the concrete storage pads.

Refer to Section 2.6.1.6 for discussion of the static engineering properties of the materials 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.

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 1 x 10^{-3} per year or a 1000-year return period (PFS letter April 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 ($\geq$ magnitude 5). Seismic sources modeled in the probabilistic seismic hazard analysis (PSHA) 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 1 x 10^{-3} per year or a 1000-yr return period. At these probability levels, the displacements associated with faulting on Faults C, D, and F were determined to be less than 0.1cm (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.

The stability of soils at the site for foundations is discussed in Sections 2.6.4.4 and 2.6.4.8.

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 2.4.1.2, 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 a cohesion of greater than 2000 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.

This material is underlain by very dense, fine sands, which 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 100 ft below grade, and the P-wave velocities (5,100 ft/sec to 5,900 ft/sec), reported by Geosphere Midwest, Inc. (Appendix 2B), indicate that these soils are also very dense. Further, these

soils are too far removed from the surface to cause problems if they were to experience dynamic settlement.

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 essentially dry from grade down to a depth greater than 100 ft. Figure 2.6-5 presents a generalized subsurface profile, which was developed based on the borings that were drilled in late 1996. As indicated in Section 2.4.1.2, the groundwater table is about 125 ft below grade at the site.

Figure 2.6-5 illustrates that from a depth of about 30 ft down to 100 ft (the depth of the deepest boring) the soils are very dense, as 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 60-ft-thick, very dense layer is expected to preclude any surface manifestation of liquefaction (e.g., sand boils) of soils below the groundwater table, if it were possible for them to liquefy. This is considered unlikely, however, because the density of the soils encountered in the borings increases with depth, and the P-wave velocities (5,100 ft/sec to 5,900 ft/sec), reported by Geosphere Midwest, Inc. (Appendix 2B), of the soils below the groundwater table indicate that these soils are also very dense.

2.6.4.9 Design Basis Ground Motion

Design basis ground motions were determined by a probabilistic seismic hazard analysis and are defined as having a peak horizontal ground acceleration of 0.40 g and a peak vertical ground acceleration of 0.39 g. The development of the design basis ground motions is described in Geomatrix Consultants, Inc. (1999a and 1999b). The

site specific response spectra are presented in Figure 4 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

The subsurface conditions at the PFSF site are suitable for support of the proposed structures; therefore, no special construction techniques are required for improving the subsurface conditions.

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.

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.

2.7 SUMMARY OF SITE CONDITIONS AFFECTING CONSTRUCTION AND OPERATING REQUIREMENTS

The PFSF site is located near the middle of Skull Valley. The valley is approximately 50 miles long, 22 miles wide, and slopes gently to the north at about a 0.6% grade. The finished grade of the storage facility is sloped gently from elevation 4,475 ft at the southeastern corner to elevation 4,457 ft at the north, where a stormwater detention basin is located. The stormwater detention basin is at elevation 4,450 ft. The Canister Transfer Building, located within the restricted area, has a floor elevation of 4,475 ft. The access road is graded to match the Skull Valley road at the east end (elevation 4,487 ft) and the storage facility at the west end (elevation 4,475 ft) with a maximum slope of approximately 3%.

The entire site, including the access road, is above the 100-year flood elevation. However, diversion berms are required to deflect the anticipated flows from the PMF due to both the Stansbury Mountains and the Hickman Knolls watersheds.

A diversion berm is provided perpendicular to the access road to channel the Stansbury Mountain PMF to flow away from the PFSF. The PMF berm elevation is 4,505 ft where it intersects the access road. The berm is a maximum of 9 ft high at the access road and tapers down in height as it joins the Hickman Knolls to the south. The berm is located approximately 200 ft east of the storage facility along the access road and extends approximately 1,600 ft south and 280 ft north of the access road. The berm will divert PMF-generated flow emanating from the Stansbury Mountains, east of the site, to the north and east of the storage facility. Box culverts are provided under the access road to allow normal and 100-yr. floodwater to pass beneath the access road toward the north. PMF flows will flow over the top of the access road.

The storage facility and buildings within the restricted area are protected from the smaller Hickman Knolls PMF by a partial perimeter berm. The berm is located along the entire southern side and halfway along the western side of the storage facility. The berm is approximately 5 ft high and is designed to divert the PMF sheet flow of less than one foot deep to the west and north of the storage facility.

A probabilistic seismic hazard analysis was performed to determine the design basis ground motions at the PFSF. The peak horizontal ground acceleration was determined to be 0.40 g and the peak vertical ground acceleration is 0.39 g at an appropriate 1000-yr return period.

Subsurface soils at the site are suitable for supporting conventional foundations under both the static and dynamic loading conditions. There is no potential for liquefaction, collapse, or excessive settlement of these soils. There are no slopes, natural or manmade, close enough to the proposed important to safety facilities that their failure could adversely affect these facilities.

Dry casks will be used to store canisters containing spent fuel. The canisters will be drained of all liquid prior to being shipped to the facility. Therefore, liquid releases cannot result from operation of the facility.

Groundwater is about 125 ft deep at the site. The method of storage (dry cask), the nature of the storage casks, and the depth to groundwater beneath the site preclude the possibility of groundwater contamination from operation of the facility.

2.8 REFERENCES

American National Standards Institute, 1982, American national standard minimum design loads for buildings and other structures: ANSI A58.1-1982, published by the American National Standards Institute, Inc., New York, New York.

Anderson, J.G., Wesnousky, S.G., and Stirling, M.W., 1996, Earthquake size as a function of fault slip rate: Bulletin of the Seismological Society of America, v. 86, No. 3, p. 683-690.

Anderson, R.E., 1989, Tectonic evolution of the Intermontane system; Basin and Range, Colorado Plateau, and High Lava Plains, in Pakiser, L.C., and Mooney, W.D., eds., Geophysical framework of the continental United States: Geological Society of America Memoir 172, pp. 163-176.

Arabasz, W.J., Pechmann, J.C., and Brown, E.D., 1987, Evaluation of seismicity relevant to the proposed siting of a Superconducting Supercollider (SSC) in Tooele County, Utah: Technical report for the Dames and Moore Utah SSC Proposal Team, June 1987, 107 pp.

Arabasz, W.J., Pechmann, J.C., and Brown, E.D., 1992, Observational seismology and the evaluation of earthquake hazards and risk in the Wasatch Front area, Utah, in Gori, P.L., and Hays, W.W., eds., Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah: U.S. Geological Survey Professional Paper 1500-A-J, pp. D1-D36.

Arabasz, W.J., Smith, R.B., and Richins, W.D., 1980, Earthquake studies along the Wasatch Front, Utah: Network monitoring, seismicity, and seismic hazards: Bulletin of Seismological Society of America, vol. 70, pp. 1479-1499.

Ashcroft, G.L., D.T. Jensen and J. L. Brown, 1992, Utah climate: Logan, UT, Utah Climate Center, Utah State University, 127 p.

Atwood, G., and Mabey, D.R., 1995, Flooding hazards associated with Great Salt Lake, in Lund, W.R., ed., Environmental and engineering geology of the Wasatch Front Region: Utah Geological Assoc. Pub. 24, pp. 483-493.

Baer, J.L. and Bensen, A.K., 1987, Results of gravity survey, Skull Valley - Ripple Valley, Tooele County, Utah, in Dames and Moore, The Ralph M. Parsons Company, and Roger Foott Associates, Inc. (preparers), Site Proposal for the Superconducting Super Collider, Geotechnical Report, v. 2, pages E1-E8.

Barnhard, T.P. and Dodge, R.L., 1988, Map of fault scarps formed on unconsolidated sediments, Tooele 1° x 2° quadrangle, northwestern Utah: U.S. Geological Survey Miscellaneous Field Studies Map MF-1990, scale 1:250,000.

Bay Geophysical Associates, Inc., 1999, High-resolution seismic shear-wave reflection profiling for the identification of faults at the Private Fuel Storage Facility, Skull Valley, Utah-final report, prepared for Stone and Webster Engineering Corp., Denver, CO, 16 pp.

BLM, 1985, Skull Valley Allotment Management Plan. Salt Lake District, BLM, US Department of the Interior, Salt Lake City, UT. August, 1985.

BLM, 1986, South Skull Valley Allotment Management Plan. Salt Lake District, BLM, US Department of the Interior, Salt Lake City, UT. January, 1986.

BLM, 1988, Bureau of Land Management, Draft Pony Express Resource Management Plan and Environmental Impact Statement. Salt Lake District, BLM, US Department of the Interior, Salt Lake City, UT. May 1988.

BLM, 1992, Horseshoe Springs Habitat Management Plan. UT-020-WHA-T-7. Salt Lake District, BLM, US Department of the Interior, Salt Lake City, UT. February 26, 1992

Christie-Blick, N., 1983, Structural geology of the southern Sheeprock Mountains, Utah: regional significance, in Miller, D.M., Todd, V.R., and Howard, K.A., editors, Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 101-124.

Coffman, J.L. and von Hake, C.A., 1973, Earthquake history of the United States, revised edition (through 1970): U.S. Dept. of Commerce - NOAA Publication 41-1, 208 pp.

Cook, K.L., Bankey, V., Mabey, D.R., and DePangher, M., 1989, Complete Bougher gravity anomaly map of Utah: Utah Geological and Mineral Survey Map 122, scale 1:500,000.

Currey, D.R., 1990, Quaternary paleolakes in the evolution of semi-desert basins, with special emphasis on Lake Bonneville and the Great Basin, U.S.A.: Paleogeography, Paleoclimatology, and Paleoecology, vol. 76, pp. 189-214.

dePolo, C.M., 1994, The maximum background earthquake for the Basin and Range Province, western North America: Bulletin of the Seismological Society of America, v. 84, pp. 466-472.

dePolo, C.M., Clark, D.G., Slemmons, D.B., and Aymard, W.G., 1989, Historical Basin and Range Province surface faulting and fault segmentation, in Schwartz, D.P., and Sibson, R.H., editors, Fault segmentation and controls of rupture initiation and termination--proceedings of conference XLV: U.S. Geological Survey Open-file Report 89-315, pp. 131-162.

Federal Highway Administration (FHWA), 1977, Runoff Estimate for Small Rural Watersheds and Development of a Sound Design Method, 248 pp.

Everitt, B.L. and Kaliser, B.N., 1980, Geology for assessment of seismic risk in the Tooele and Rush Valleys, Tooele County, Utah: Utah Geological and Mineral Survey Special Study 51, 33 pp.

Geomatrix Consultants, Inc, 1997, PFSF Calculation 05996.01-G(PO5)-1, "Development of Soil and Foundation Parameters in Support of Dynamic Soil-Structure Interaction Analyses," San Francisco, CA, March (73 pp).

Geomatrix Consultants, Inc., 1999a, Fault evaluation study and seismic hazard assessment study-final report, prepared for Stone and Webster Engineering Corp., Denver, CO, 3 volumes.

Geomatrix Consultants, Inc., 1999b, Development of design ground motions for the Private Fuel Storage Facility, prepared for Stone and Webster Engineering Corp., Denver, CO, 19 pp.

Gilbert, G.K., 1890, Lake Bonneville, U.S. Geological Survey Monograph 1, 428 pp.

Goter, S.K., compiler, 1990, Earthquakes in Utah, 1884-1989: U.S. Geological Survey, National Earthquake Information Center, scale 1:500,000.

Grazulis, Thomas P., 1993, Significant tornadoes 1680 - 1991: Published by The Tornado Project of Environmental Films, St. Johnsbury, Vermont.

Hecker, Suzanne, 1993, Quaternary tectonics of Utah with emphasis on earthquake-hazard characterization: Utah Geological Survey Bulletin 127, Salt Lake City, UT, 156 pp.

Helm, J.M., 1994, Structure and tectonic geomorphology of the Stansbury fault zone, Tooele County, Utah, and the effect of crustal structure on Cenozoic faulting patterns, M.S. thesis, Univ. of Utah, Salt Lake City, Utah, 128 pp.

Helm, J.M., 1995, Quaternary faulting in the Stansbury fault zone, Tooele County, Utah, in Lund, W.R., editor, Environmental and engineering geology of the Wasatch Front Region: Utah Geological Association Publication 24, pp. 31-44.

Hintze, L.H. (compiler), 1980, Geologic Map of Utah, Utah Geological and Mineral Survey, Salt Lake City, UT, scale 1:500,000.

Holzworth, G.C., 1972, Mixing heights, wind speeds, and potential for urban air pollution throughout the contiguous United States: Environmental Protection Agency, Office of Air Programs, Research Triangle Park, North Carolina.

Holzworth, G.C., 1974, Meteorological episodes of slowest dilution in contiguous United States: National Environmental Research Center, Research Triangle Park, North Carolina, Report No. EPA-650/4-74-002.

Hood,J.W., and Waddell,K.M., 1968, Hydrologic reconnaissance of Skull Valley, Tooele County, Utah, DNR Tech Pub. No. 18, 57 pp.

Hosler, Charles R., 1961, Low-level inversion frequency in the contiguous United States: Monthly Weather Review, pp. 319-339.

Howard, K.A. editors, Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 61-73.

Huschke, R. E., ed., 1959, Glossary of meteorology: Published by the American Meteorological Society, Boston, Massachusetts.

Johnson, J.B., and Cook, K.L., 1957, Regional gravity survey of parts of Tooele, Juab, and Millard Counties, Utah: Geophysics, vol. 22, pp. 48-61.

Lambe, T.W., and R.V. Whitman, 1969, Soil Mechanics, John Wiley & Sons, Inc., N.Y., 553 pp.

Lund, W.R., Christenson, G.E., Harty, K.M., Hecker, S., Atwood, G., Case, W.F., Gill, H.E., Gwynn, J.W., Klauk, R.H., Mabey, D.R., Mulvey, W.E., Sprinkel, D.A., Tripp, B.T., Black, W.D., and Nelson, C.V., 1990, Geology of Salt Lake City, Utah, U.S.A.: Assoc. of Engineering Geologists Bull., vol. XXVII, pp. 391-478.

Machette, M.N., and Scott, W.E., 1988, Field trip introduction-A brief review of research on lake cycles and neotectonics of the eastern Basin and Range province: Utah Geological and Mineral Survey Misc. Publ. 88-1, p. 7-14.

Malde, H.E., 1968, The catastrophic late Pleistocene Bonneville flood in the Snake River plain, Idaho: U.S. Geological Survey Professional Paper 596, 52 pp.

Moore, W.J., and McKee, E.H., 1983, Phanerozoic magmatism and mineralization in the Tooele 1° x 2° quadrangle, Utah, in Miller, D.M., Todd, V.R., and Howard, K.A., eds., Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 183-190.

Moore, W.J., and Sorensen, M.L., 1979, Geologic map of the Tooele 1° x 2° quadrangle: U.S. Geological Survey Miscellaneous Investigations Series Map I-1132, scale 1:250,000.

National Oceanic and Atmospheric Administration, National Environmental Satellite, Data, and Information Service, National Climatic Data Center, 1960, Climatography of the United States No. 60, Climate of Utah.

National Oceanic and Atmospheric Administration, National Environmental Satellite, Data, and Information Service, National Climatic Data Center, 1992, Local climatological data, annual summary with comparative data for 1991: Salt Lake City, Utah.

National Oceanic and Atmospheric Administration, National Environmental Satellite, Data, and Information Service, National Climatic Data Center, 1975-1995, Storm data and unusual weather phenomena with late reports and corrections.

Newmark, N. M., and Rosenblueth, E., 1971, Fundamentals of Earthquake Engineering, Prentice-Hall, Englewood Cliffs, NJ.

Oaks, S.D., 1987, Effects of six damaging earthquakes in Salt Lake City, Utah, in Gori. P.L., and Hays, W.W., editors, Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah: U.S. Geological Survey Open-file Report 87-585, vol. 2, pp. P-1-95.

Oviatt, C.G., Currey, D.R., and Miller, D.M., 1990, Age and paleoclimatic significance of the Stansbury shoreline of Lake Bonneville, northwestern Great Basin: Quaternary Research, vol. 33, pp. 291-305.

Pacific Gas and Electric Company, 1988, Final Report of the Diablo Canyon Long Term Seismic Program, Docket Nos. 50-275 and 50-323, July 31, 1988.

Pasquill, F., 1961, The estimation of the dispersion of windborne material: Meteorol. Mag., 90, 1063, 33-49.

Pechmann, J.C. and Arabasz, W.J., 1995, The problem of the random earthquake in seismic hazard analysis: Wasatch Front region, Utah, in Lund, W.R., editor, Environmental and engineering geology of the Wasatch Front region: Utah Geological Association Publication 24, pp. 77-93.

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

Quintana, 1995. Letter from Danny Quintana & Associates to Scott Northard, dated March 9, 1995. Subject: Private Spent Fuel Storage Facility on the Skull Valley Goshute Reservation.

Ramsdell, J. V. and G. L. Andrews, 1986, Tornado climatology of the contiguous United States: Prepared by Pacific Northwest Laboratory for the U.S. Nuclear Regulatory Commission, NUREG/CR-4461, PNL-5697.

Rigby, J.K., 1958, Geology of the Stansbury Mountains, Tooele County, Utah: Utah Geological Society Guidebook 13, 168 pp.

Roberts, R.J., Crittenden, M.D., Jr., Tooker, E.W., Morris, H.T., Hose, R.K., and Cheney, T.M., 1965, Pennsylvanian and Permian basins in northwestern Utah, northeastern Nevada and south-central Idaho: Amer. Assoc. Petrol. Geologists Bulletin, vol. 49, pp. 1926-1956.

Sack, Dorothy, 1993, Quaternary geologic map of Skull Valley, Tooele County, Utah: Utah Geological Survey Map 150, Scale 1:100,000, 16 p.

Sbar, M.L., and Barazangi, M., 1970, Tectonics of the intermountain seismic belt, western United States, Part I, microearthquake seismicity and composite fault plane solutions: Geological Society of America Abst. with Programs, vol. 2, p. 675.

Scott, W.E., 1988, Temporal relations of lacustrine and glacial events at Little Cottonwood Canyon and Bells Canyon, Utah, in Machette, M.N. and Currey, D.E., editors: In the footsteps of G.K. Gilbert - Lake Bonneville and neotectonics of the eastern Basin and Range Province, guidebook for field trip twelve, Utah Geological and Mineral Survey Misc. Publ. 88-1, pp. 78-82.

Silver, M. and Seed, H. B., 1971, "Volume Changes in Sands During Cyclic Loading," Proceedings of the American Society of Civil Engineers, Journal of the Soil Mechanics and Foundations Division, Vol 97, SM9, September.

Simiu, E., M. J. Changery, and J. J. Filliben, 1979, Extreme wind speeds at 129 stations in the contiguous United States, NBS building science series 118: U.S. Department of Commerce, National Bureau of Standards.

Slemmons, D.B., 1980, Design earthquake magnitudes for the western Great Basin, in Proc. of Conference X, Earthquake hazards along the Wasatch-Sierra Nevada frontal fault zones: U.S. Geological Survey Open-file Report 80-801, pp. 62-85.

Smith, R.B., 1978, Seismicity, crustal structure, and intraplate tectonics of the interior of the western Cordillera, in Smith, R.B., and Eaton, G.P., editors, Cenozoic tectonics and regional geophysics of the western Cordillera: Geological Society of America Memoir 152, pp. 111-144.

Smith, R.B., and Arabasz, W.J., 1991, Seismicity of the intermountain seismic belt, in Slemmons, D.B., Engdahl, E.R., Zoback, M.D., and Blackwell, D.C., eds., Neotectonics of North America: Geological Society of America, Decade Map Volume 1, pp. 185-228.

Smith, R.B., and Sbar, M.L., 1970, Seismicity and tectonics of the intermountain seismic belt, western United States, Part II, Focal mechanism of major earthquakes: Geological Society of America Abst. with Programs, vol. 2, p. 657.

Smith, R.B., and Sbar, M.L., 1974, Contemporary tectonics and seismicity of the western United States with emphasis on the intermountain seismic belt: Geological Society of America Bulletin, vol. 85, pp. 1205-1218.

Smith, R.B., Nagy, W.C., Julander, D.R., Viveiros, J.J., Baker, C.A., and Gants, D.G., 1989, Geophysical and tectonic framework of the eastern Basin and Range-Colorado Plateau-Rocky Mountain transition, in Pakiser, L.C., and Mooney, W.D., eds., Geophysical framework of the continental United States: Geological Society of America Memoir 172, pp. 205-233.

Stewart, J.H., 1976, Late Precambrian evolution of North America: plate tectonic implication: Geology, vol. 4, pp. 11-15.

Stewart, J.H., 1978, Basin-range structure in western North America: A review, in Smith, R.B. and Eaton, G.P., editors, Cenozoic tectonics and regional geophysics of the western Cordillera: Geological Society of America Memoir 152, pp. 1-31.

Stickney, M.C., and Bartholomew, M.J., 1987, Seismicity and late Quaternary faulting of the northern Basin and Range province, Montana and Idaho: Seismological Society of America Bulletin, vol. 77, pp. 1602-1625.

Stokes, W.L., 1986, Geology of Utah, Utah Museum of Natural History and Utah Geological and Mineral Survey, Salt Lake City, UT, 280 pp.

Stone & Webster Engineering Corporation (SWEC), 1999a, Calc. No. 05996.02-G(B)-12, Rev. 1, Flood Analysis with a Larger Drainage Basin.

Stone & Webster Engineering Corporation (SWEC), 1999b, Calc. No. 05996.02-G(B)-15, Rev. 0, Determination of Aquifer Permeability from Constant Head Test and Estimation of Radius of Influence for the Proposed Water Well.

Stone & Webster Engineering Corporation (SWEC), 1999c, Calc. No. 05996.02-G(B)-16, Rev. 1, Flood Analysis at 3-mile Long Portion of Rail Spur.

Stone & Webster Engineering Corporation (SWEC), 1999d, Calc. No. 05996.02-G(B)-17, Rev. 1, PMF Flood Analysis with Proposed Access Road and Rail Road.

Stover, C.W. and Coffman, J.L., 1993, Seismicity of the United States, 1568-1989 (Revised): U.S. Geological Survey Professional Paper 1527, 418 pp.

Stover, C.W., Reagor, B.G., and Algermissen, S.T., 1986, Seismicity map of the State of Utah, U.S. Geological Survey Miscellaneous Field Studies Map MF-1856, scale 1:1,000,000.

Thom, H. C. S., 1963, Tornado probabilities: Monthly Weather Review 91, pp. 730-736.

Tooele County Commission, 1995, Brochure entitled "Tooele County, Utah, Where Land And Sky Embrace."

Tooele, 1995. Tooele County General Plan, November 1995.

Tooker, E.W., 1983, Variations in structural style and correlation of thrust plates in the Sevier foreland thrust belt, Great Salt Lake area, Utah, in Miller, D.M., Todd, V.R., and Howard, K.A. editors, Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 61-73.

Tooker, E.W., and Roberts, R.J., 1971, Structures related to thrust faults in the Stansbury Mountains, Utah: U.S. Geological Survey Professional Paper 750-B, pp. B1-B12.

U.S. Army Corps of Engineers, Office of the Chief Engineers, 1990, Hydrologic Engineering Center, Flood Hydrograph Package, HEC-1, 283 pp.

U.S. Army Corps of Engineers, 1952, Standard Project Flood Determinations, Civil Engineer Bulletin, No. 52-8, Washington, D.C., 19 pp.

U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1997, River analysis system, HEC-RAS, Davis, CA.

U.S. Army Corps of Engineers, Office of the Chief of Engineers, 1990, Flood hydrograph package, HEC-1, Hydrologic Engineering Center, 283 pp.

U.S. Department of Commerce, National Oceanic and Atmospheric Administration, 1977, Probable maximum precipitation estimates, Colorado River and Great Basin drainage, Hydrometeorological Report No. 49 (HMR 49), 161 pp.

U.S. Geological Survey, 1994, Methods for estimating magnitude and frequency of floods in the southwestern United States, Open-File Report 93-419, 211 pp.

USDA, undated, Soil survey of Tooele County, Utah, unpublished maps and data, National Resources Conservation Service, Tooele, UT.

U.S. Nuclear Regulatory Commission, 1991, Safety Evaluation Report related to the operation of Diablo Canyon Nuclear Power Plant Units 1 and 2, NUREG-0675, Supplement No. 34, June 1991.

U.S. Weather Bureau, 1947, Thunderstorm rainfall. Hydrometeorological Report No. 5, Department of Commerce, Washington, D.C., 330 pp.

Wells, D.L., and Coppersmith, K.J., 1994, Analysis of empirical relationships among magnitude, rupture length, rupture area, and surface displacement: Seismological Society of America Bulletin, vol. 84, pp. 974-1002.

Wong, I., Olig, S., Green, R., Moriwaki, Y., Abrahamson, N., Baures, D., Silva, W., Somerville, P., Davidson, D., Pilz, J., and Dunne, B., 1995, Seismic hazard analysis of the Magna tailings impoundment, in Lund, W.R., ed., Environmental and engineering geology of the Wasatch Front Region: 1995 Symposium and Field Conference, Utah Geological Association Publication 24, pp. 95-110.

Youngs, R.R., Swan, F.H., III, Power, M.S., Schwartz, D.P., and Green, R.K., 1987, Probabilistic analysis of earthquake ground shaking hazard along the Wasatch Front, Utah, in Gori. P.L. and Hays, W.W., editors, Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah: U.S. Geological Survey Open-File Report 87-585, Vol. 2, pp. M-1-110.

Zoback, M.L., 1983, Structure and Cenozoic tectonism along the Wasatch fault zone, in Miller, D.M., Todd, V.R., and Howard, K.A., editors, Tectonic and stratigraphic studies in the eastern Great Basin: Geological Society of America Memoir 157, pp. 3-27.

Zoback, M.L., and Zoback, M.D., 1989, Tectonic stress field of the continental United States, in Pakiser, L.C., and Mooney, W.D., eds., Geophysical framework of the continental United States: Geological Society of America Memoir 172, pp. 523-539.

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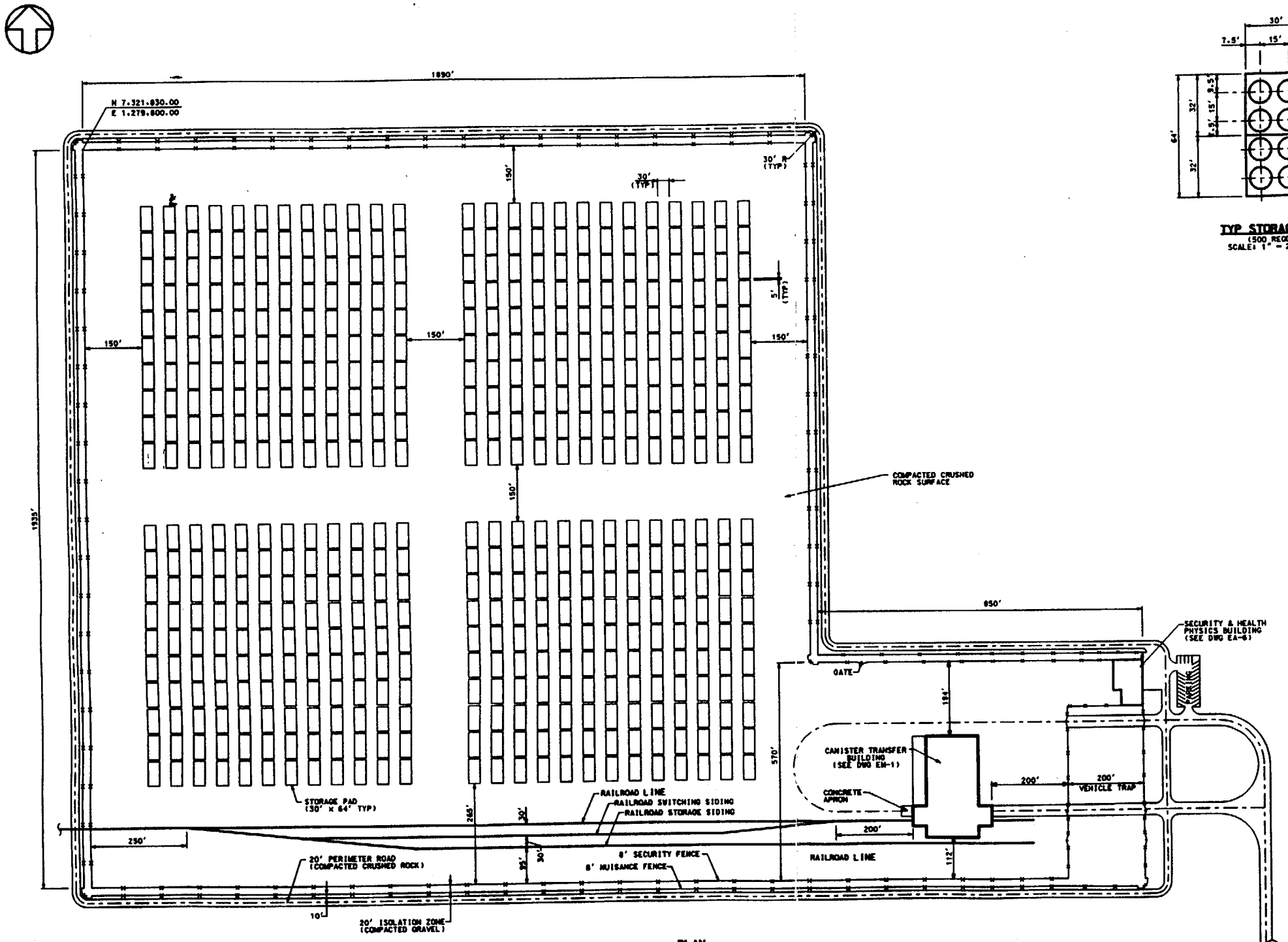
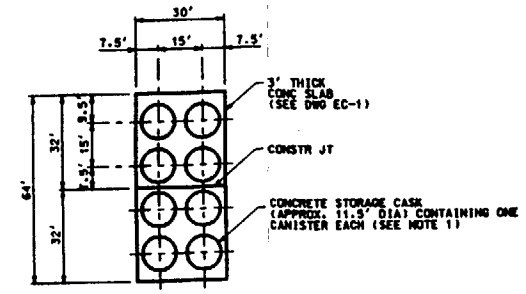


PLAN
PRIVATE FUEL STORAGE FACILITY
(STORAGE CAPACITY = 40,000 MTU)



TYP STORAGE PAD
(500 REQD)
SCALE: 1" = 20'-0"

APERTURE CARD
Available on Aperture Card

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Figure 1.2-1

PFSF GENERAL ARRANGEMENT

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

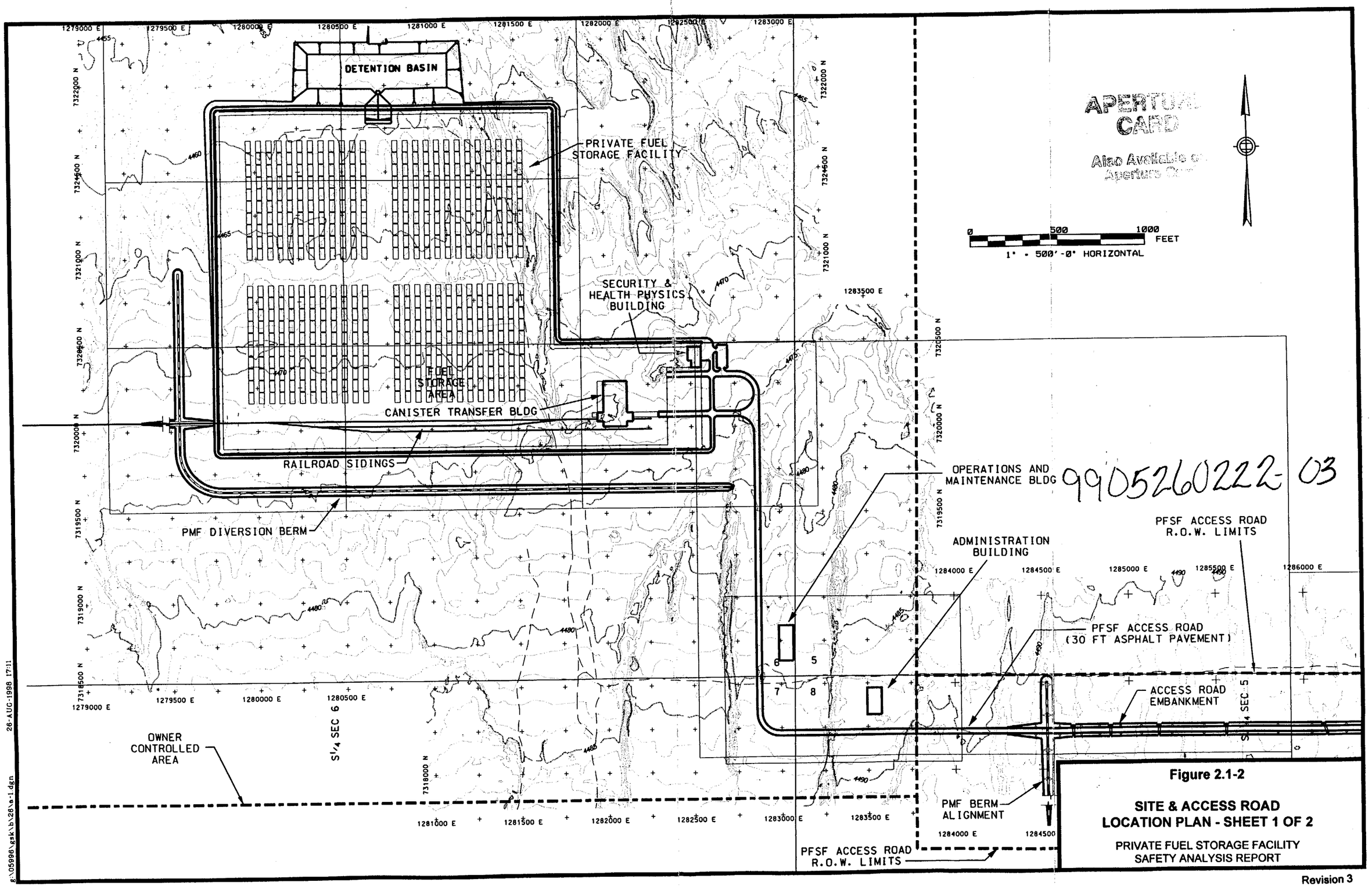

Figure 2.1-2

SITE & ACCESS ROAD LOCATION PLAN - SHEET 1 OF 2

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

APPENDIX 2D

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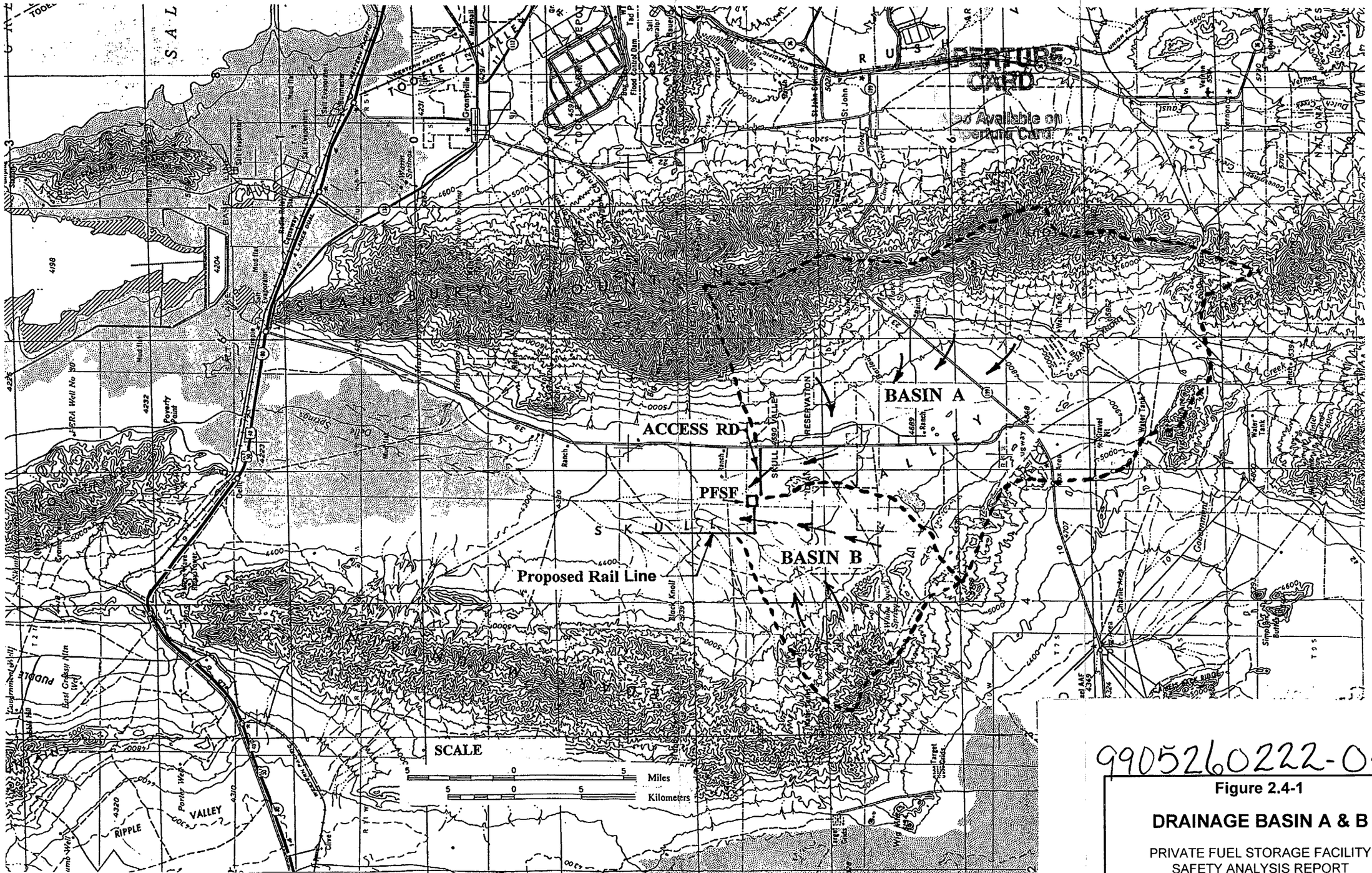

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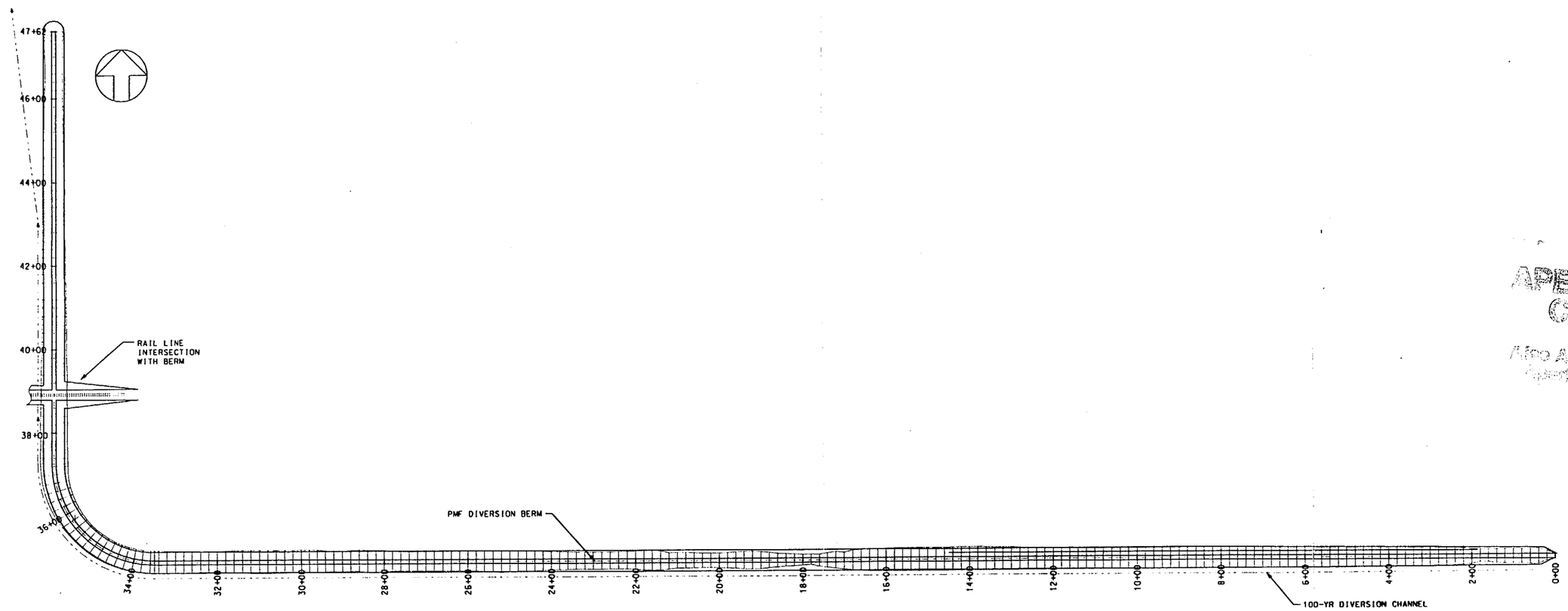
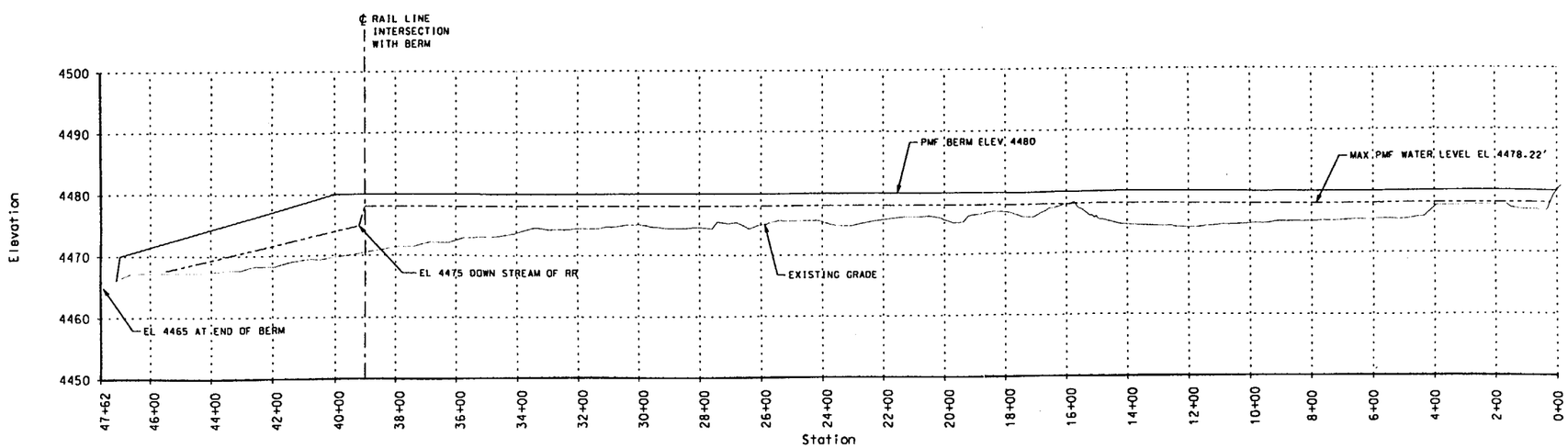
Figure 2.4-1

DRAINAGE BASIN A & B

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



PLAN
(BASIN B - BERM)



PROFILE

APERTURE CARD
Also Available on Aperture Card

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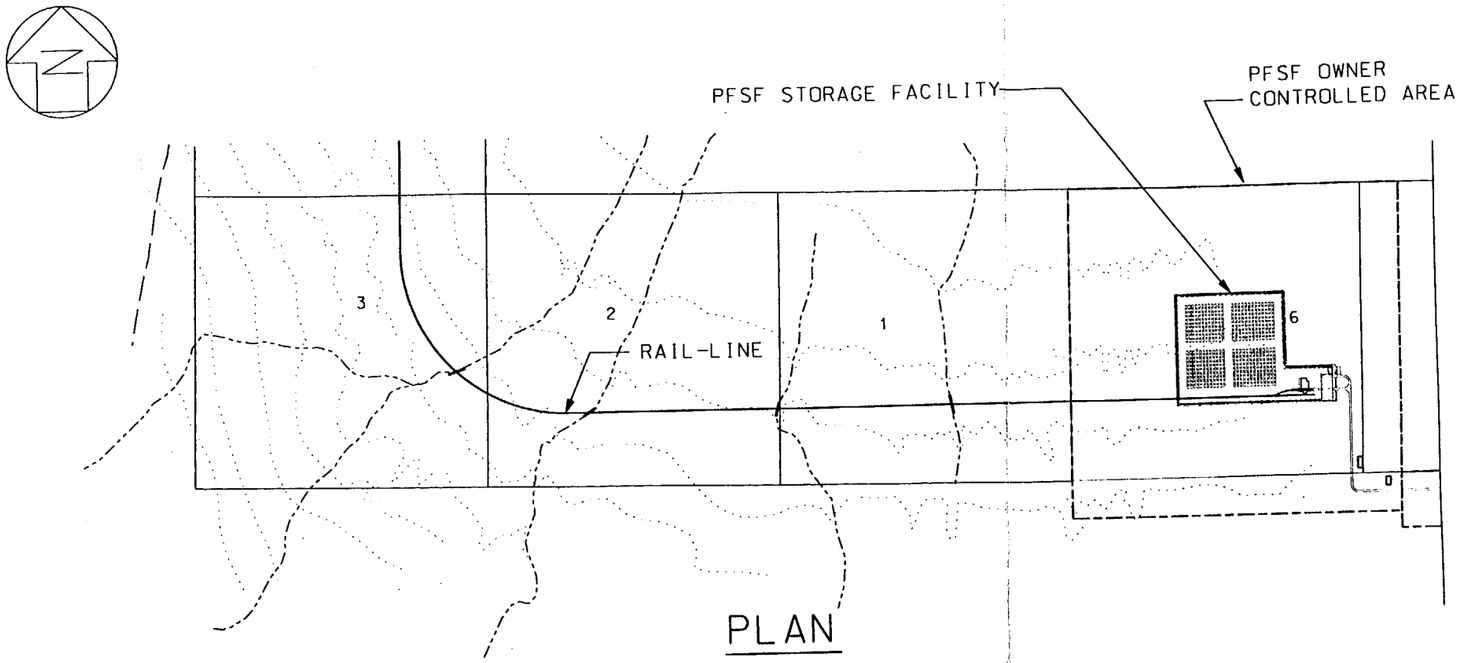
NOTE:
SEE FIGURE 2.1-2 FOR OVERALL SITE AND ACCESS ROAD LOCATION PLAN.

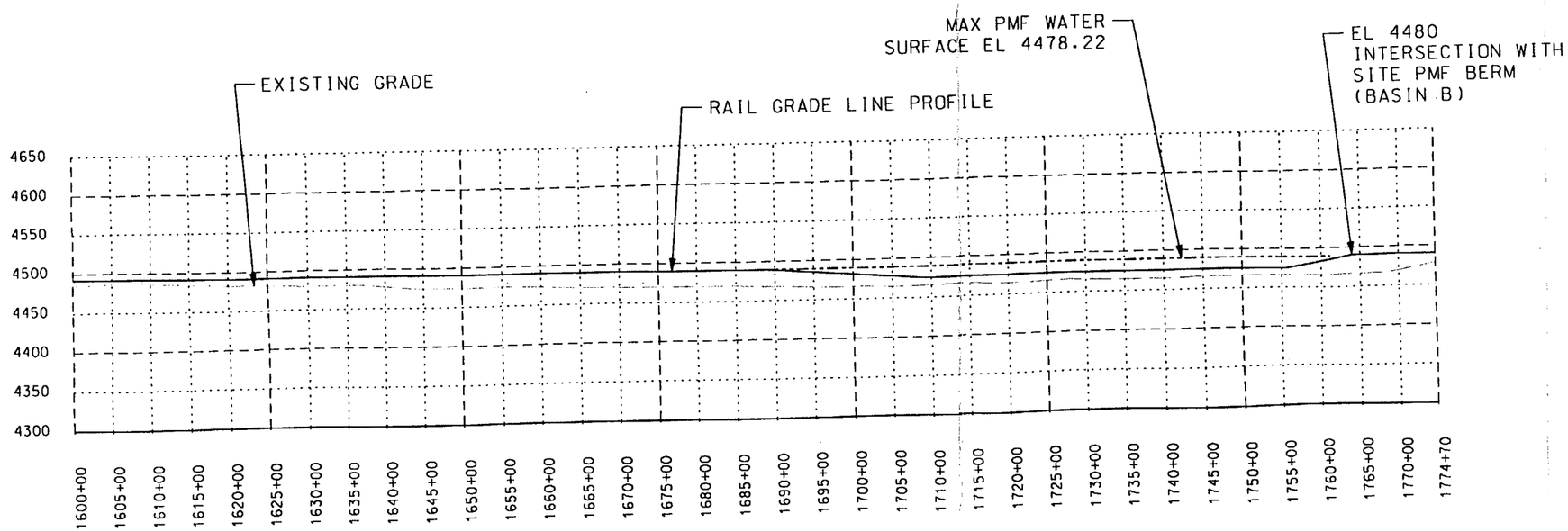
Figure 2.4-2

PFSF SITE PMF BERM PLAN & PROFILE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



PLAN



VERTICAL ALIGNMENT
PROFILE

APERTURE CARD
Also Available on Aperture Card

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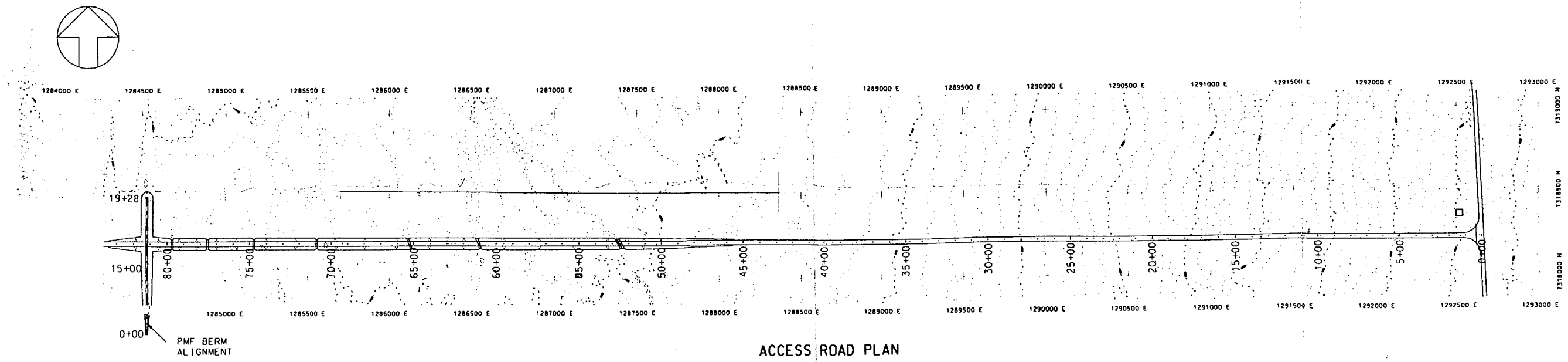
NOTE:
SEE FIGURE 2.1-2 FOR OVERALL SITE AND ACCESS ROAD LOCATION PLAN.

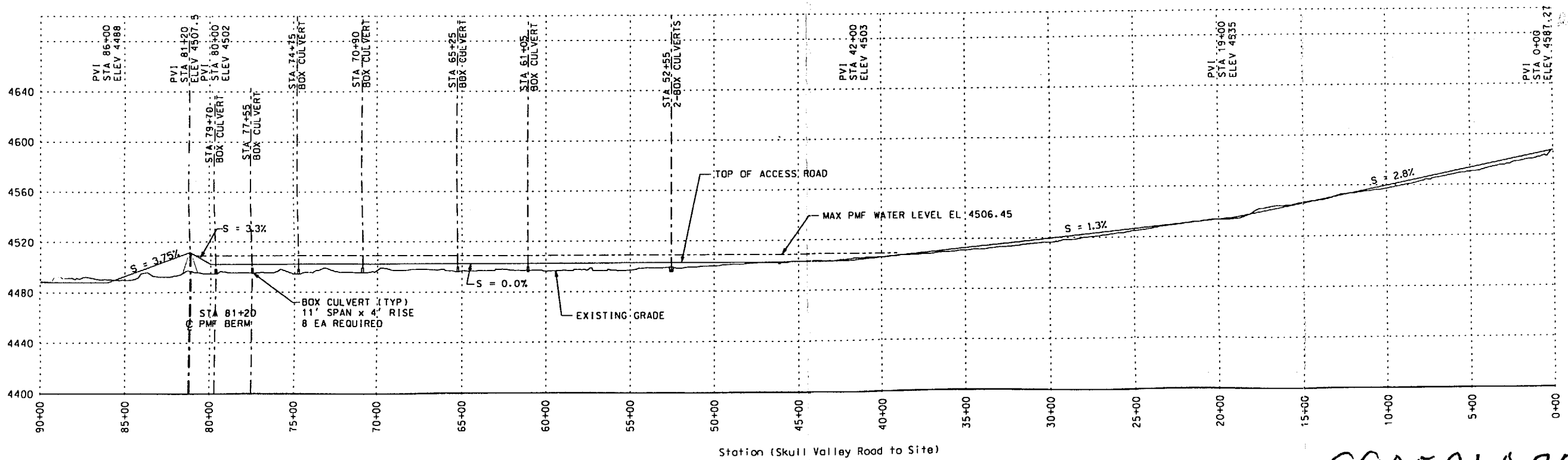
Figure 2.4-3

PFSF RAIL LINE
PLAN & PROFILE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



ACCESS ROAD PLAN



ACCESS ROAD PROFILE

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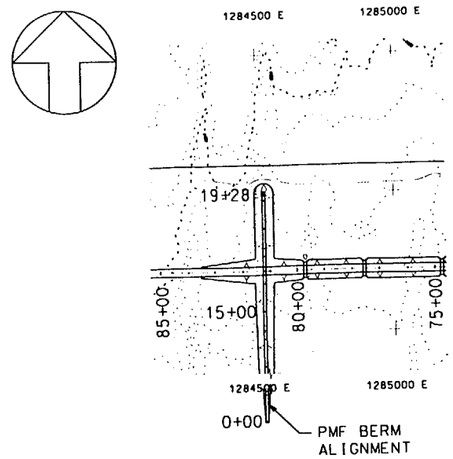
NOTE:
SEE FIGURE 2.1-2 FOR OVERALL SITE AND ACCESS ROAD LOCATION PLAN.

Figure 2.4-4

PFSF ACCESS ROAD
PLAN & PROFILE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



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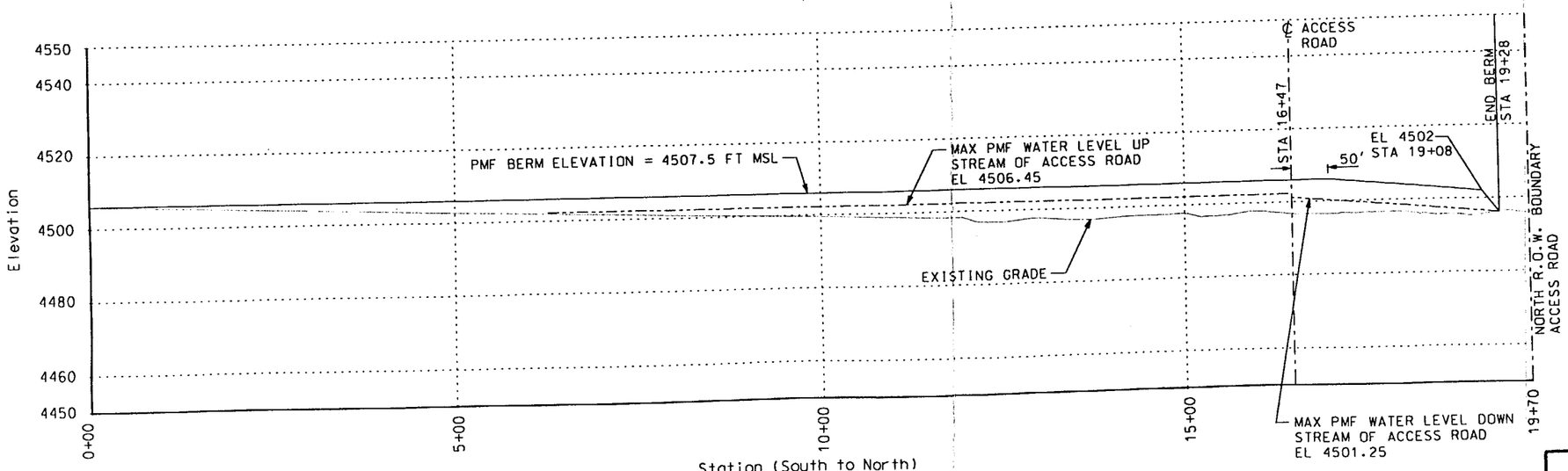

PMF BERM PROFILE
(BETWEEN BASIN A/B)

APERTURE
CARD
Also Available on
Aperture Card

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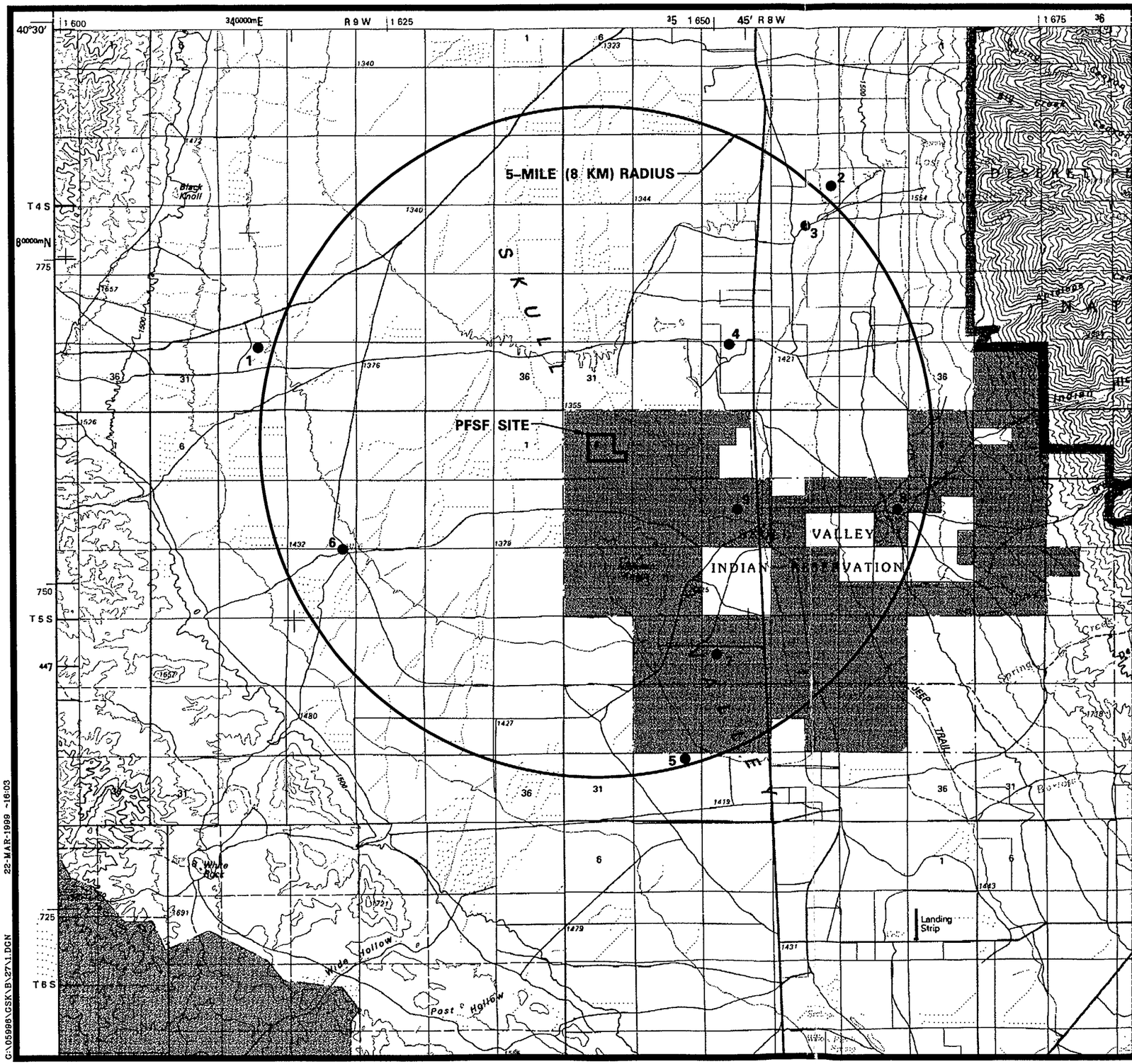
NOTE:
SEE FIGURE 2.1-2 FOR OVERALL SITE AND ACCESS ROAD LOCATION PLAN.

Figure 2.4-5

PFSF ACCESS ROAD PMF BERM
PLAN & PROFILE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



APERTURE CARD

Also Available on Aperture Card

WELL MAP NO.	OWNER	TOTAL DEPTH	DATE DRILLED	USE	DEPTH TO WATER	YIELD	CURRENT WATER RIGHT
1	SKULL VALLEY CO., LTD.	340'	1956	STOCK WATERING	295'	35 gpm	YES (1955) 0.015 cfs
2	ISLAND RANCHING CO., INC.	325'	NO INFO.	IRRIGATION, STOCK	NO DATA	NO DATA	YES (1959) 2.180 cfs
3	ISLAND RANCHING CO., INC.	347'	1954	IRRIGATION, STOCK	90'	NO DATA	YES (1954) 1.226 cfs
4	ANSCHUTZ LAND CO., LTD.	408'	NO INFO.	IRRIGATION, STOCK	NO DATA	NO DATA	YES (1960) 0.7487 cfs
5	ANSCHUTZ LAND CO., LTD.	209'	1948	STOCK WATERING	150'	20 gpm	YES (1948) 0.015 cfs
6	ANSCHUTZ LAND CO., LTD.	292'	1940	STOCK WATERING	280'	12 gpm	YES (1940) 0.015 cfs
7	SKULL VALLEY INDIAN RESERV.	401'	1975	DOMESTIC, INDUSTRIAL	77.5'	15 gpm	NOT REQUIRED
8	SKULL VALLEY INDIAN RESERV.	651'	1976	DOMESTIC	519.5'	60 gpm	NOT REQUIRED
9	SKULL VALLEY INDIAN RESERV.	NO DATA	NO DATA	DOMESTIC	NO DATA	NO DATA	NOT REQUIRED

SOURCES: BASE MAP IS RUSH VALLEY, UT, BUREAU OF LAND MANAGEMENT SPECIAL EDITION, SURFACE MANAGEMENT STATUS, 1:100,000 SCALE METRIC, 1981. WELL DATA ARE FROM STATE OF UTAH, DIVISION OF WATER RIGHTS AND HOOD AND WADDELL (1968).

LEGEND:

● WATER WELL LOCATION

SCALE 1 : 100,000

CONTOUR INTERVAL IS 50 METERS

KILOMETERS 0 1 2 3 4 5 6

MILES 0 1 2 3 4

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Figure 2.5–1

WATER WELLS WITHIN 5 MILES (8 KM) OF PFSF SITE

PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

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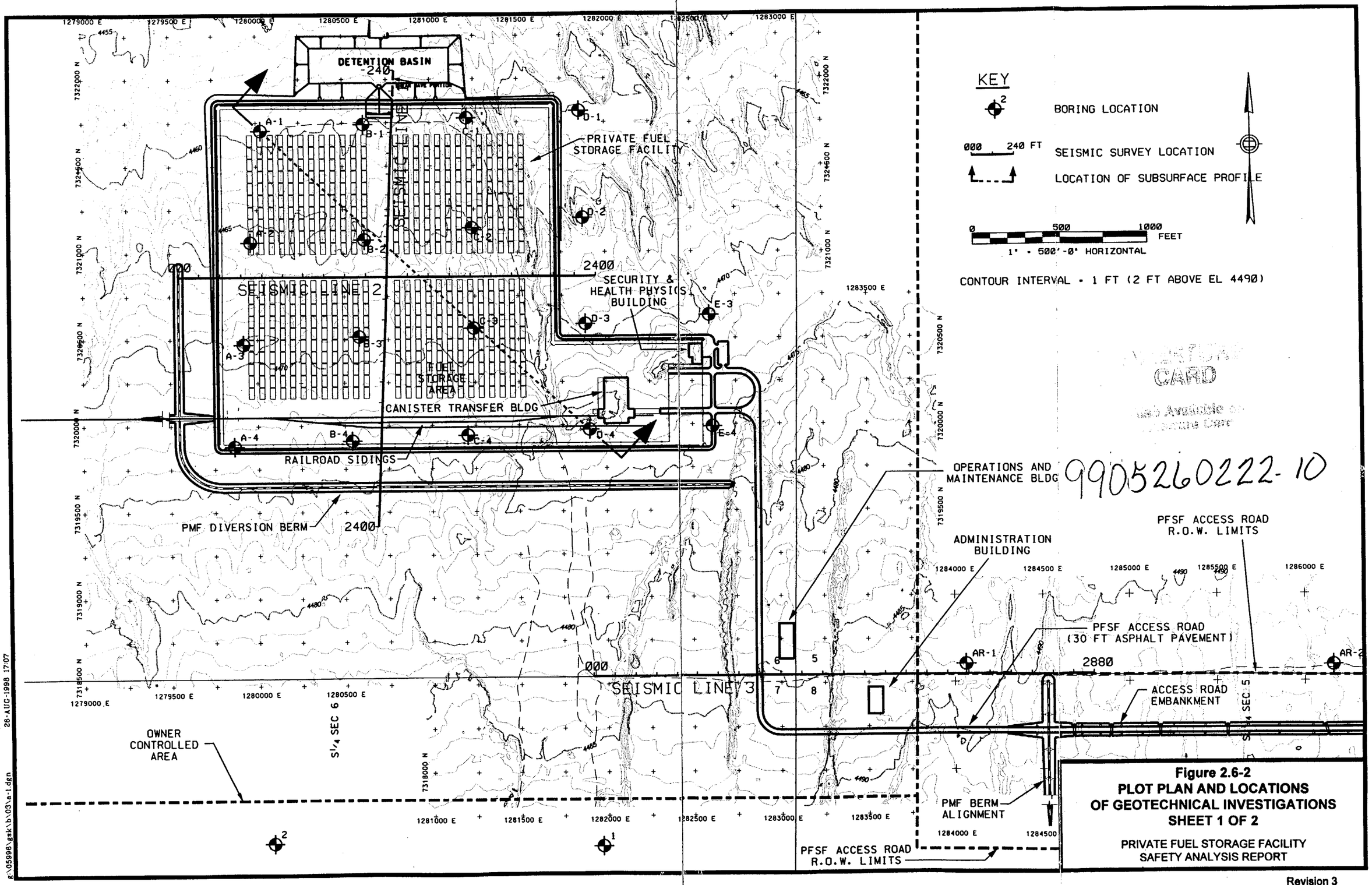

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Figure 2.6-2
PLOT PLAN AND LOCATIONS OF GEOTECHNICAL INVESTIGATIONS SHEET 1 OF 2

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

CHAPTER 3

PRINCIPAL DESIGN CRITERIA

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The PFSF shall be designed to utilize two transport modes to haul the shipping cask from the railroad mainline to the site. The preferred mode is to haul the shipping cask by rail on a railroad line to be constructed from Low Junction to the PFSF. The railroad line and associated equipment shall be designed in accordance with railroad industry standards. The alternate mode is to haul the shipping cask by highway on a heavy haul tractor/trailer from an intermodal transfer point, located next to the railroad mainline 1.8 miles West of Timpie, to the PFSF via Skull Valley Road. The intermodal transfer point shall include the necessary components (crane, rail siding, and truck access area) to accommodate the rail to tractor/trailer transfer.

At the PFSF the canister shall be transferred from the shipping cask to the storage cask. The shipping cask shall be off-loaded from the transport vehicle inside the Canister Transfer Building using an overhead crane and placed in a shielded transfer cell. Once the shipping cask has been opened a transfer cask shall be placed on top of the shipping cask and the canister hoisted up and secured into the transfer cask. The transfer cask shall then be moved by crane onto the top of a concrete storage cask and the canister shall be lowered into the storage cask. The storage cask lid shall be installed and bolted. The storage cask shall then be moved to the cask storage pad using a cask transporter. Storage of the loaded concrete storage cask shall include temperature monitoring and periodic surveillance of the storage casks.

When the fuel is to be shipped offsite, the storage cask shall be moved back into the Canister Transfer Building using the cask transporter. The transfer cask shall be placed on top of the storage cask and the canister lifted up and secured into the transfer cask. The transfer cask shall then be moved by crane onto the top of a shipping cask. The canister shall be lowered into the shipping cask, which shall be closed and shipped offsite.

The PFSF shall be designed with the necessary equipment (such as, the Canister Transfer Building, cranes, cask transporter, storage area) to accommodate shipping cask receipt, canister transfer from the shipping cask to the concrete storage cask, cask transport to and from the storage pads as detailed above with provisions for security, health physics, maintenance, document control, and inventory management.

3.1.2.2 Onsite Generated Waste Processing, Packaging and Storage

The selected canister-based storage systems shall be designed to confine spent fuel within a sealed canister at the originating nuclear power plant. Therefore, handling of spent fuel is not required and no radioactive waste is generated at the PFSF.

Health physics survey material (i.e. smears, disposable clothing) shall be collected, identified, packaged in low level waste (LLW) containers, marked in accordance with 10 CFR 20 requirements, and temporarily stored in the LLW holding cell of the Canister Transfer Building while awaiting shipment to an offsite low-level radioactive disposal facility.

There shall be no other systems or facilities for processing, packaging, storing, or transporting any other type of radioactive waste at the PFSF.

3.1.2.3 Utilities

The PFSF shall be designed to include utilities necessary for facility operation. These utilities include (1) electrical power for operation of equipment, lights, monitoring equipment, communication systems, security systems; (2) backup electrical power for operation of security systems, emergency lights, monitoring equipment, and communication systems; and (3) mechanical systems for operation of fire protection equipment, building HVAC systems, compressed air systems, water supply systems,

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 -35° F, recorded at Salt Lake City (Reference 5).

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 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. 24-hour average temperatures are not typically recorded. However, a conservative approach is to use the "average daily maximum temperature," which is an average of the peak temperatures throughout the hottest month, July. The highest average daily maximum temperature taken near the site, which bounds any 24-hour average temperature is 95° F, recorded at Iosepa Ranch, 8 miles NW of the site (Reference 6).

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.

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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

Two major watersheds have been identified which can contribute runoff to the PFSF site area as described in Section 2.4.1.2. A relatively large watershed from the lower Stansbury Mountains in the east to the Lookout Mountain in the south is identified as Basin A and a relatively smaller watershed from the lower Cedar Mountains in the west is identified as Basin B (see Figure 2.4-1). Basin A is separated from Basin B by an earthen berm (PMF Berm) which will be constructed at the PFSF to control runoff from these offsite sources.

Analyses of the probable maximum precipitation (PMP) were performed to determine a probable maximum flood (PMF) for stormwater drainage Basins A and B. For an extremely conservative PMF (Q_{PMF} = 85,000 cfs), the Basin A PMF water elevation predicted at the southeast and northeast corner locations of the site is 4,468.8 and 4,456.7 feet, respectively. The site grade elevations at these locations are 4,476 and 4,462 feet, respectively, which are higher than the predicted flood elevations. Consequently, all SSCs that are classified as Important to Safety are located above the Basin A PMF flood plain.

Basin B stormwater runoff from the lower Cedar Mountain drains as a sheet flow toward the PFSF site. An earthen berm and drainage ditch system will be constructed on the south and west sides of the PFSF storage site to divert the PMF stormwater flows around the site and into the Skull Valley natural drainage system. Consequently, all SSCs that are classified as Important to Safety are protected from the sheet flow associated with the Basin B PMF by the earthen berm.

Therefore, forces due to flood waters and flood protection measures need not be considered in the design of SSCs that are classified as Important to Safety.

3.2.10 Seismic Design

The design of SSCs classified as Important to Safety shall consider loadings associated with the ISFSI design basis ground motion, which was determined by a probabilistic seismic hazard analysis as discussed in Section 2.6. Probabilistic analysis does not result in the determination of a unique Design Earthquake, such as is the case for a deterministic analysis. Instead, various scenarios and models are used to estimate the likelihood of earthquake ground motions at a site and systematically take into account uncertainties that exist in various hazard parameters. The results are in the form of hazard curves that express the mean annual probabilities or frequencies with which various levels of fault displacement and ground motion are expected to be exceeded. Regulatory Guide 1.29 (Reference 10) was used to define the SSCs that are required to withstand the loadings associated with the ISFSI design basis ground motions. These SSCs are identified in Regulatory Guide 1.29 as seismic Category I.

3.2.10.1 Input Criteria

Tooele County is located west of the Rocky Mountain Front, which is defined in 10 CFR 72.102 as approximately 104° west longitude. As described in Section 2.6, a probabilistic seismic hazard analysis was performed to establish the appropriate seismic design basis for the facility. This analysis applies the guidance in Regulatory Guide 1.165 (Reference 25) to the PFSF site. A return period of 1,000 years was determined to be appropriate (Reference 26).

In addition, a site-specific geotechnical investigation was performed to ensure the geological characteristics and soil are stable under earthquake conditions as described in Section 2.6.

3.2.10.1.1 Design Response Spectra

The ISFSI design basis ground motion for the PFSF is described by site-specific response spectrum curves anchored at 0.40g in two directions of the horizontal plane and 0.39 g in the vertical plane. The response spectra curves are free field at the ground surface and account for the local soil conditions. The horizontal and vertical design response spectra curves for the site are shown in Figure 4 of Reference 27.

3.2.10.1.2 Design Response Spectra Derivation

Site-specific horizontal and vertical design response spectra curves for the facility are developed in accordance with 10 CFR 100, Appendix A (Reference 11) as described in References 27 and 28.

3.2.10.1.3 Design Time History

Design time histories shall be used in the cask stability analyses and in the storage pad design. Statistically independent artificial time histories shall be developed in accordance with NUREG-0800, Sections 3.7.1 and 3.7.2 shall be shown to envelope the site-specific response spectra.

3.2.10.1.4 Use of Equivalent Static Loads

The HI-STORM storage system is dynamically analyzed and does not use equivalent static loads.

Equivalent static loading is used on the TranStor storage system since the concrete cask is a very rigid body with natural frequencies in excess of the zero period acceleration cutoff. No dynamic amplification by the cask is expected.

Equivalent static loads are not used for onsite structures since dynamic analyses are used in the seismic analysis and design.

3.2.10.1.5 Critical Damping Values

Critical damping values shall be in accordance with Regulatory Guide 1.61 (Reference 13) for a SSE.

3.2.10.1.6 Basis for Site-Dependent Analysis

Site-specific vibratory ground motion is established through evaluation of the seismology, geology, and the seismic and geologic history of the site and surrounding region. This information is contained in the site-specific probabilistic seismic hazard analysis (References 27 and 28).

3.2.10.1.7 Soil-Supported Structures

The soil-supported structures that shall be analyzed for the ISFSI design basis ground motion are the concrete cask storage pads and the Canister Transfer Building. These structures shall be founded on in-situ soil at a minimum depth of 2 ft 6 inches for frost

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 modal response spectrum 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.

3.2.10.2.4 Rocking and Translational Response Summary

Rocking and translational response shall be modeled by including equivalent rocking and translational soil springs in accordance with the spring constants described in Table 3300-2 of ASCE-4.

3.2.10.2.5 Methods Used to Couple Soil with Seismic-System Structures

The soil can be represented by discrete springs or a finite element model to represent the soil substratum.

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3.2.10.2.6 Methods Used to Account for Torsional Effects

The storage pads and the Canister Transfer Building shall be modeled and analyzed as 3-dimensional multimass systems. Therefore, torsional effects due to eccentricities of the mass are taken into account in the analysis.

3.2.10.2.7 Methods for Seismic Analysis of Dams

There are no dams onsite or in the immediate area.

3.2.10.2.8 Methods to Determine Overturning Moments

Overturning stability shall be assured for the storage casks on the pads.

Overturning stability of loaded concrete casks located on the storage pad shall be proved by both storage system vendors with a dynamic analysis using the site-specific seismic design parameters and considering soil-structure interaction.

3.2.10.2.9 Analysis Procedure for Damping

Critical damping values shall be developed in accordance with Regulatory Guide 1.61 for a SSE.

3.2.10.2.10 Seismic Analysis of Overhead Cranes

The overhead bridge and semi-gantry cranes shall be analyzed for seismic effects in accordance with the requirements of NUREG-0554 for single-failure-proof cranes. The seismic analysis of the cranes shall include the Maximum Critical Load in the lifted position during a seismic event. The seismic analysis methods shall be in accordance

with ASME NOG-1 (Reference 16). A set of amplified response spectra curves at the crane rail locations shall be developed for use in the crane seismic analysis and design.

3.2.10.2.11 Seismic Analysis of Specific Safety Features

SSCs classified as Important to Safety shall meet the requirements of 10 CFR 72.122(b)(2), which requires SSCs be designed such that design basis ground motion will not result in an uncontrolled release of radioactive material or increased radiation exposure to workers or members of the general public.

3.2.11 Combined Load Criteria

The design shall consider all appropriate loads and load combinations as required by the specific SSC design code(s). Design loads shall be determined from normal, off-normal, and accident-level conditions. Design loads shall be combined to simulate the most adverse load conditions

3.2.11.1 HI-STORM Storage System Load Combinations

Loads and load combinations used in the design of the HI-STORM 100 Cask System are identified in the HI-STORM SAR, Sections 2.2.7 and 3.1.2.1.2. Exceptions to the various code criteria are shown in HI-STORM SAR, Table 2.2.15.

HI-STORM Canister

The canister shell and internals are required by the HI-STORM SAR to be designed to the applicable requirements of Subsections NB and NG of the ASME BPVC, Section III (Reference 17). The load combinations for all normal, off-normal and accident conditions and corresponding Service Levels of the canister design are as follows:

3.7 REFERENCES

1. Topical Safety Analysis Report for the Holtec International Storage and Transfer Operation Reinforced Module Cask System (HI-STORM 100 Cask System), Holtec Report HI-951312, Docket 72-1014, Revision 1, January 1997,.

2. Safety Analysis Report for the TranStor Storage Cask System, SNC-96-72SAR, Sierra Nuclear Corporation, Docket 72-1023, Revision B, March 1997.

3. ASCE-7 (formerly ANSI A58.1), Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, 1995.

4. ANSI/ANS 57.9, Design Criteria For An Independent Spent Fuel Storage Installation (Dry Storage Type), 1984.

5. Local Climatological Data, Annual Summary with Comparative Data for 1991, Salt Lake City, Utah, National Oceanic and Atmospheric Administration, National Environmental Satellite Data and Information Service, National Climatic Data Center, March 1992.

6. Ashcroft, G.L., D.T. Jensen and J.L. Brown, 1992, Utah Climate; Utah Climate Center, Utah State University.

7. NUREG-0800, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, U.S. Nuclear Regulatory Commission, June 1987.

8. Regulatory Guide 1.76, Design Basis Tornado for Nuclear Power Plants, U.S. Nuclear Regulatory Commission, April 1974.

9. Private Fuel Storage Facility Storage Facility Design Criteria, Section 4.0, Geotechnical Design Criteria, Revision 2.

10. Regulatory Guide 1.29, Seismic Design Classification, U.S. Nuclear Regulatory Commission, September 1978.

11. 10 CFR 100, Appendix A, Seismic and Geologic Siting Criteria for Nuclear Power Plants.

12. Deterministic Earthquake Ground Motion Analysis, Private Fuel Storage Facility, prepared by Geomatrix Consultants, Inc. and William Lettis & Associates, SWEC Report No. 0599601-G(P05)-1, Revision 0.

13. Regulatory Guide 1.61, Damping Values For Seismic Design Of Nuclear Power Plants, U.S. Nuclear Regulatory Commission, October 1973.

14. ASCE-4, Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety-Related Nuclear Structures, American Society of Civil Engineers, 1986.

15. NUREG-0554, Single-Failure-Proof Cranes for Nuclear Power Plants, U.S. Nuclear Regulatory Commission, 1979.

16. ASME NOG-1, Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Bridge), 1989.

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.

TABLE 3.2-3

STRUCTURAL DESIGN CRITERIA FOR STEEL COMPONENTS USED IN THE TRANSTOR CANISTER

COMPONENT / (APPLICABLE CODE)	CRITERIA
1. Basket Normal Operation (ASME III, NC/shell/ and NG/internals/, Service Level A)	$P_m \leq S_m$ $P_m + P_b \leq 1.5\ S_m$ $P_L + P_b + Q \leq 3S_m$
Lifting Devices (ANSI N14.6 and NUREG 0612, 10% dynamic factor)	Redundant load path: max principal stress $\leq S_u/5$ or $S_y/3$ Non-redundant load path: max principal stress $\leq S_u/10$ or $S_y/6$
2. Basket Off-Normal Operation (ASME III, NC/shell/ and NG/internals/, Service Level B)	$P_m < 1.1\ S_m$ $P_L + P_b < 1.65\ S_m$ $P_L + P_b + Q < 3\ S_m$
3. Basket Off-Normal Operation (ASME III, NC/shell/ and NG/internals/, Service Level C)	$P_m < 1.2\ S_m$ (shell), $< 1.5\ S_m$ (sleeve) $P_L + P_b < 1.8\ S_m$ (shell), $< 2.25\ S_m$ (sleeves)
4. Basket Accident Conditions, (ASME III, NC/shell/ and NG/internals/, Service Level D, NUREG/CR-6322)	$P_m \leq 2.4\ S_m$ or $0.7\ S_u$ (whichever is less) $P_L + P_b \leq 3.6\ S_m$ or $1.0\ S_u$ (whichever is less) Buckling interaction ratios < 1
5. Brittle Fracture (ASME III, NC/shell/ and NG/internals/)	Selection of structural material with adequate toughness. Control by operating procedures based on minimum temperature. Carbon steel below 5/8" in thickness and stainless steel are exempt from fracture toughness testing and requirements.

TABLE 3.4-1

QUALITY ASSURANCE CLASSIFICATION OF STRUCTURES, SYSTEMS, AND COMPONENTS

IMPORTANT TO SAFETY	NOT IMPORTANT TO SAFETY
Classification Category A	Storage Facility Infrastructure
Spent Fuel Canister	Security and Heath Physics Building
	Administration Building
Classification Category B	Operations and Maintenance Building
Concrete Storage Cask	Intrusion Detection System
Transfer Cask	CCTV System
Associated Lifting Devices	Restricted Area Lighting
Canister Transfer Building	Security Alarm Stations
Canister Transfer Overhead Bridge Crane	Electrical Power - UPS
Canister Transfer Semi-gantry Crane	Electrical Power - Backup Diesel Generator
Seismic Support Struts	Electrical Power - Normal
	Yard/Building Lighting
Classification Category C	Cask Transporter
Cask Storage Pads	Radiation Monitors
	Temperature Monitoring System
	Communication Systems
	Fire Detection/Suppression
	Water Supply Systems
	Septic Systems
	Access Road
	Road Transport Alternative components
	Railroad Line Alternative components

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	HI-STORM 24 PWR assemblies/canister 68 BWR assemblies/canister TranStor 24 PWR assemblies/canister 61 BWR assemblies/canister	HI-STORM SAR, Section 1.1 TranStor SAR, Section 1.1
Weights (maximum)	HI-STORM Storage Cask - 267,664 lbs. Loaded Canister - 86,131 lbs. Transfer Cask - 151,963 lbs. Shipping Cask - 153,080 lbs. TranStor Storage Cask - 223,435 lbs. Loaded Canister - 84,460 lbs. Transfer Cask - 126,630 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 (at 84 mph)	1800 kg automobile 125 kg 8" armor piercing artillery shell 1" solid steel sphere	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	0.40g, horz.(both directions) & 0.39 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 = -35°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
HI-STORM 100 Cask System Load Criteria	Canister: } See HI-STORM SAR, Table 2.2.6 Internals: } See HI-STORM SAR, Table 2.2.6 Storage Cask: } See HI-STORM SAR, Table 2.2.6 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

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conditions, therefore, the HI-STORM design meets the PFSF design criteria in Section 3.2.9 for flood design.

H. Earthquake

Earthquake loads are addressed in the HI-STORM SAR Sections 2.2.3.7 and 3.4.7 and Appendix 3B. The HI-STORM SAR shows that the storage system will withstand the imposed loads and not tip over when subjected to a generic seismic event. A generic seismic event was defined for the HI-STORM system using response spectra curves from Regulatory Guide 1.60 (Reference 6) with a zero period acceleration of up to 0.80 g in three orthogonal directions.

The cask vendor also performed a site specific analysis and determined the storage cask will withstand the imposed loads and not tip over when subjected to the PFSF deterministic design earthquake (0.67g vertical, 0.69g horizontal – See Section 8.2.1.1) (References 7 and 8). Soil-structure interaction was considered in the site specific analysis. The seismic cask stability analyses are fully described in Section 8.2.1.

Inertia loads produced by the seismic event are less than the 45 g loads due to the postulated HI-STORM vertical drop and non-mechanistic tip over events as discussed in Section 8.2.6. Stresses in the canister due to the seismic event are bounded by those analyzed for the 45 g deceleration load from the hypothetical end drop and side drop events discussed in HI-STORM SAR Section 3.4.4.3.1. As discussed in HI-STORM SAR Appendix 3.B, shell and concrete stresses in the storage cask resulting from the seismic shear and moment forces are evaluated and determined to be acceptable.

Even though the storage cask will not tip over during an earthquake, the cask is conservatively analyzed for a hypothetical cask tip over event in HI-STORM SAR Section 11.2.3. Both the cask and canister are shown to withstand this non-credible hypothetical event without loss of integrity.

Therefore, the HI-STORM 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 HI-STORM SAR Section 11.2.11. Regulatory Guide 1.91 (Reference 9) requires a detailed review of the system for overpressures that exceed 1 psi. The HI-STORM storage system is analyzed and designed for accident external pressures up to 60 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 HI-STORM 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 loads are addressed in HI-STORM SAR Section 11.2.4. The HI-STORM storage system was analyzed for a fire of 200 gallons of combustible fuel encircling the cask, resulting in temperatures up to 1,475° F and lasting for a period of 15 minutes. Nonetheless, the analysis also evaluated the temperatures of the system for a fire duration of 10 hours. The results of the analysis show that the intense heat from the fire only partially penetrated the concrete cask wall and that the majority of the concrete experienced only minor temperature increases. The cask transporter will contain up to 50 gallons of fuel and therefore would have less fire consequences than the HI-STORM

BWR assembly (HI-STORM SAR Tables 2.1.7 and 2.1.8). The analysis assumed HI-STORM storage casks are in an array, subjected to an 80° F annual average ambient temperature, with solar radiation. The results of this analysis are presented in Table 4.2-3 for MPC-24 and MPC-68 canisters. The results indicate that temperatures of all components are within maximum allowable temperatures.

Holtec considered stainless steel clad fuels in the thermal analysis, as discussed in HI-STORM SAR Section 4.3.1. Stainless steel cladding is less conductive than zircaloy clad fuel and the net thermal resistance of a basket full of stainless steel clad fuel is greater, which would result in higher cladding temperatures for stainless steel fuel assemblies having the same decay heat generation rate as zircaloy clad fuel. However, the design basis decay heat for stainless steel clad fuel is significantly lower than that of zircaloy clad fuel, as noted previously, and the allowable temperature limit for stainless steel cladding is considerably higher than for zircaloy cladding. Holtec determined that the reduction in heat duty is much more pronounced than the nominal increase in the resistance to heat transfer, and concluded that the peak cladding temperature for stainless steel clad fuel will be bounded by the results for zircaloy clad fuel and a separate analysis for stainless steel clad fuel was not required.

HI-STORM SAR Section 11.1.2 evaluates temperatures of the HI-STORM storage system for a maximum off-normal daily average ambient temperature of 100° F, an increase of 20° F from the normal conditions of storage discussed above. The maximum off-normal temperatures were calculated by adding 20° F to the maximum normal temperatures from the highest component temperature for MPC-24 and MPC-68. All the maximum off-normal temperatures are below the normal condition temperature limits except the canister shell temperature for MPC-24 (452° F). However, the off-normal high ambient temperature is of a short duration, and the resultant temperatures were evaluated against the accident condition temperature limits. The accident condition temperature limit for the canister shell is 775° F

(HI-STORM SAR Table 2.2.3). Therefore, all components are within allowable temperatures for the 100° F ambient temperature condition.

The HI-STORM storage system was analyzed for a -40° F extreme low ambient temperature condition, as discussed in HI-STORM SAR Chapter 4. Holtec conservatively assumed zero decay heat generation from spent fuel, and no solar radiation, resulting in all storage system components reaching the -40° F temperature. As stated in the HI-STORM SAR, all HI-STORM materials of construction will satisfactorily perform their intended function in the storage mode at this minimum temperature condition.

The PFSF site low ambient temperature of -35° F, maximum annual average temperature of 51° F (normal), and average daily maximum temperature of 95° F (off-normal) are bounded by the corresponding temperatures used for the HI-STORM storage system of -40° F, 80° F, and 100° F, respectively. Therefore, the thermal design of the HI-STORM storage system bounds the site specific design requirements.

4.2.1.5.3 Shielding Design

Shielding design and performance for the HI-STORM storage system is addressed in HI-STORM SAR Chapter 5. The HI-STORM storage system is designed to maintain radiation exposure as low as is reasonably achievable (ALARA) in accordance with 10

generated missile loads are the same as 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 tip over 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 0.75 g horizontal (both directions) and 0.50 g vertical. The cask vendor also performed a site specific analysis and determined the storage cask will withstand the imposed loads and not tip over when subjected to the PFSF deterministic design earthquake (0.67g vertical, 0.69g horizontal – See Section 8.2.1.1). Soil-structure interaction was considered in the site specific analysis. The seismic cask stability analyses are 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 of fire. The thick concrete walls of the storage cask are capable of protecting the

system operates well within thermal design limits. Therefore, no degradation due to temperature effects on materials or components is expected. The analyses results represent maximum temperatures, since the heat source from the fuel assemblies decays with time. While allowable temperatures for the TranStor construction materials do not change, the fuel temperature limits decrease with time. However, SNC notes in TranStor SAR Chapter 4 that the heat load decays faster than the corresponding maximum allowable cladding temperatures, and margins between actual and allowable fuel cladding temperatures increase with time.

Off-normal and accident cases are described in TranStor SAR Chapter 11. The following steady state conditions have been analyzed:

1. Normal condition, average ambient temperature = 75° F, no solar radiation.
2. Off-normal condition, ambient temperature = 100° F, solar radiation.
3. Off-normal condition, ambient temperature = -40° F, no solar radiation.
4. Off-normal condition, ambient temperature = 75° F, no solar radiation, 1/2 of air inlets blocked.
5. Accident condition, ambient temperature = 125° F, solar radiation.
6. Accident condition, ambient temperature = 75° F, no solar radiation, all air inlets blocked.

The TranStor thermal analysis performed for the concrete storage cask verifies that material temperature limits are not exceeded for normal, off-normal, and accident conditions. The TranStor thermal analysis verifies that fuel cladding allowable temperature limits are not exceeded. The minimum temperatures for the TranStor system correspond to the coldest environmental conditions of -40° F and no heat load in the cask. However, even at these extreme conditions the components are above their minimum material temperature limits. The TranStor cask does not employ any temperature-sensitive features such as gaskets, packing, or O-rings.

The results of the thermal analysis for normal, off-normal, and accident conditions is shown in TranStor SAR Table 4.1-1. These results are summarized in Table 4.2-6.

The PFSF site low ambient temperature of -35° F, maximum annual average temperature of 51° F (normal), and average daily maximum temperature of 95° F (off-normal) are bounded by the corresponding temperatures used for the TranStor storage system of -40° F, 75° F, and 100° F, respectively. The heat generation of the fuel to be stored at the PFSF is bounded by the heat generation of the TranStor design basis fuel. Therefore, the thermal design of the TranStor storage system bounds the site specific design requirements.

4.2.2.5.3 Shielding Design

Shielding for the TranStor storage system is addressed in TranStor SAR Chapter 5. The TranStor storage system is designed to maintain ALARA radiation exposure in accordance with 10 CFR 72.126(a). The concrete storage cask is designed to limit the average external dose rate (gamma and neutron) one meter from the cask to less than 15 mrem/hr on the sides (30 mrem/hr for stainless steel clad fuel) and 200 mrem/hr on top at the cover lid centerline based on TranStor design basis fuel. The design dose rates allow limited personnel access during canister closure operations.

Radiation shielding of the TranStor storage system is provided by the 0.75 inch thick steel canister shell, the 2 inch thick steel storage cask liner, and the 29 inch thick reinforced concrete cask wall. Axial shielding at the top is provided primarily by the steel canister shield and structural lids, which have a combined thickness of 11 inches. The 0.75 inch thick steel storage cask lid also provides axial shielding. The inlet and outlet ducts are configured to prevent direct radiation streaming from the spent fuel assemblies to the outside of the cask.

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 356,521 lbs. This maximum weight is associated with the HI-STORM storage cask loaded with an MPC-32 (32 fuel assembly capacity PWR canister) and envelopes the maximum loaded weight of both the TranStor and HI-STORM concrete storage casks proposed for use at the PFSF. The TranStor storage cask has a maximum loaded

weight of 309,130 lb. as shown on TranStor SAR Table 3.2-1. The HI-STORM storage cask has a maximum loaded weight of 356,521 lb. (MPC-32) as shown on HI-STORM SAR Table 3.2.1. The Holtec MPC-32 has the maximum weight of all of the HI-STORM series canisters and is conservatively used in the design, even though it is not proposed for use at the PFSF. The HI-STORM canisters proposed for use at the PFSF are the MPC-24 and the MPC-68 with maximum weights of 346,495 lb. and 353,796 lb., respectively, when in the HI-STORM storage cask, both of which are bounded by the weight of the MPC-32, when in the HI-STORM storage cask.

The cask storage pad design also considers the weight of the loaded concrete storage casks in combination with the seismic loads due to the PFSF deterministic design earthquake (0.67g vertical, 0.69g horizontal – 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 4,000 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 PFSF deterministic design earthquake loading also 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 (20 kips/ft^3) of the supporting soil medium (Reference 19). For the short-term PFSF deterministic design earthquake dynamic 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 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 PFSF deterministic design 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 then used for checking the structural adequacy of the concrete pad and the soil bearing and sliding stabilities. 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 for the pad, was conducted for the dead load equal to the gravitational dead weight of the pad and the live load of the casks. 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 356.5 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 factor equal to 1.0 was used for the load of one fully-loaded cask plus the transporter to account for any dynamic effect that may arise during transporting of the cask. From the analysis results obtained, the worst-case 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 a transporter. The maximum response results obtained from the static analyses are summarized in Table 4.2-7.

B. Dynamic Analysis

The dynamic analysis was performed to determine the pad response stresses under the PFSF deterministic design earthquake loading. The global seismic time-history response analysis was performed utilizing a series of cask-pad-soil interaction models representing 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 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 on the sliding displacements of casks on the pad and a lower-bound estimate on the cask dynamic forces acting on the pad; whereas, the upper-bound friction case results in a lower-bound estimate on the sliding displacements but an upper-bound estimate on 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. These dynamic forcing function inputs were provided by the Holtec site specific cask stability analysis for the HI-STORM storage cask (Reference 7). The HI-STORM storage cask weight and center of gravity loadings bound those of the TranStor storage cask. For the pad analysis, 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 represent 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 dynamic 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 as considered in the static analyses were also considered in the dynamic response analysis. 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 bearing and sliding stabilities of the soil supporting the pad. The results of the maximum dynamic response values obtained from the dynamic analysis described above are summarized in Table 4.2-8. Based on the analysis results obtained, the worst-case cask-loading pattern that produces the maximum dynamic pad internal stresses and soil pressures is that of two casks and the worst-case loading that produces the largest seismic horizontal soil reaction forces is that of 8 casks.

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

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 #7 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 4,000 psi and the reinforcing steel has a minimum yield strength of 60,000 psi. The design provides an ultimate static moment capacity of 338 k-ft/ft, an ultimate dynamic (impulse or impactive) moment capacity of 423 k-ft/ft, and a corresponding ultimate static and dynamic shear capacity of 123 k/ft and 135 k/ft, respectively. The maximum bending moment and shear force demands associated with the normal loading condition (1.4D + 1.7L + 1.7H) are 145.4 k-ft/ft and 19.2 k/ft, respectively; the maximum bending moment and shear force demands associated with the accident-level loading condition (D + L + H + E) are 383.3 k-ft/ft and 111.1 k/ft. Therefore, the storage pad as designed provides adequate strength for accommodating the design loading conditions.

Soil pressures beneath the storage pad were also verified to be within the acceptance criteria for all 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 for both static and dynamic load combinations. The maximum static soil pressure was calculated to be 2.12 ksf, 2.31 ksf, and 2.02 ksf for the pad dead load plus 2 casks, 4 casks, and 8 casks, respectively. A worst case static soil pressure was determined for a loading condition consisting of 7 casks plus 1 cask on the transporter with a dynamic impact factor of 1.0. This case resulted in a maximum soil bearing pressure of 3.50 ksf. All of the static load cases were within the allowable soil bearing pressure of 4.0 ksf. The maximum dynamic soil pressure, which includes earthquake

loadings, was 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 pressure. All of the dynamic load cases were below the minimum allowable dynamic soil bearing pressure.

4.2.3.5.3 Storage Pad Settlement

The relationship of major foundations to subsurface materials is contained in Section 2.6.16. Storage pad soil bearing capacity and settlement analysis is 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.

Foundation preparation for the storage pad consists of the necessary soil excavation and placement of a 4 inch concrete mud mat on the insitu soil to preclude excessive disturbance of the existing soil and its natural cohesive structure. The bottom of the mud mat and the bottom of the cask storage pad are below the specified local frost depth of 30 inches below grade in accordance with the Geotechnical section of the Storage Facility Design Criteria (Reference 22).

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.

4.2.3.5.4 Cask Stability

Cask stability ensures the storage casks will not tip over or slide excessively during a seismic event. Both the HI-STORM SAR and TranStor SAR contain a generic cask stability analysis for a selected upper bound PFSF deterministic design earthquake (0.67g vertical, 0.69g horizontal – See Section 8.2.1.1). However, the generic analyses do not consider soil-structure interaction, which can amplify seismic accelerations. Consequently, a site-specific cask stability analysis has been performed by both Holtec and SNC that demonstrates the storage casks will not tip over or slide excessively during a PFSF deterministic design earthquake. 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.

4.5 SHIPPING CASKS AND ASSOCIATED COMPONENTS

Spent fuel shipping casks are used to transport the spent fuel canisters from the originating power plants to the PFSF and later offsite. The shipping casks are designed to protect the canisters from the effects of environmental conditions, natural phenomena, and accidents in accordance with 10 CFR 71. Shipping casks are not licensed under 10 CFR 72. However, since the shipping casks are used to transport spent fuel to and from the PFSF and are part of the canister transfer process in the Canister Transfer Building, this section provides a brief summary of the shipping casks and associated components.

The shipping casks are shipped to the PFSF and shipped offsite at a later date complete with impact limiters, a shipping cradle, and tie downs. The shipping casks are shipped from the railroad mainline to the PFSF either by rail on a railroad line or by highway. Shipment by highway requires the shipping casks be transferred from the rail car to a heavy haul tractor/trailer at an intermodal transfer point. During the rail to trailer transfer, the cask and shipping components remain an integral unit under 10 CFR 71 packaging requirements. At the PFSF, the shipping cask is unloaded from the rail car or heavy haul tractor/trailer and moved to a canister transfer cell where the shipping cask is opened and the canister is removed. After the canister is unloaded, the shipping cask is resealed and sent back to the power plants for reloading of another sealed canister of spent fuel.

The shipping components addressed in this section are:

- HI-STAR shipping cask system
- TranStor shipping cask system
- Shipping cask repair and maintenance area
- Skull Valley Road / Intermodal transfer point
- Low Corridor rail line

The shipping casks and associated components are described below. Figures are provided to illustrate the systems and their function.

4.5.1 HI-STAR Shipping Cask System

The HI-STAR system is one of the shipping systems used to ship spent fuel from the originating power plants to the PFSF. The HI-STAR (Holtec International Storage, Transport, and Repository) is a spent fuel packaging design in compliance with DOE's design procurement specifications for multi-purpose canisters and large transportation casks. The HI-STAR system consists of the same sealed metal canister as used in the HI-STORM storage system, which is confined within a metal overpack or cask with impact limiters. Holtec submitted a SAR to the NRC in accordance with 10 CFR 71 for the HI-STAR system (Reference 3). The HI-STAR system components are shown on Figure 4.5-1. Details of the system and design parameters are addressed in the HI-STAR shipping SAR.

4.5.2 TranStor Shipping Cask System

The TranStor system is one of the shipping systems used to ship spent fuel from the originating power plants to the PFSF. The TranStor system is a multi-purpose canister system used for the safe storage and offsite shipping of spent nuclear fuel. The TranStor system includes a sealed metal canister, a shipping cask with impact limiters, a concrete storage cask, and a transfer cask. The canister is used in combination with the storage cask and the shipping cask components. Offsite shipping of spent fuel is performed using only the canister and the shipping cask components. SNC submitted a SAR to the NRC in accordance with 10 CFR 71 for the TranStor system (Reference 13). The TranStor system components are shown on Figure 4.5-2. Details of the system and design parameters are addressed in the TranStor shipping SAR.

4.5.3 Shipping Cask Repair and Maintenance

If shipping cask repair or maintenance activities are necessary, they will be conducted at the Operation and Maintenance Building or at a vendor designated location. No special contamination control measures are anticipated for repair or maintenance activities since the spent fuel is contained within a sealed canister and the shipping casks used for the PFSF do not enter any nuclear plant spent fuel pools and therefore, remain free of radioactive contamination.

Health physics surveys will be taken on all incoming canisters as normal receiving operations at the PFSF. In the event contamination above acceptance levels is discovered, the canister will be shipped back to the originating nuclear power plant for canister decontamination and/or spent fuel repackaging.

4.5.4 Skull Valley Road / Intermodal Transfer Point

4.5.4.1 Intermodal Transfer Point

Shipments that utilize the Skull Valley Road / intermodal transfer point are moved by the use of roads from the rail mainline to the PFSF using heavy-haul tractor/trailers. The intermodal transfer point is located 1.8 miles West of Timpie, approximately 24 miles north of the PFSF. The intermodal transfer point equipment is designed to accommodate transfer of the shipping casks from the rail car to the heavy haul tractor/trailer unit for highway shipping. The intermodal transfer point consists of rail sidings off the Union Pacific Railroad mainline, a 150 ton gantry crane, and a tractor/trailer yard area. The gantry crane is a single-failure-proof crane to preclude the accidental drop of a shipping cask even though the cask is designed to withstand such drops in accordance with 10 CFR 71. The crane is housed in a weather enclosure, which provides a clean, dry environment for transfer of the shipping cask.

The intermodal transfer point is shown on Figure 4.5-3.

4.5.4.2 Shipping Cask Heavy Haul Tractor/Trailer

Heavy haul transport tractor/trailers are used to transport the shipping cask from the intermodal transfer point to the PFSF by highway. The maximum weight of a loaded shipping cask with impact limiters and shipping cradle is approximately 142 tons, which requires the use of overweight trailers. The heavy haul tractor/trailers are designed to accommodate road conditions at the intermodal transfer point, frontage road, Skull Valley Road, and PFSF. The unit is designed to travel at low speeds and is 12 ft wide with multiple wheel sets to provide stable transport of the shipping cask.

The unit is classified as not Important to Safety since safety of the spent fuel canister is maintained by the shipping cask. A typical heavy haul transport tractor/trailer unit is shown on Figure 4.5-4.

4.5.5 Low Corridor Rail Line

4.5.5.1 Rail Line

Shipments that utilize the railroad line continue on from the rail mainline to the PFSF by rail car. A rail line will be built from the Union Pacific mainline located at Low Junction to the PFSF. The rail line is designed to standard railroad load, grade, and clearance requirements per the Union Pacific Railroad and industry standards to facilitate use of Union Pacific and standard railroad equipment.

The Low Corridor rail line is shown on Figure 4.5-6

4.7.1 Canister Transfer Building

The Canister Transfer Building is provided for physical protection and shielding of the canisters during transfer from the transportation cask to the storage cask. The Canister Transfer Building consists of the shipping cask loading/unloading bays, canister transfer cells, a 200 ton overhead bridge crane, a 150 ton semi-gantry crane, a low level waste storage room, and personnel offices/restroom areas.

Preliminary design of the Canister Transfer Building has been performed for critical areas of the structure for the most severe load combinations. Details of the analysis and design are summarized in Section 4.7.1.5.3. Final design will be completed during the detailed design phase of the project. Loads provided by the crane vendor have been incorporated in the design and will be verified during the detailed design phase of the project.

No floor drains are located in the Canister Transfer Building to preclude the possibility of contamination entering the septic system. Shallow floor sumps are located in the center of each shipping cask load/unload bay to collect water from rain and snow that may run off onto the floor from a spent fuel shipment. Collected water will be sampled to ensure no contamination is present prior to removal.

4.7.1.1 Design Specifications

The building will be designed in accordance with the Principal Design Criteria contained in Chapter 3. The Canister Transfer Building is a massive reinforced concrete structure with thick walls provided for tornado-generated missile protection and radiation shielding. The building will be designed in accordance with the provisions of ACI-349 (Reference 15).

4.7.1.2 Plans and Sections

The Canister Transfer Building is shown in Figure 4.7-1.

4.7.1.3 Function

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

Canister Transfer Building functions include:

- Load or unload spent fuel shipping casks from the heavy haul tractor/trailers.
- Provide weather and tornado proof protection for performing the canister transfer operations.
- Provide the support structure for the single failure-proof cranes required for the transfer operations.
- Provide radiological shielding during the transfer operation.
- Store potential low-level radioactive waste from health physics surveys.
- Provide storage and laydown space for transfer and shipping equipment.
- Provide a staging area for storage casks.

4.7.1.4 Components

The major components that comprise the Canister Transfer Building are the cask loading/unloading bays, three canister transfer cells, the 200 ton overhead bridge crane, the 150 ton semi-gantry crane, crane runway girders and their supports, cask transporter bay, tornado-missile barriers, low level waste storage room, radiation shield

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 transfer cell walls 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 connected to the shipping cask after it is moved into the transfer cell. The struts are attached to the transfer cask when it is placed on top of the shipping cask or storage cask. Each cask utilizes two struts that provide restraint in the x and y directions.

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 transfer cell wall.

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. The preliminary design has considered

the worst loading case and areas of the structure that are considered to be the most likely to cause problems have been designed. During the detailed design phase, all load cases as described in Chapter 3 and all areas will be addressed in detail.

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
- Bridge crane and semi-gantry crane loads
- Normal crane handling loads and transfer operations
- Normal wind load
- Concrete storage cask, transfer cask, and shipping cask loads
- Vehicle loads

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 SSCs 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 building layout and specifically designed labyrinths will prevent tornado missiles entering through door or ventilation openings in the walls and roof from impacting or damaging the fuel canisters, single failure proof cranes and their supports, or other SSC's housed within the building.

F. Earthquake

The Canister Transfer Building has been analyzed for the PFSF deterministic design earthquake (0.67g vertical, 0.69g horizontal – See Section 8.2.1.1). 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).

H. Lightning

The Canister Transfer Building is approximately 77 feet tall and is a possible lightning target. The Canister Transfer Building is designed with lightning protection features in accordance with NFPA 780.

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 using the DYNRE 7 module of the STARDYNE computer program (Reference 36). The time histories were first baseline corrected and then verified to meet the requirements of ASCE-4. This was accomplished using the Stone & Webster computer programs TIMHIS6 (Reference 37) for calculating the response spectra magnitude and CORREL (Reference 38) to verify statistical independence. The analysis is documented in calculation SC-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 the PFSF Storage

Facility Design Criteria (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. Two additional load cases were evaluated to account for potential variations in soil properties, including a 33% reduction in the soil shear modulus and a 50% increase in the soil modulus. 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 lumped mass model and 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 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 Bearing Capacity and Settlement

In addition to a finite element soil-structure interaction analysis described above, a conventional static and dynamic bearing capacity analysis of the building mat foundation was performed. The analysis was 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 were used in this analysis. 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 static, seismic uplift, and horizontal seismic in E-W and N-W directions was the most critical load case. This load case resulted in actual maximum foundation soil bearing pressure of 2.8 kips per square foot (ksf), compared with an ultimate bearing capacity of 4.5 ksf, for this load case. The resulting factor of safety against a bearing capacity failure for this load case is 1.6, compared with the minimum 1.1 factor of safety required for seismic load cases. 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. The bearing capacity analysis is contained in calculation G(C)-13 (Reference 48).

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). A stability analysis of the building is performed in calculation SC-9 (Reference 50) to check the building for overturning and sliding during a seismic event. The building is found to be stable.

4.7.2 Canister Transfer Cranes

The Canister Transfer Building houses two cranes, a 200 ton overhead bridge crane and a 150 ton semi-gantry crane. The cranes are provided for the purpose of loading and unloading shipping casks off or on the heavy haul tractor/trailers and transferring spent fuel canisters between the shipping cask and the storage casks. The 200 ton bridge crane can be used for both load/unload and transfer operations. The semi-gantry crane can only be used for transfer operations and provides additional crane availability because of the time requirements involved in the transfer operations.

The PFSF canister transfer cranes are designed by Ederer Incorporated. The cranes utilize a patented hoisting safety system called the X-SAM (eXtra Safety And Monitoring) system. Ederer submitted a Topical Report to the NRC for the X-SAM system (Reference 51). The bridge and semi-gantry cranes are incorporated into the Ederer Topical Report by References 52, 53, 54, and 55.

4.7.2.1 Design Specifications

The canister transfer cranes meet the requirements of the Design Criteria contained in Chapter 3, which requires the cranes be designed in accordance with ASME NOG-1 (Reference 32) and be single-failure-proof in accordance with NUREG-0554 (Reference 33).

During the detailed design stage, design requirements will be specified that provide for the performance of testing, inspection, and maintenance activities on the cranes in accordance with 10 CFR 72.122(f). Inspection and acceptance of the cranes will be performed during fabrication, in accordance with the QA Program described in Chapter 11, to ensure that the design requirements are satisfied.

The functional parameters of the overhead bridge crane are as follows:

Capacity:	Main hoist - 200 tons (Maximum Critical Load)
	Auxiliary hoist - 25 tons
Hoist type:	Main hoist - Single-failure-proof
	Auxiliary hoist - Single-failure-proof
Hoist ropes:	Main hoist - Carbon steel
	Auxiliary hoist - Carbon steel
Bridge span:	65'-0"
Length of runway:	260'-0"
Top of rail elev:	70'-6" above floor slab
Bridge/trolley:	40/25 fpm (Maximum speed)
Motion controls:	DC hoist and traverse
Operator controls:	Radio remote and pendant

The functional parameters of the semi-gantry crane are as follows:

Capacity:	Main hoist - 150 tons (Maximum Critical Load)
	Auxiliary hoist - 25 tons
Hoist type:	Main hoist - Single-failure-proof
	Auxiliary hoist - Single-failure-proof
Hoist ropes:	Main hoist - Carbon steel
	Auxiliary hoist - Carbon steel
Bridge span:	35'-0"
Length of runway:	180'-0"
Top of rail elev:	55'-9" above floor slab
Bridge/trolley:	50/30 fpm (Maximum speed)
Motion controls:	DC hoist and traverse
Operator controls:	Radio remote and pendant

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 are 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:

The crane operations are designed not to exceed the handling loads (live loads) assumed in the HI-STORM and TranStor SARs. The analysis assumes the 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).

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).

The weight of loaded concrete storage cask is approximately 177 tons (HI-STORM system) and 155 tons (TranStor system).

The overhead bridge crane capacity is 200 tons, which exceeds the heaviest load of 177 tons.

4.7.2.5.2 Maximum Loads Applicable To Both Overhead Bridge Crane And Semi-Gantry Crane

The weight of transfer cask with a loaded canister and transfer cask lifting yoke is approximately 121 tons (HI-TRAC system) and 109 tons (TranStor system).

The semi-gantry crane capacity is 150 tons, which exceeds the heaviest load of 121 tons.

4.7.2.5.3 Seismic Analysis

The PFSF overhead and semi-gantry cranes have been seismically analyzed in accordance with ASME NOG-1 to ensure they will remain in place and support the load during and after a seismic event. The analyses were performed for both cranes by Anatech Corporation to qualify the crane designs for the PFSF deterministic design earthquake (0.67g vertical, 0.69g horizontal – See Section 8.2.1.1). The analysis methods, modeling, and results for the 200 ton bridge crane and 150 ton semi-gantry crane are documented in References 56 and 57 respectively. The maximum stresses were calculated for the major structural components through response spectrum analyses using the amplified response spectra at the crane rail elevation in the Canister Transfer Building. The calculated normal and shear stresses for all components were within allowables as defined in ASME NOG-1.

A. Analysis Method

For the analysis, the base motion was the design response spectrum given in terms of acceleration versus frequency at 4% of critical damping for 3 component directions at the building elevation supporting the crane railways. The basic assumptions for this type of analysis were that the response is linear and that it is relative to the base motion. The eigenmodes, frequencies, and modal participation factors were first extracted for the finite element model of the structural system. The peak modal response of the "generalized" variables in frequency space was calculated from the given input response spectrum (acceleration as a function of frequency and damping) and the modal participation factors. The corresponding peak physical response was then calculated for each natural frequency mode through the corresponding eigenvector. These peak physical responses for each natural mode were then combined to estimate the total peak response of the variable. Since the peak response in the different modes will not typically occur at the same time, the combination into a peak value is conservative.

The method used to combine the individual modal response was the square root of the sum of the squares (SRSS) method described in USNRC Regulatory Guide 1.92 (Reference 58) for closely spaced modes. The modes were determined to be closely spaced since the frequencies were within 10 percent of the lower value. The SRSS method was modified by adding twice the absolute value of the product of the peak modal response from each pair of modes to the sum of the squares of all the modal peak values. Finally, these peak responses were combined for the different component directions using a SRSS combination. The resulting peak values from the seismic loads were then algebraically added to static values to compare with design allowables as specified in ASME NOG-1 for seismic qualification.

The above procedure for response spectrum analysis is fully implemented in the ABAQUS/Standard general purpose finite element program (Reference 59). The program is used extensively in the nuclear industry for this type of analyses. The program allows input of 3 orthogonal base motion response spectrums and options for summing the modal contributions and the component contributions, which are automatically calculated separately.

B. Load Cases

The cranes were analyzed with six load positions that would produce the maximum forces on the system; trolley at end of travel, trolley at ¼ span, and trolley at mid-span, with hook up and down at each position. Stresses were calculated on the main and auxiliary hoists for each of the three positions considering no load and credible critical load conditions.

For the semi-gantry crane, the trolley positions are not symmetric with respect to the midspan since the crane is supported by a wall on one end and by the gantry on the other end. The ¼ span trolley position was evaluated relative to the trolley at the end of travel in each direction to determine the trolley location that produces the maximum stress for the site specific response spectra. The three positions included trolley at wall end, trolley at mid-span, and trolley at gantry leg end.

C. Design Allowables

The allowable design criteria for the seismic qualification is established in ASME NOG-I, Section 4300.

D. Models

The finite element models used for seismic qualification of the overhead bridge crane and semi-gantry crane are illustrated in References 56 and 57 respectively. The models show the trolley at midspan along the bridge girders with the hook loaded and in the up position. Similar models are used for the other trolley positions and for hook up and down in each position. The models use 3-node Timoshenko beam elements for the structural members.

E. Properties and Mass Distribution

The response spectra analysis assumed linear response. For all structural members, an elastic modulus of 29E6 psi and Poisson's ration of 0.318 were used. The beam section properties were computed from the cross-section shapes and input directly into the computer model. The gantry leg strut bracing uses an 8 inch diameter extra strong pipe section. Truss elements were used to model the hoist rope. A lumped mass for the payload and lower block is attached to the end of the hoist rope for the main hoist and auxiliary hoist cases.

F. Response Spectra

The analysis is based on the response spectra provided in terms of acceleration versus frequency for 4% damping at elevation 100 ft and 170 ft in the Canister Transfer Building. The gantry legs are mounted on rails at elevation 100 ft and 155 ft 3 in. In the ABAQUS implementation of the response spectrum analysis technique, only one response spectra for each of 3 orthogonal directions may be input. The approach used was to include the support wall in the model and apply the 100 ft acceleration spectra at the base of the wall and at the gantry leg support.

H. Results

The maximum stresses for the various components were developed for different trolley positions and loading conditions. For all trolley positions, the maximum critical load on the main hoist with the hook in the up position is the worst load case for both cranes. The no-load and auxiliary hoist load conditions for the overhead bridge crane typically develop stresses less than 50 percent of allowables. The no-load and auxiliary hoist load conditions for the semi-gantry crane typically develop stresses less than 60 percent of allowables except for the gantry legs, which are typically stressed to 80 percent of allowable. The maximum stresses are presented in Section 4.2 of References 56 and 57.

4.7.2.5.4 Single-Failure-Proof Analysis

The PFSF cranes hoist systems are designed in accordance with ASME NOG-1 with single failure proof features so that any credible failure of a single component will not result in the loss of capability to stop and hold the load. The Ederer X-SAM crane design includes several safety systems to ensure single-failure-proof requirements are met. These are addressed in the X-SAM Generic Topical Report (Reference 51), Section III.B, which provides the single-failure-proof analysis for the X-SAM crane design. The safety systems are limited to the hoist and brake for the trolley and bridge and comply with the guidelines in NUREG-0554 (Reference 33). The safety systems are designed to allow the cranes to safety withstand failures caused by overload, load hangup, two blocking, hoist drive train failure, drum support failure, overspeed, loss of power during a critical lift, hoist control failure, off center lifts, holding brake failure, and cable failure.

The X-SAM crane consists of three types of safety systems; the hoist integrated protective system (HIPS), conventional hoist safety system, and the balanced dual

reeving system. HIPS is a series of special hoist safety systems and subsystems that have been integrated to monitor problems with the hoist, limit the amount of abuse the hoist can be subjected, protect the hoist against abnormal abuse, and report abuse for prevention. The HIPS includes the energy absorbing torque limiter (EATL), emergency drum brake system (EDBS), failure detection system, drum safety structure, wire rope protection, and emergency stop button.

The EATL is incorporated into the hoist gear case and acts as an energy absorber and torque limiter. If any off-normal condition occurs, such as two blocking, the EATL would limit the maximum load imposed on the reeving system and would dissipate rotational kinetic energy, keeping stresses at known safe levels during and after shutdown of the hoisting system. The EATL is set to slip at 130% of rated load torque, setting the emergency brake on the rope drum. The EDBS provides an independent means for reliability and safety stopping and holding the load following a failure in the hoist machinery. The failure detection system detects a loss of mechanical continuity in the hoist machinery and actuation of the EATL as well as improper rope spooling, reeving continuity, and drum overspeed. The drum safety structure ensures that a shaft or bearing failure would not allow the drum to disengage from it drive gear or EDBS. The wire rope protection is provided by the design of the hoist, which ensures that the hoist is able to withstand two blocking without mechanically damaging the wire rope. The emergency stop button, located at the control station, removes power from the crane and sets the EDBS as soon as the load begins to lower.

The second type of safety system is the conventional hoist safety system, which features safety systems that are typically installed on conventional overhead cranes. These systems include the dual upper limit switches, overload sensing and indication, load control system, and high speed holding braking. The HIPS protects against maloperation of these systems.

The third type of safety system is the balanced dual reeving system. The HIPS provides the primary protection of the reeving by preventing overloads and mechanical damage of the cables and the balanced dual reeving system provides further protection against loss of the load in the event of a cable failure. This system includes dual reeving, the hydraulic load equalization system, and the wire rope

The design margin for the wire rope system used on the PFSF cranes exceeds the requirements in the Generic Topical Report. Each wire rope system is designed with a minimum safety factor of 10:1 of ultimate in accordance with NUREG-0612 (Reference 35), Section 5.1.6, paragraph 1(A) and ANSI N14.6 (Reference 34), Section 7.2.1.

The auxiliary hoist is also designed as single-failure-proof and has the same X-SAM features as the main hoist to prevent two blocking and to protect the crane and load from single failures.

Amendments to Ederer's General Topical Report referencing the PFSF overhead and semi-gantry cranes are documented in References 52 through 55 respectively.

4.7.2.5.5 Crane Design

The bridge crane is designed with double bridge girders spanning 65 ft supported along one end on rails 70 ft above the building floor. The bridge girders are welded plate box sections rigidly connected to box section end ties that serve as equalizing sills for the bridge trucks. 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 17 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 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.

4.7.3 HI-STORM Transfer Equipment

The HI-STORM transfer equipment consists of a metal transfer cask (HI-TRAC), HI-TRAC lifting trunnions, shipping cask and transfer cask lift yokes, canister downloader, canister lift cleats, and HI-STORM lifting lugs.

4.7.3.1 Design Specifications

The HI-TRAC transfer cask, trunnions, lift yokes, canister downloader, canister lift cleats, and storage cask lifting lugs are designed as special lifting devices in accordance with ANSI N14.6 (Reference 34) and NUREG-0612 (Reference 35).

4.7.3.2 Plans and Sections

The transfer cask assembly is shown in Figure 4.7-2.

4.7.3.3 Function

The function of the HI-TRAC transfer cask is to provide a shielded lifting device for carrying the canister between the HI-STAR shipping cask and the HI-STORM storage cask. The function of the lifting yokes is to provide a lifting interface between the crane and the shipping cask or transfer cask. The function of the canister lift cleats is to provide a means to lift the canister. The function of the HI-STORM storage cask lifting lugs is to provide a means to lift the storage cask.

in HI-STORM SAR Table 4.5.4. The table shows a summary of temperature differences in the basket periphery and canister shell between the top and bottom of the canister. The table indicated temperature differences between the top and bottom section of the basket periphery of approximately 238° F, and approximately 166° F between top and bottom sections of the canister shell. The temperature gradients were evaluated to determine the cask and canister thermal growths and shown to be minimal. The temperature gradients were also used to calculate thermal stresses in the canister, which were shown to be within code allowables and therefore meet the PFSF design criteria in Section 3.2.6 for thermal loads.

C. Tornado Winds and Missiles

Evaluation of the transfer cask for tornado wind or missile is not required since the canister transfer operations are conducted within the Canister Transfer Building.

D. Earthquake

The transfer cask has been evaluated for stability during a PFSF deterministic design earthquake (0.67g vertical, 0.69g horizontal – See Section 8.2.1.1) when in the stacked cask arrangement. The stacked cask arrangement occurs when the transfer cask is resting on top of the storage cask. It was concluded that during transfer operations, it is necessary to ensure the transfer cask is supported throughout the transfer operation to prevent the cask from toppling during a seismic event. Therefore, facility procedures will ensure that the transfer cask be secured to the cell walls with struts when in the stacked arrangement to preclude a cask toppling accident.

E. Fire

Fires concerning the HI-TRAC transfer cask are addressed in HI-STORM SAR Section 11.2.4. The HI-TRAC was analyzed for a fire around the cask of 50 gallons of

combustible fuel. The fire had a duration of less than 5 minutes. A bounding cask temperature rise of less than 16° F per minute was determined from the combined radiant and convection heat input to the cask. As a result, the fuel cladding was shown not to exceed the accident condition fuel cladding temperature limits. As shown in Section 8.2.5, the only fuel source near a loaded HI-TRAC transfer cask would be diesel fuel from the cask transporter, whose fuel tanks have a capacity of 50 gallons, which would fuel a fire for a duration of less than 5 minutes, as analyzed. In addition, it is anticipated any fires would be put out by the Canister Transfer Building sprinkler system.

The elevated temperatures from a fire could cause the pressure in the transfer cask water jacket to increase and cause the overpressure relief valve to vent steam to the atmosphere. However, this would not have any adverse affect on systems classified as Important to Safety and would vent less water and cause less disruption than the sprinkler system. Therefore the HI-TRAC design and building provisions meet the PFSF design criteria in Section 3.2.6 for accident-level thermal loads in accordance with 10 CFR 72.122(c).

4.7.3.5.2 Thermal Design

The thermal analysis for the HI-TRAC transfer cask is described in HI-STORM SAR Section 4.5.1. The analysis uses the same approach as the HI-STORM storage cask/canister thermal analysis (Section 4.2.1.5.2) and was performed using the ANSYS finite element computer code (Reference 10).

Heat generated in the fuel assemblies is transported to the shell of the canister, in the manner described in Section 4.2.1.5.2. From the outer surface of the canister, heat is transported across a total of six concentric layers, representing the air gap, the HI-TRAC inner steel shell, the lead shielding, the outer steel shell, the water jacket, and

are required, therefore the hoist rings meet the NUREG-0612 requirements for redundancy.

The TranStor storage cask is designed to be lifted using four lifting lugs, which are steel plates integrally welded to the concrete steel reinforcing. The lifting lugs are not classified as Important to Safety since the cask lift height is limited by the Technical Specifications but are designed with a minimum factor of safety of 3 on material yield strength and 5 on material ultimate strength.

B. Tornado Winds and Missiles

Evaluation of the TranStor transfer cask for tornado wind or missile is not required since protection of the transfer operation from the effects of tornado wind and tornado missiles is provided by the Canister Transfer Building structure.

C. Earthquake

The transfer cask has been evaluated for stability during a PFSF deterministic design earthquake (0.67g vertical, 0.69g horizontal - See Section 8.2.1.1) when in the stacked cask arrangement. The stacked cask arrangement occurs when the transfer cask is resting on top of the shipping or storage cask. Since the TranStor system operation requires the crane be removed from the transfer cask in order to hook to the canister for lifting, a condition would exist where the transfer cask could topple during a seismic event. Therefore, the transfer cask is designed to be secured to the cell walls with struts when in the stacked arrangement, prior to removing the crane to preclude a cask toppling accident.

D. Fire

As shown in Section 8.2.5, fires near a loaded transfer cask would have a small effect on the canister temperature because of the short duration of the fire accidents. The only fire fuel source would be diesel fuel from the cask transporter. In addition, it is anticipated any fires would be put out by the Canister Transfer Building sprinkler system. Therefore the transfer cask design and building provisions meet the PFSF design criteria in Section 3.2.6 for accident-level thermal loads in accordance with 10 CFR 72.122(c).

4.7.4.5.2 Thermal Design

The thermal analysis performed for the transfer cask is described in TranStor SAR Section 4.4.1.3. The analysis uses the same approach as the TranStor storage cask/canister thermal analysis evaluation (Section 4.2.2.5.2) and was performed with the ANSYS thermal finite element code (Reference 10). The transfer cask geometry and materials were accurately modeled in detail, while the canister is treated simply as a 26 kW cylindrical heat source, with only the canister shell modeled in detail. Heat is transferred from the canister shell to the inner steel liner of the TranStor transfer cask, then conducted through this inner liner, the lead gamma shield, the solid neutron shield material, and the steel outer liner. The major difference between this case and that of the storage cask is the absence of convective air flow along the canister exterior. For conservatism, only the steel fins were considered in determining the effective conductivity of the neutron shield region, with no credit taken for heat conduction by the neutron shield material. A key objective of this analysis was to determine the highest temperature of the canister shell, which is used for thermal evaluation of the canister internals and fuel during transfer operations.

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

36. STARDYNE (R)-4.10/windows, MAR 01/94, Titan Corp.

37. TIMHIS6, Stone & Webster computer program, ST-239, Ver-01, Level 01.

38. CORREL, Stone & Webster computer program, ST-308, Ver-00, Level 02.

39. PFSF Calculation No. 05996.02 SC-3, Development of Artificial Time Histories, Revision 0, Stone & Webster.

40. PFSF Storage Facility Design Criteria, Revision 2, Stone & Webster.

41. REFUND, Stone & Webster computer program, ST-232, Ver-0, Level 1.

42. PFSF Calculation No. 05996.02 SC-4, Development of Soil Impedance Functions for Canister Transfer Building, Revision 0, Stone & Webster.

43. FRIDAY, Stone & Webster computer program, ST-243, Ver-02, Level 01.

44. PFSF Calculation No. 05996.02 SC-5, Seismic Analysis of Canister Transfer Building, Revision 0, Stone & Webster.

45. ANSYS computer program, Version 5.4, Swanson Analysis Systems, Inc.

46. PFSF Calculation No. 05996.02 SC-6, Finite Element Analysis of Canister Transfer Building, Revision 0, Stone & Webster.

47. PFSF Calculation No. 05996.02 SC-7, Design of Reinforcing Steel for Canister Transfer Building, Revision 0, Stone & Webster.

48. PFSF Calculation No. 05996.02 G(C)-13, Allowable Bearing Capacity of the Canister Transfer Building Supported on a Mat Foundation, Revision 0, Stone & Webster.

49. PFSF Calculation No. 05996.02 G(C)-14, Static Settlement of the Canister Transfer Building Supported on a Mat Foundation, Revision 0, Stone & Webster.

50. PFSF Calculation No. 05996.02 SC-9, Stability of Canister Transfer Building, Revision 0, Stone & Webster.

51. Generic Licensing Topical Report for Ederer's Nuclear Safety Related eXtra Safety And Monitoring (X-SAM) Cranes, EDR-1(NP)-A, Ederer Incorporated, Revision 3, October 1982.

52. Appendix B Supplement to Generic Licensing Topical Report EDR-1, Summary of Facility Specific Crane Data Supplied by Ederer Incorporated for PFSF, 200/25 Ton Bridge Crane, Revision 0, November 1998.

53. Appendix C Supplement to Generic Licensing Topical Report EDR-1, Summary of Regulatory Positions to be Addressed by Applicant for PFSF, 200/25 Ton Bridge Crane, Revision 0, November 1998.

54. Appendix B Supplement to Generic Licensing Topical Report EDR-1, Summary of Facility Specific Crane Data Supplied by Ederer Incorporated for PFSF, 150/25 Ton Semi-gantry Crane, Revision 0, November 1998.

55. Appendix C Supplement to Generic Licensing Topical Report EDR-1, Summary of Regulatory Positions to be Addressed by Applicant for PFSF, 150/25 Ton Semi-gantry Crane, Revision 0, November 1998.

56. Seismic Qualification Analysis 200 Ton Bridge Crane, PFSF, No. ANA-QA-147, Anatech Corporation, Revision 0, November 1998.

57. Seismic Qualification Analysis 150 Ton Semi-gantry Crane, PFSF, No. ANA-QA-148, Anatech Corporation, Revision 0, November 1998.

58. Regulatory Guide 1.92, Combining Modal Responses and Spatial Components in Seismic Response Analysis, Revision 1, February 1976.

59. ABAQUS/Standard, Version 5.7, User Manual, Example Problem Manual, and Theory Manual, Hibbitt, Karlsson, & Sorensen, Inc., Pawtucket, RI, 1997.

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TABLE 4.2-8

DYNAMIC PAD ANALYSIS MAXIMUM RESPONSE VALUES

(based on PFSF deterministic design earthquake – See Section 8.2.1.1)

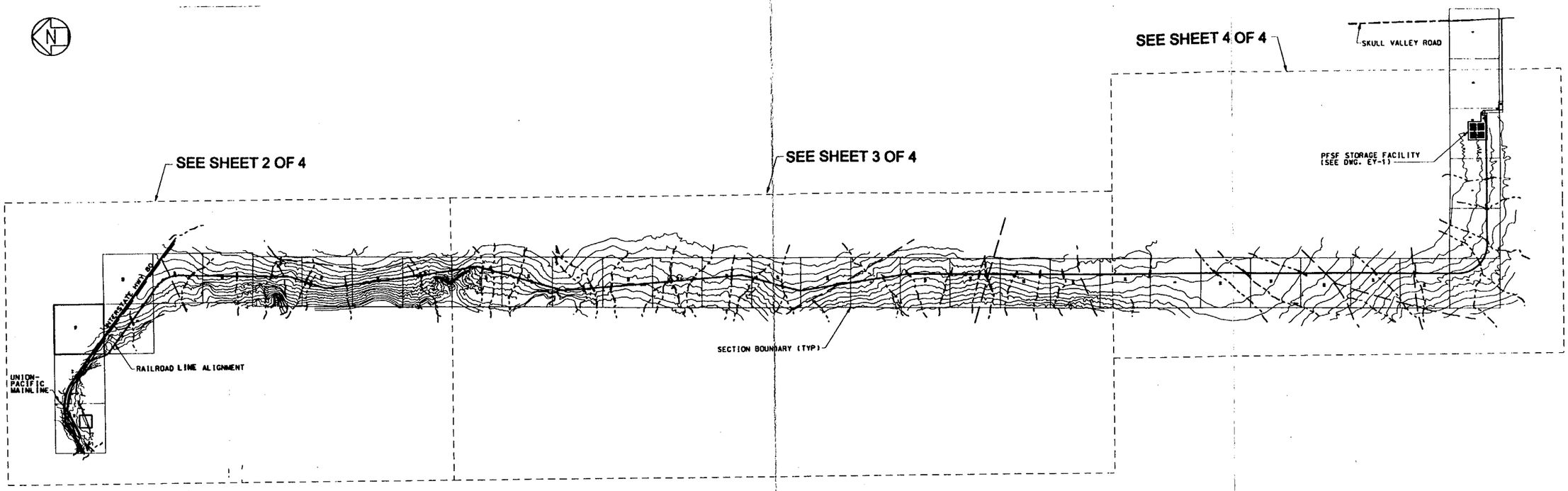
PFSF DETERMINISTIC DESIGN EARTHQUAKE LOADING	MAXIMUM MOMENT (k-ft/ft)	MAX. SHEAR FORCE (k/ft)	MAXIMUM SOIL PRESSURE (k/ft^2)	MAX. HORIZONTAL TOTAL SOIL REACTION (kips)	
				Longitudinal	Transverse
2 Casks	344.3	76.3	3.34	670	730
4 Casks	132.6	25.2	2.10	1,195	910
8 Casks	114.0	27.7	2.80	1,330	2,030

TABLE 4.7-1

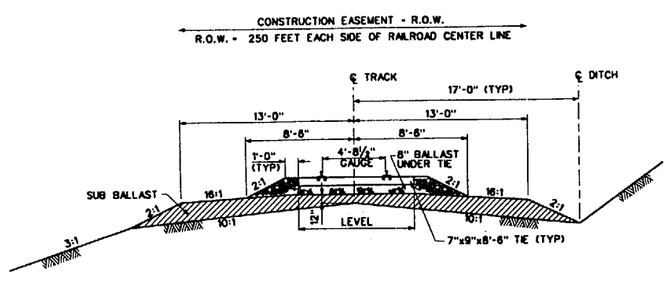
PHYSICAL CHARACTERISTICS OF THE HI-TRAC TRANSFER CASK

PARAMETER	VALUE
Inside Diameter	68.75 inches
Outside Diameter	94.625 inches
Height	199.25 inches
Materials of Construction	Steel (inner and outer shell) Lead (gamma shield) Water (neutron absorber)
Weight (empty)	151,963 lb
Maximum Working Dose Rate[1] (1 meter from surface) Side	49 mrem/hr

1. Dose rates are based on HI-STORM design basis fuel.



KEY PLAN



LOW RAILROAD LINE
TYPICAL SECTION

NOT TO SCALE

LEGEND:

- DRAINAGE PATH
- EXISTING CONTOUR
- EXISTING DIRT ROAD

APERTURE CARD

Also Available on Aperture Card

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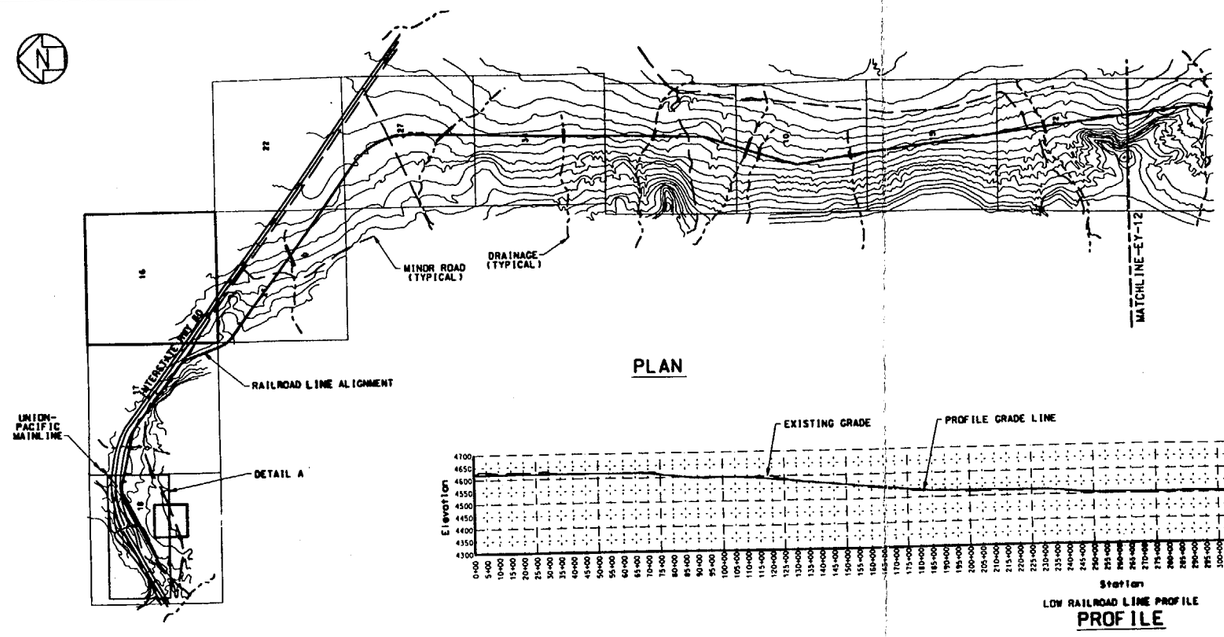
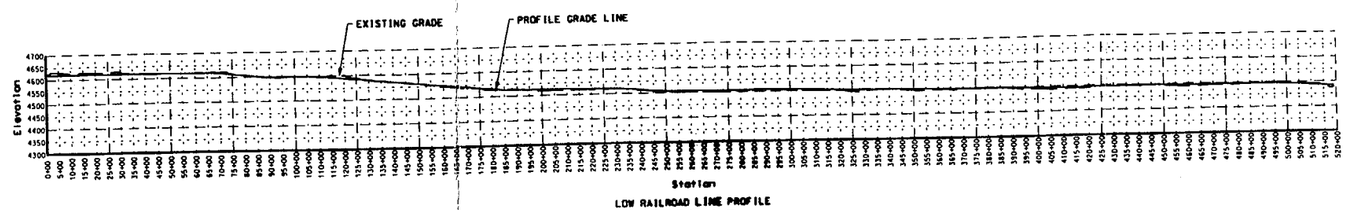
Figure 4.5-6 (Sheet 1 of 4)

LOW CORRIDOR RAIL LINE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

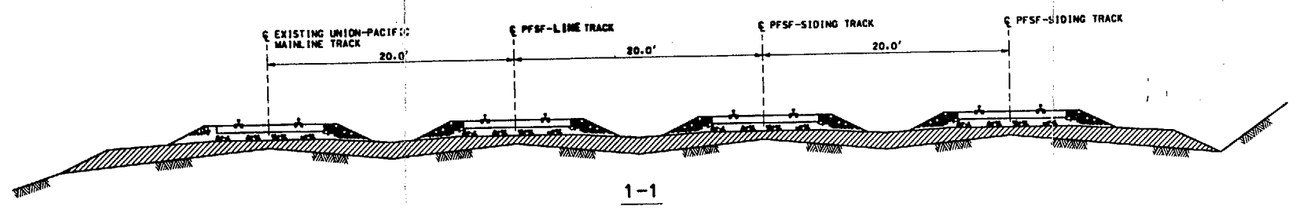


PLAN

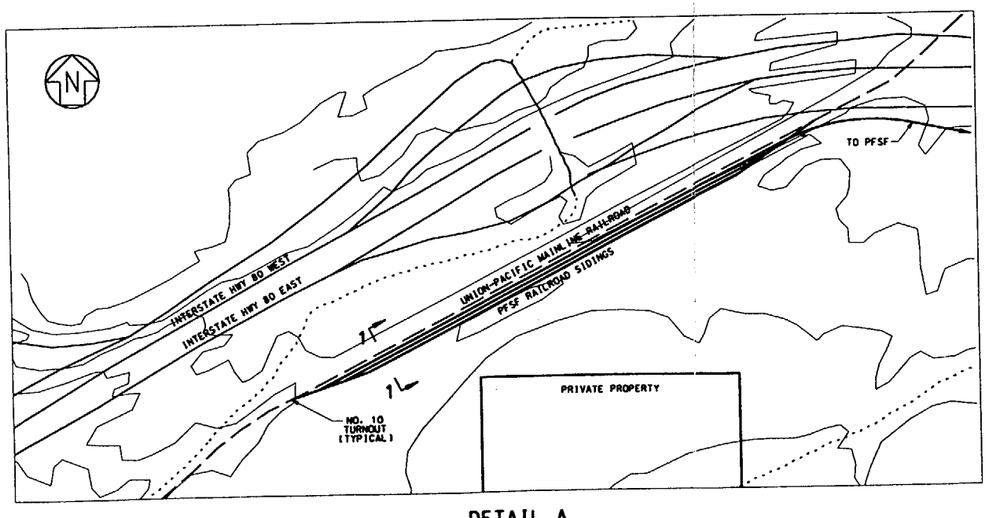


LOW RAILROAD LINE PROFILE

PROFILE



1-1
N.T.S.



DETAIL A
SCALE: 1" = 500'

APERTURE CARD
Also Available on Aperture Card

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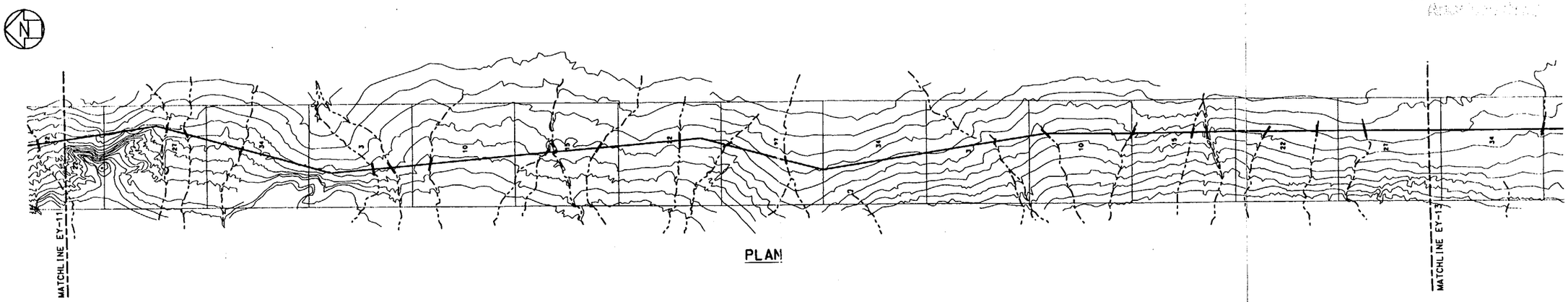
Figure 4.5-6 (Sheet 2 of 4)

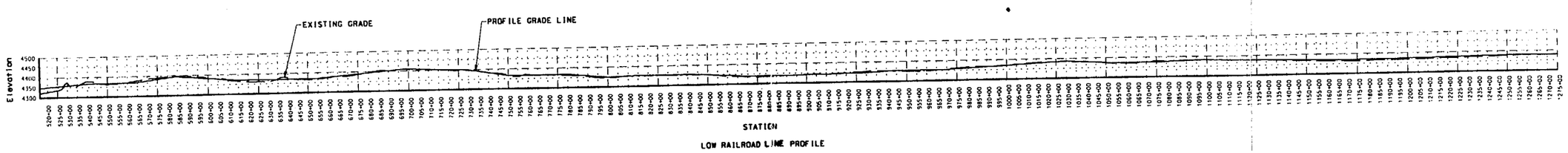
LOW CORRIDOR RAIL LINE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

APERTURE CARD



PLAN



PROFILE

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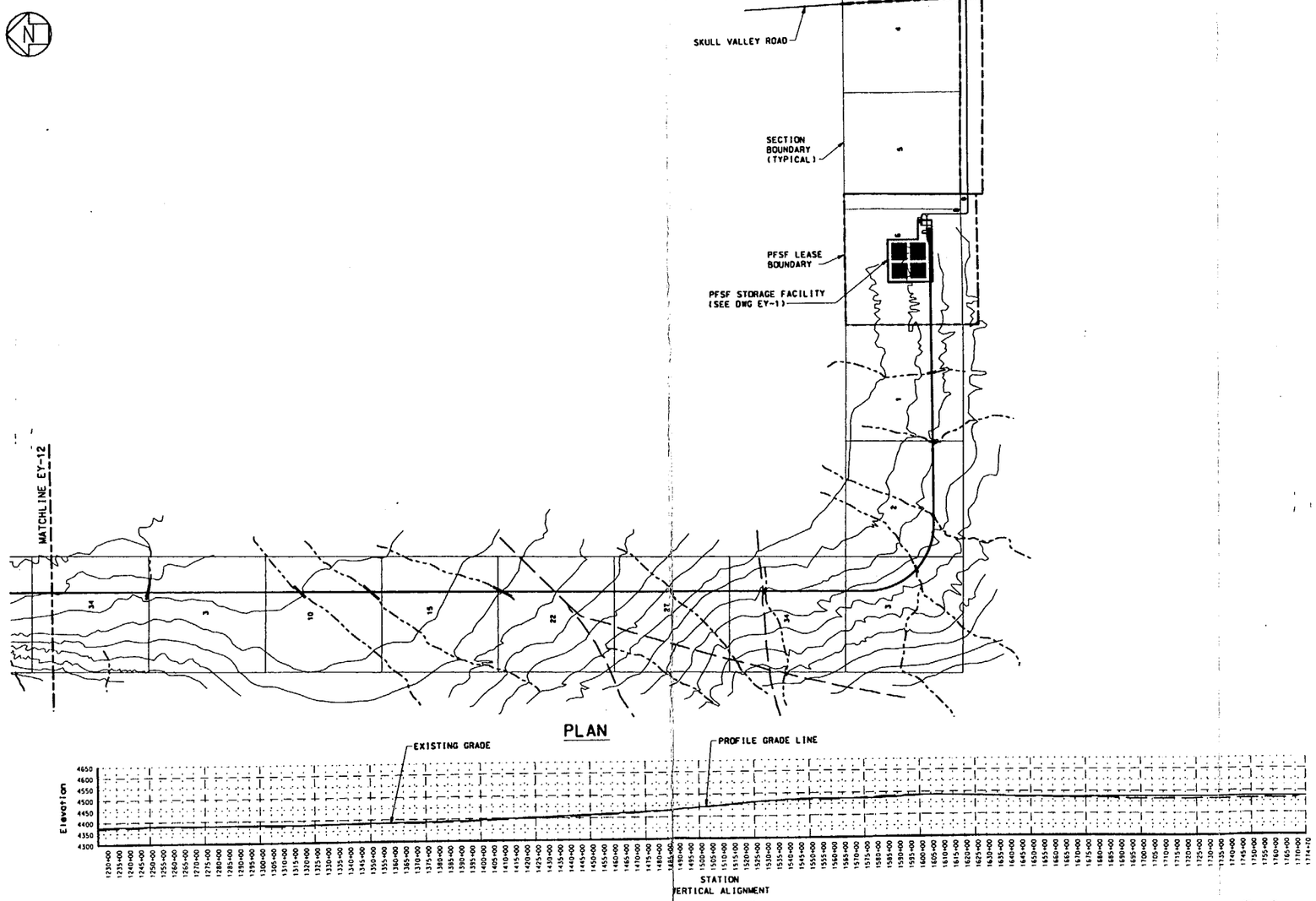
Figure 4.5-6 (Sheet 3 of 4)

LOW CORRIDOR RAIL LINE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



APERTURE CARD
Also Available on Aperture Card

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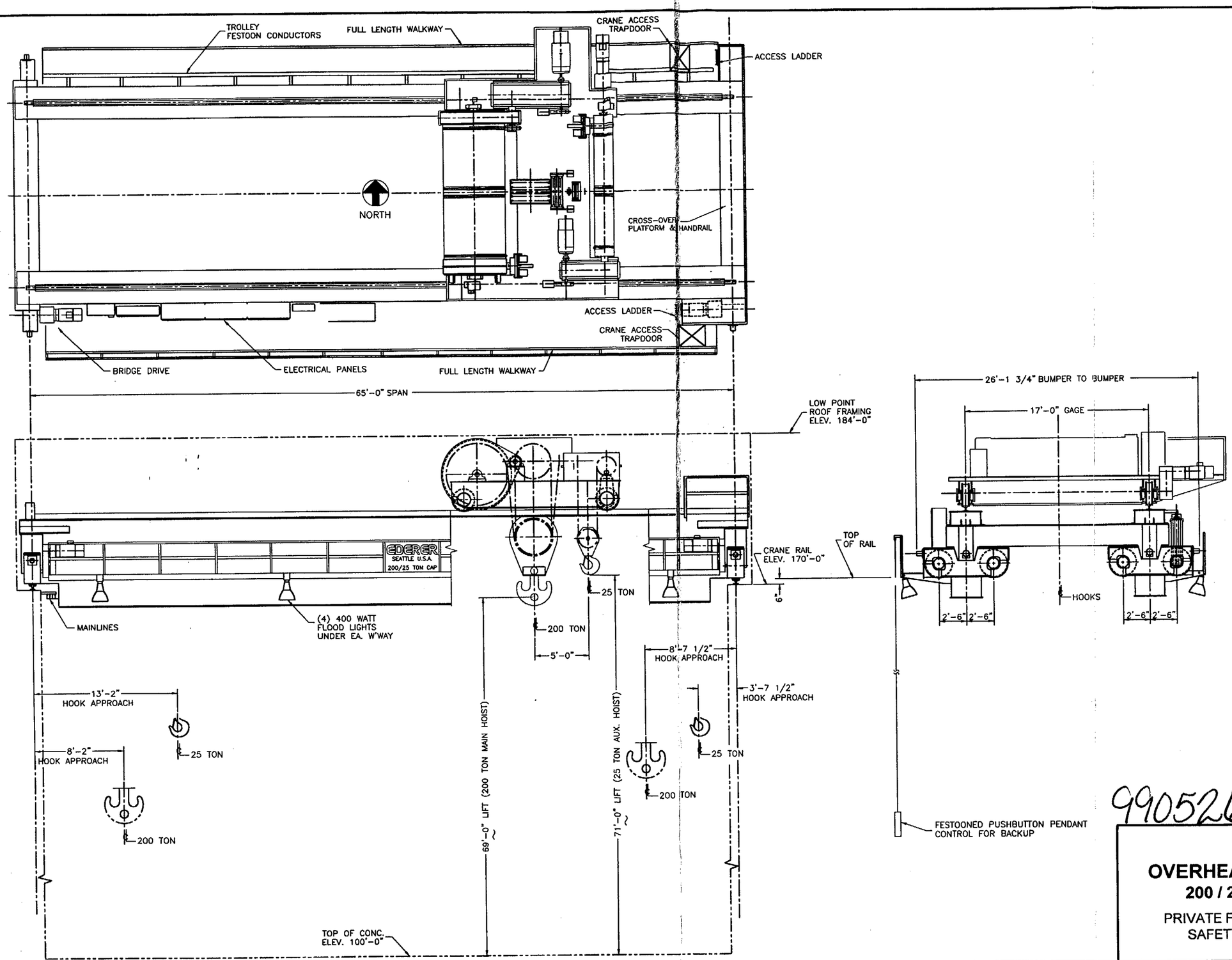
Figure 4.5-6 (Sheet 4 of 4)

LOW CORRIDOR RAIL LINE

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



APERTURE CARD
Also Available on Aperture Card

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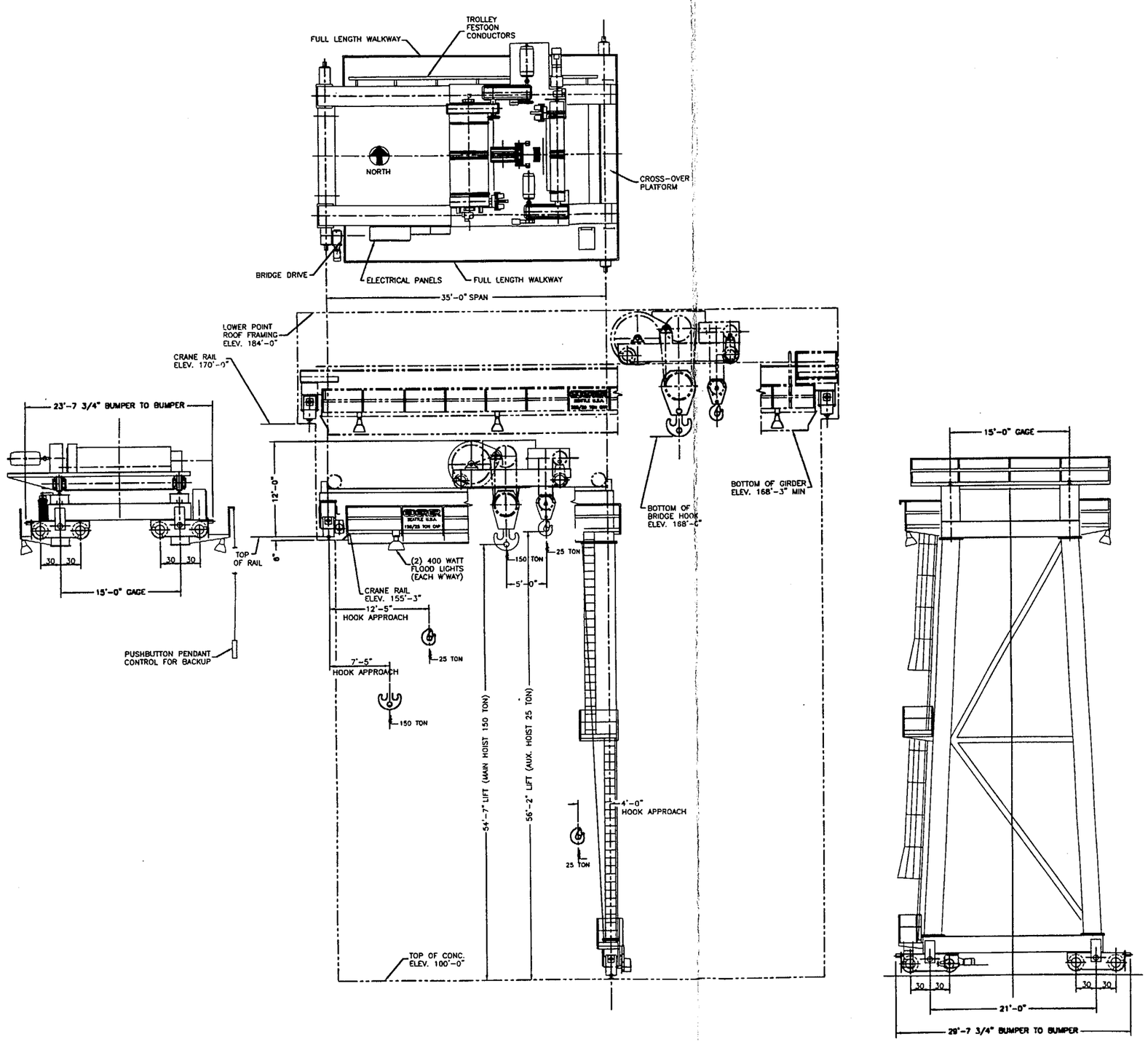
Figure 4.7-5

OVERHEAD BRIDGE CRANE
200 / 25 TON CAPACITY

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT



APERTURE CARD
Also Available on Aperture Card

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Figure 4.7-6

SEMI-GANTRY CRANE
150 / 25 TON CAPACITY

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

The canister is then backfilled with inert helium gas and the drain and fill ports are welded closed, thereby sealing the canister. The outer surfaces of the transfer cask are then checked for surface contamination and decontaminated, if necessary.

The sealed canister is transferred from the transfer cask to a shipping cask, and the shipping cask closure bolted in place. The shipping cask is then loaded onto the shipment vehicle, fitted with impact limiters to protect the shipping cask during transportation, and shipped to the PFSF.

5.1.2 Operations Between the Originating Nuclear Power Plant and the PFSF

The shipping cask, containing the sealed spent fuel canister, is shipped by rail to the PFSF. During transportation, the shipping cask provides a complete confinement barrier for the canister that is capable of withstanding any accident that could occur. The shipping cask is fitted with impact limiting devices for additional protection during transit. The offsite transportation system is licensed in accordance with the requirements of 10 CFR 71, "Packaging and Transportation of Radioactive Material", and complies with the requirements of 49 CFR 171, "General Information, Regulations, and Definitions", 49 CFR 172, "Hazardous Materials Tables and Hazardous Materials Communications Regulations", 49 CFR 173, "Shippers - General Requirements for Shipments and Packages", 49 CFR 174, "Carriage by Rail", and 49 CFR 177, "Carriage by Public Highway".

5.1.3 Operations Between the Railroad Mainline and the PFSF

The PFSF is located approximately 24 miles south of the rail mainline and currently does not have rail access. The shipping casks are shipped to the PFSF by rail to either a new rail line located at Low Junction, where the shipping casks remain on the rail car and are transported 32 miles directly to the PFSF, or to an intermodal transfer point

located 1.8 miles West of Timpie, where the loaded shipping casks are transferred from the rail car to a heavy haul tractor/trailer for transport via highway the remaining 26 miles to the PFSF. At the intermodal transfer point are rail sidings, a single-failure-proof gantry crane, and a weather enclosure over the crane.

5.1.4 Operations at the PFSF

5.1.4.1 Receipt and Inspection of Incoming Shipping Cask and Canister

During spent fuel shipment, the canister is contained within the shipping cask, which is mounted horizontally on a rail car or heavy haul trailer. Impact limiters are mounted on either end of the shipping cask and a personnel barrier cover is located over the shipping cask between the impact limiters.

When the shipping cask arrives at the PFSF, the shipping cask, impact limiters, and shipping cradle are visually inspected for damage prior to entry into the Restricted Area (RA). After initial receipt approval the shipment is moved into the security vehicle trap for inspection by security personnel to ensure no unauthorized devices enter the RA. When security clearance is complete, the shipment proceeds into the RA and into the Canister Transfer Building where the personnel barrier is removed and the shipping cask is surveyed for dose rates and contamination levels

5.1.4.2 Transfer of Canister from Shipping Cask to Storage Cask

Transfer of the canister containing spent fuel from the shipping cask to the storage cask takes place within the Canister Transfer Building. After the receipt inspection, the overhead bridge crane is used to remove the impact limiters. The shipping cask lifting yoke is attached to the crane and hooked to the shipping cask, which is uprighted on

The Canister Transfer Building itself is located approximately 425 ft (130 meters) from the nearest storage pad. It has reinforced concrete walls and roof designed to withstand tornado-driven missiles. The walls and roof provide substantial shielding of gamma and neutron radiation emitted from the sides (direct) and tops (scattered) of storage casks in the cask storage area. The Security and Health Physics Building is located approximately 948 ft (289 meters) from the nearest storage pad, and approximately 396 ft (121 meters) from the Canister Transfer Building. Shielding provided by the walls and roof of this structure reduce dose rates from the cask storage area to security and radiation protection personnel who will spend a large fraction of their working hours in this building. The Administration Building is located approximately 2,580 ft (786 m) and the Operations and Maintenance Building is located approximately 1,960 ft (597 m) from the nearest storage pad. Dose rates are sufficiently low at these distances such that the buildings do not require shielding to assure dose rates are ALARA to personnel in the buildings.

- Regulatory Position 2c on process instrumentation is met since the cask temperature monitoring system will utilize a data acquisition system to record cask temperature instrumentation readings, avoiding time spent by PFSF staff to make daily cask vent blockage surveillances and to read instrumentation in the vicinity of the storage casks.

- Regulatory Position 2d on control of airborne contaminants is not applicable because gaseous releases are precluded by the sealed canister design. No surface contamination is expected on the outer surfaces of the canister since process controls are maintained during fuel loading into the canister at the originating nuclear power plants. Assuming the outer surfaces of a canister have removable Co-60 contamination at the maximum levels permitted by

Section 10.2.2.1, and all of this is postulated to be released into the Canister Transfer Building atmosphere, general area radionuclide concentrations in the Canister Transfer Building would not exceed 10 CFR 20 Appendix B, Table 1, allowable airborne concentrations for occupational workers.

- Regulatory Position 2e on crud control is not applicable to the PFSF because there are no systems at the PFSF that could produce crud.

- Regulatory Position 2f on decontamination is met because the internal surfaces of shipping, transfer, and storage casks have hard surfaces that lend themselves to decontamination by wiping. Surfaces of the transfer cells' walls and floors are painted with a special paint that is easily decontaminated.

- Regulatory Position 2g on radiation monitoring is met with the use of area radiation monitors in the Canister Transfer Building for monitoring general area dose rates from the casks and canisters during canister transfer operations, and with thermoluminescent dosimeters (TLDs) along the perimeters of the RA and OCA to provide information on radiation doses. Continuous air monitors will be located in the exhaust of each canister transfer cell.

- Regulatory Position 2h on resin treatment systems is not applicable to the PFSF because there will not be any radioactive systems containing resins.

- Applicable portions of Regulatory Position 2i concerning other miscellaneous ALARA items is met because PFSF features provide a favorable working environment and promote efficiency (paragraph 2i(13)). These include:

adequate lighting in the Canister Transfer Building, including in the canister transfer cells, and on the storage pads; adequate ventilation in the Canister Transfer Building; adequate working space in the Canister Transfer Building and at the storage pads; and accessibility - with platforms or scaffolding and ladders that facilitate ready access to the tops of the shipping casks and storage casks and to the transfer cask doors where operators need to perform tasks during canister transfer operations. Regulatory Position 2i(15) is met because the emergency lighting system is adequate to permit prompt egress from any high radiation areas that could possibly exist in the vicinity of the canister/casks during canister transfer operations.

7.1.3 Operational Considerations

Specific PFSF operational considerations to achieve ALARA conditions are as follows:

- Fuel loading operations take place at the originating nuclear power plants, away from the PFSF. There are no fuel assembly handling operations at the PFSF.

- No surface contamination is expected on the canisters as the result of controls applied during the fuel loading operations at the originating nuclear power plants. Workers will therefore not be exposed to surface contamination or airborne contamination during canister transfer operations.

- Canister transfer between the shipping cask and the storage cask will take place within a shielded transfer cask.

- Prior to canister transfer operations, "dry runs" will be performed to train personnel on canister transfer procedures, discuss methods to minimize exposures, and refine procedures to achieve minimum probable exposures.

- The PFSF procedures and work practices will reflect ALARA lessons learned from other ISFSIs that use dry cask storage, as applicable.

- Operations research will be performed to determine types of tools, portable shielding, and equipment that will help to minimize exposures to workers involved in canister transfer operations.

- The overhead bridge crane and the semi-gantry crane, located in the Canister Transfer Building, are both single-failure-proof and are designed to withstand the design basis ground motion, as described in Chapter 4. The overhead bridge crane, whose range of travel covers the length and width of the Canister Transfer Building, including the transfer cell area, handles the shipping casks and moves the shipping casks into and out of the transfer cells. It can also be used to lift a transfer cask, a canister, or a storage cask, as necessary. The semi-gantry crane is designed to serve the transfer cells, and is used to lift the transfer casks and canisters during the canister transfer operations. Operation of these cranes during canister transfer operations is discussed in Chapter 5. The HI-STORM canister is handled by the canister downloader, which is also a single-failure-proof lifting device. The cranes and lifting devices used during the canister transfer operation comply with single-failure criteria to avoid a cask or canister drop.

- A self-propelled cask transporter is used to move storage casks from the Canister Transfer Building to the storage pads. The cask transporter requires

minimum personnel and allows for quick and accurate placement of a storage cask.

- The storage casks are spaced on the storage pads with sufficient tolerance to facilitate ease of placement operations and minimize the time spent by operators near adjacent casks.

- The storage systems do not require any systems that process liquids or gases or contain, collect, store, or transport radioactive liquids. Therefore, there are no such systems to be maintained or operated. Any solid radioactive waste collected during canister transfer operations will be temporarily staged in steel drums in a holding cell in the Canister Transfer Building while awaiting shipping offsite, as described in Section 6.4.

- As discussed in Section 7.5.2, any water collected in the Canister Transfer Building shipping cask load/unload bay drain sumps is sampled and analyzed to verify it is not contaminated prior to its release. If contaminated water is detected, it will be collected in a suitable container, solidified by the addition of a suitable compound, staged in a holding cell while awaiting shipment offsite, and shipped offsite as solid waste in accordance with Radiation Protection procedures.

Regulatory position 4 of Regulatory Guide 8.8 is met with the use of area radiation monitors in the Canister Transfer Building and TLDs around the RA and OCA boundaries. In addition, radiation protection personnel will use portable monitors during shipping cask receipt, inspection, and canister transfer operations, and the operating staff will have personal dosimetry (Section 7.5.2). Continuous air monitors will be located in the exhaust of each canister transfer cell (Section 7.3.5). The access control point will be at the Security and Health Physics Building, as described in Section 7.5.2.

Protective equipment, including anti-contamination clothing and respirators, will be available in the Security and Health Physics Building and controlled by radiation protection personnel. Airborne monitoring will be performed using portable monitors as needed. A low-radiation background counting room is included in the Security and Health Physics Building.

Regulatory Guide 8.10 is incorporated into the PFSF operational considerations as described below:

1. Facility personnel are made aware of management's commitment to keep occupational exposures ALARA.
2. Ongoing reviews are performed to determine how exposures might be lowered.
3. There is a well-supervised radiation protection capability with specific, defined responsibilities.
4. Facility workers receive sufficient training.
5. Sufficient authority to enforce safe facility operation is provided to radiation protection personnel.
6. Modification to operating and maintenance procedures and to equipment and facilities are made where they substantially reduce exposures at a reasonable cost.
7. The radiation protection staff understands the origins of radiation exposures in the facility and seeks ways to reduce exposures.
8. Adequate equipment and supplies for radiation protection work are provided.

introduction of contaminated fuel pool water into the annulus when the transfer cask/canister is submerged in the spent fuel pool. As an alternative to this method, an inflatable seal can also be used in the TranStor fuel loading operation. The outside of the transfer cask is washed down after being lifted out of the spent fuel pool to remove loose surface contamination. For dry transfers, it is less likely for contamination of the canister to occur since the fuel loading process is done outside of the pool.

Following fuel loading operations at the originating nuclear power plants, a smear survey is performed to determine removable contamination levels on accessible outer canister surfaces near the top of the canister (canister lid and approximately 3 to 6 inches along canister sides down from the lid). In addition, smears are taken on internal surfaces of the transfer cask, following transfer of the canister from the transfer cask into the shipping cask, and removable contamination levels on the transfer cask internal surfaces are considered to be representative of removable contamination levels on the outer surfaces of the canister. In the event canister removable contamination levels (measured on canister top surfaces or inferred from levels measured inside the transfer cask) exceed the criteria specified in Section 10.2.2.1, the canister will not be released for shipment to the PFSF. Canisters with unacceptable levels of removable contamination must be decontaminated prior to release for transport to the PFSF. Once the shipping cask arrives at the PFSF and its closure is removed, a smear survey of accessible portions of the canister is again performed. If removable surface contamination levels on the top of the canister exceed the limits specified in Section 10.2.2.1 (22,000 dpm/100 cm^2 beta/gamma and 2,200 dpm/100 cm^2 alpha), the canister is returned to the originating nuclear power plant for decontamination.

Section 8.1.5 evaluates doses resulting from an off-nomal event involving the postulated release of Co-60 contamination assumed to cover the entire exterior surface of a canister at a concentration of 1.0 E-4 $\mu Ci/cm^2$ (approximately 22,000 dpm/100 cm^2). The evaluation concludes that the consequences of such a release to an individual at a

distance of 492 ft (150 meters) would be 0.03 mrem committed effective dose equivalent, with lower doses at the OCA boundary, a distance of 1,640 ft (500 meters) from the Canister Transfer Building at its nearest point.

Section 8.2.7 evaluates canister leakage under hypothetical accident conditions, conservatively assuming cladding rupture of all the fuel rods within the canister, and release of conservative fractions of fission and activation products from the fuel and canister. The doses calculated to result from this hypothetical accident are below the 5 rem total effective dose equivalent regulatory limit specified in 10 CFR 72.106 (b) at the OCA boundary.

canisters are sealed by welding and with the canisters in storage casks and the casks on the storage pads, there are no credible events that could result in releases of radioactive material from within the canisters or unacceptable increases in direct radiation levels. Area radiation and airborne radioactivity monitors are therefore not needed at the storage pads. However, TLDs will be used to record dose rates in the RA and along the OCA boundary fence.

Local radiation monitors with audible alarms will be installed in the Canister Transfer Building. These will provide warning to personnel involved in the canister transfer operation of abnormal radiation levels that could possibly occur during transfer operations. Because of the measures taken at the originating nuclear power plants to minimize loose surface contamination levels on the exterior of the canisters during fuel loading operations, as discussed in Section 7.2.2, and limits on surface contamination concentrations, as discussed in Chapter 10, it is unlikely that canister transfer operations would generate significant levels of airborne contaminants. 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. The radiological alarm systems will be designed with provisions for calibration and operability testing. There are no liquid or gaseous effluent releases from the PFSF. This satisfies the requirements of 10 CFR 72.126(b) and (c).

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7.5 RADIATION PROTECTION PROGRAM

7.5.1 Organization

The PFSF Radiation Protection Manager reports to the General Manager and to the Board of Managers (Figure 9.1-3) and is responsible for administering the radiation protection program and for the radiation safety of the facility. Minimum qualification requirements are set forth in Section 9.1.3.1. The radiation protection and ALARA programs are discussed in Section 7.1.

Responsibilities of the PFSF Radiation Protection Manager include the following:

- Administer the Radiation Protection program policies and procedures
- Review and approve radiation protection procedures
- Coordinate radiation protection group activities with operations and maintenance personnel
- Ensure adequate staffing, facilities, and equipment are available to perform the functions assigned to radiation protection personnel
- Establish goals for the Radiation Protection program
- Initiate and implement an exposure control program that factors dosimetry results into operational planning
- Issue or rescind "stop work" orders as appropriate
- Ensure that locations, operations, and/or conditions that have the potential for causing significant exposures to radiation are identified and controlled
- Review and approve training programs related to work in radiological areas or involving radioactive material
- Administer shipments of solid radioactive waste offsite for disposal
- Review root causes and corrective actions for incidents and deficiencies associated with Radiation Protection

- Ensure an effective ALARA program is maintained, in accordance with the guidance provided in Regulatory Guides 8.8 and 8.10
- Supervise the collection, analysis and evaluation of data obtained from radiological surveys and monitoring activities
- Participate in the event of an emergency, as required

Radiation protection technicians report to the Radiation Protection Manager. Responsibilities of the radiation protection technicians include the following:

- Conduct radiation, contamination, and airborne surveys and prepare complete and accurate records
- Prepare Radiation Work Permits to control access to and activities in radiologically controlled areas
- Identify and post radiation, contamination, hot particle, airborne and radioactive material areas in accordance with 10 CFR 20 requirments
- Monitor PFSF operations to assure good radiological work practices
- Implement ALARA program requirements
- Maintain and calibrate portable monitoring instruments
- Issue "stop work" orders whenever activities have the potentail to jeopardize the health and safety of workers, visitors, or the general public
- Verify proper packaging of any radioactive material
- Participate in the event of an emergency, as required

7.5.2 Equipment, Instrumentation, and Facilities

A sufficient inventory and variety of operable and calibrated portable and fixed radiological instrumentation will be maintained to allow for effective measurement and control of radiation exposure and radioactive material and to provide back-up capability for inoperable equipment. Equipment will be appropriate to enable the assessment of

sources of gamma, neutron, beta, and alpha radiation, including the capability to measure the range of dose rates and radioactivity concentrations expected. Radiation protection procedures will govern instrument calibration, instrument inventory and control, and instrument operation.

Portable survey and personnel monitoring instrumentation will include, but not be limited to, the following:

- Low-level contamination meters
- Beta/gamma portable survey meters
- Alarming beta/gamma personnel friskers
- Portable air samplers

Radiological instrument storage, calibration and maintenance facilities will also be located in the Security and Health Physics Building, along with a low-radiation background counting room containing laboratory equipment for measuring radioactivity.

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. External radiation dose monitoring will be accomplished through the use of thermoluminescent dosimeters (TLDs) and self reading dosimeters (SRDs) or digital alarming dosimeters (DADs). All operating personnel inside an active canister transfer cell will be required to utilize alarming dosimeters during the canister transfer process to warn of excessively high direct radiation to maintain exposures ALARA, thereby providing additional assurance that occupation exposures will not exceed the limits of 10 CFR Part 20. The official record of external dose to beta and gamma radiations will normally be obtained from the TLDs, with SRDs or DADs used as a means for tracking dose between TLD processing periods and as a backup to TLDs. Self-reading

dosimeters will be administered in accordance with the guidance in Regulatory Guide 8.4 (Reference 15).

Provisions exist in the Security and Health Physics 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 due to off-normal or accident conditions. The respiratory protection program will be established in accordance with 10 CFR 20 and consistent with the guidance of NUREG-0041 (Reference 16). Radiation protection procedures will include the conduct of bioassays including criteria for the performance of bioassay, dose tracking and methods for data analysis and interpretation. The bioassay program will be based on NRC Regulatory Guide 8.26 (Reference 17) and Regulatory Guide 8.9 (Reference 18).

Provisions for personnel decontamination are contained in the Security and Health Physics Building. Contamination of equipment or personnel is not expected to occur under normal conditions of operation. In accordance with the PFSLLC's policy of preventing generation of liquid radioactive waste, any necessary decontamination of equipment and personnel will be conducted using methods that produce only solid radioactive waste. Decontamination methods would typically include wiping the contaminated item with rags or paper wipes. Drain sumps are provided in the cask load/unload bay of the Canister Transfer Building which catch and collect water that drips from shipping casks (e.g. from melting snow) onto the floor. Water collected in the cask load/unload bay drain sumps is sampled and analyzed to verify it is not contaminated prior to its release. In the event contaminated water is detected, it will be collected in a suitable container, solidified by the addition of an agent such as cement or "Aquaset" so that it qualifies as solid waste, staged in the LLW holding cell while awaiting shipment offsite, and transported to a LLW disposal facility, in accordance with Radiation Protection procedures.

No process or effluent monitors are necessary because of the design of the PFSF storage system, in which spent fuel assemblies are stored in welded canisters. During routine storage operations at the PFSF, the only radiological instrumentation in use in the storage area will be the TLDs, as described in Section 7.3.5. Routine radiological surveys will use instruments that are controlled by the Radiation Protection Program and governed by existing procedures. Calibration procedures for radiological instrumentation will be established and applied to instruments used at the PFSF.

7.5.3 Procedures

Radiation protection requirements for all radiological work at the PFSF will be governed by radiation protection procedures. Radiation protection practices for cask loading and unloading operations, canister transfer, canister storage, and monitoring will also be based on these procedures, as well as on anticipated conditions when the task is to be performed. These procedures include, but are not limited to, the following:

- Procedure for performing badging functions for access authorization to the RA.
- Procedure for issuing personnel dosimetry, and monitoring, recording, and tracking individual exposures.
- Procedure for performing radiological safety training and refresher training.
- Procedure for performing ALARA reviews of plant procedures and monitoring of operations.
- Procedure for determining radiation doses on a periodic basis at RA and OCA boundaries using TLDs.

- Procedure for issuing, revising, and terminating radiation work permits and standing radiation work permits.
- Procedure for roping off, barricading, and posting radiation control zones.
- Procedure for decontaminating personnel, equipment, and areas.
- Procedure for performing radiation surveys.
- Procedure for smear swab sampling, counting, and calculation.
- Procedure for calibrating detection, monitoring, and dosimetry instruments.
- Procedure for quantifying airborne radioactivity.
- Procedure for maintaining records of the radiation protection program, including audits and other reviews of program content and implementation; radiation surveys; instrument calibrations; individual monitoring results; and records required for decommissioning.

CHAPTER 8

ACCIDENT ANALYSES

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8.1.5.2 Detection of Event

A release of some removable activity from the exterior surface of the canister could possibly occur as the result of impacts during the canister transfer operation. Significant impact of the canister during transfer operations would be observed by personnel associated with the transfer operation, which includes health physics coverage that would detect an activity release.

8.1.5.3 Analysis of Effects and Consequences

The following assesses the effects of postulated release of contamination from the external surfaces of a canister, conservatively assuming removable contamination levels of 1 E-4 μCi/cm^2 (22,200 dpm/100 cm^2) over the entire external surface area of a canister, much higher than is anticipated for canisters received at the PFSF and slightly above the removable surface contamination limit for accessible canister surfaces specified in Section 10.2.2.1 (22,000 dpm/100 cm^2) for beta/gamma activity. It is conservatively assumed that an event causes 100 percent of the canister external surface contamination to be released to the atmosphere.

Assuming the contamination is Co-60 particulate activity evenly distributed at a concentration of 1 E-4 μCi/cm^2 over the entire external surface of a HI-STORM canister (TranStor canister has a smaller area), with a surface area of approximately 312,000 cm^2 there would be a total activity inventory of approximately 31.2 μCi. The nearest distance from a storage pad to the OCA fence (site area boundary) is 646 meters, and the nearest distance from the Canister Transfer Building to the OCA fence is 500 meters. A χ/Q of 1.94 E-3 sec/cubic meter was calculated in accordance with Regulatory Guide 1.145 (Reference 6), assuming a distance of 500 meters to the dose

receptor, a wind speed of 1 meter/sec, atmospheric stability class F, with no consideration for plume meander.

The dose conversion factor for intake of Co-60 is specified in EPA Federal Guidance Report No. 11 (Reference 7) as a committed effective dose equivalent (CEDE) of 5.91 E-8 Sv/Bq, equal to 219 mrem/μCi. The highest dose conversion factor for committed dose equivalent (CDE) to any organ from Co-60 is that for the lungs, 3.45 E-7 Sv/Bq, equal to 1,277 mrem/μCi. An adult breathing rate of 3.3 E-4 cubic meters per second is assumed in accordance with Reference 7. A respirable fraction of 1.0 is assumed. Assuming an individual is located within the plume 500 meters from the release point for the duration of the release, the individual would receive a CEDE of 4.37 E-3 mrem and a CDE to the lungs of 2.55 E-2 mrem. The dose to an individual at the OCA boundary from external exposure to radiation emitted by the plume (submersion dose) was also calculated, using the effective dose conversion factor for Co-60 specified in EPA Federal Guidance Report No. 12 (Reference 30). This dose conversion factor, representative of exposure to a semi-infinite cloud of radioactive material, is 1.26 E-13 Sv/sec per Bq/m^3, equal to 4.66 E-4 mrem/sec per μCi/m^3. The submersion dose from external exposure to Co-60 in the plume was calculated to be 2.82 E-5 mrem. Adding the external dose from submersion to the internal CEDE and CDE to the lungs results in a total effective dose equivalent (TEDE) of 4.40 E-3 mrem, and a total lung dose of 2.55 E-2 mrem. These doses are well below the 10 CFR 72.106(b) criteria of 5 rem TEDE and 50 rem lung dose that apply to 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.

The dose was also calculated to onsite personnel assumed to be located 150 meters from the release point using the same methodology and assumptions discussed above,

with a calculated χ/Q of 1.40 E-2 sec/cubic meter. Onsite personnel 150 meters from the release point would receive a CEDE of 3.15 E-2 mrem, a CDE to the lungs of 1.84 E-1 mrem, and an external dose due to submersion of 2.04 E-4 mrem. Adding the external dose from submersion to the internal CEDE and CDE to the lungs results in a TEDE of 3.17 E-2 mrem and a total lung dose of 1.84 E-1 mrem.

8.1.5.4 Corrective Actions

Even if relatively high levels of contamination are encountered on the external surfaces of a canister, which is not anticipated, no corrective action is necessary. Doses at the OCA fence resulting from release of activity from a contaminated canister would be negligible.

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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, and
- Hypothetical accident pressurization.

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

seismic hazard at a nuclear power plant site. In response to the regulatory changes in seismic analysis methodology for siting nuclear power plants, and anticipated changes to Part 72 (SECY-98-126), a PSHA has been performed for the PFSF for vibratory ground motions and surface fault displacement. The seismic design basis for the PFSF has been revised (Reference 29), with the current design basis ground motions based on the PSHA, as discussed in Sections 2.6 and 3.2.10. The design basis ground motions are characterized by site specific response spectrum curves having peak ground accelerations of 0.40 g horizontal (two directions) and 0.39 g vertical, as identified in Sections 2.6.4.9 and 3.2.10.1.1.

The site specific cask stability analyses described in the following paragraphs were based on the PFSF original site specific deterministic design earthquake, which has been superseded by the current design basis ground motion established by the PSHA. The response spectrum of the PFSF original site specific deterministic design earthquake (referred to as the PFSF deterministic design earthquake) bounds that associated with the current design basis ground motion. The site specific cask stability analyses are therefore conservative, being based on significantly higher ground accelerations than those of the design basis ground motion.

The storage system structural design bases, which identifies earthquake loads and the structural design of the storage systems, are contained in Section 4.2.1.5.1 (H) for HI-STORM and Section 4.2.2.5.1 (H) for TranStor.

8.2.1.2 Accident Analysis

The HI-STORM and TranStor storage casks are analyzed for a generic design earthquake as selected by each cask vendor and as described in their respective SARs (References 2 and 3). The HI-STORM and TranStor storage casks were also analyzed for the PFSF site specific deterministic design earthquake, represented by response spectrum curves with a zero period acceleration of 0.67 g horizontal (two directions) and 0.69 g vertical. Both the HI-STORM and TranStor storage casks are analyzed for these conditions by the vendors to assure structural strength of the cask and cask stability. Since the response spectra of the PFSF deterministic design earthquake bounds that associated with the current design basis ground motion, the cask stability analyses provide assurance that the casks will not tip over or slide excessively in an earthquake.

In addition to the vendor's PFSF site specific cask stability analyses, a separate and independent site specific cask stability analysis was performed by a structural-mechanical engineering consultant specializing in seismic dynamic analysis of equipment and structures. The analysis was performed by J. D. Stevenson, Consulting Engineer, for the purpose of independently confirming the cask stability conclusions of the vendor's analyses. This bounding case analysis considered both the HI-STORM and TranStor storage casks. The analysis demonstrates the storage casks will not tip over or slide excessively in an earthquake and confirms the conclusions of the vendors.

A summary of the vendor's cask stability analyses and the independent cask stability analyses performed by J. D. Stevenson, Consulting Engineer, are as follows:

HI-STORM Cask Stability Analysis

The HI-STORM generic seismic cask stability analysis is described in Section 3.4.7 of the HI-STORM SAR. The analysis demonstrates the HI-STORM storage cask will not

tip over or slide excessively when subjected to a seismic event characterized by Regulatory Guide 1.60 response spectra curves with a zero period acceleration of 0.80 g in three orthogonal directions. The cask is allowed to rock, rim, and slide during the event. The results show the cask centroid projection on the pad remains inside of the initial contact envelope and is therefore within the stability criteria established by the cask vendor.

The generic analysis determined that inertia loads produced by the seismic event are less than the 45 g loads due to the postulated HI-STORM vertical drop and non-mechanistic tipover events (Section 8.2.6). Stresses in the canister due to the seismic event are bounded by the 45 g deceleration load from the hypothetical end drop and side drop events described in Section 3.4.4.3.1 of the HI-STORM SAR. As discussed in Appendix 3.B of the HI-STORM SAR, shell and concrete stresses in the storage cask resulting from the seismic shear and moment forces are evaluated and determined to be acceptable.

The generic cask stability analysis in the HI-STORM SAR assumes a single cask resting on a rigid surface. The analysis does not consider soil-structure interaction, which will affect the dynamic properties and seismic response of the structural system. In order to verify the cask stability under site specific conditions, a site specific cask stability analysis was performed by the cask vendor (Reference 8).

The HI-STORM storage cask was analyzed using proprietary qualified software for the PFSF deterministic design earthquake characterized by response curves with a zero period acceleration of 0.67 g in both horizontal directions and 0.69 g in the vertical direction. The analysis considered soil-structure interaction, actual storage pad size, and a variety of cask placements on the pad.

The site specific cask stability analysis was performed by developing three statistically independent acceleration time histories from the site specific response spectra. This seismic input was applied three-dimensionally to the structural system model, which included the storage pad, soil springs, and various cask placements to determine the worst case response. The site specific seismic analysis employs the finite element modeling of the cask geometry and boundary conditions and numerical integration of the dynamic equation.

The casks are modeled as a two body system with each overpack described by six degrees of freedom which captures the inertial rigid body motion of the overpack. Within each overpack the internal MPC is modeled by an additional five degree of freedom mass which is sufficient to define all but the rotational motion of the MPC about its own longitudinal axis, a motion which is of no significance in this analysis. Spring constants are developed to simulate the contact stiffness between the MPC and the overpack cavity. Interface spring constants are developed for the overpack-to-concrete pad linear compression only contact springs and for the associated friction springs at each of the 36 contact locations for each overpack on the pad.

Soil-structure interaction is incorporated into the model by the development of soil springs to reflect the characteristics of the underlying soil mass beneath the pad. Horizontal, vertical, rocking and torsional spring rates were calculated along with appropriate soil mass and damping values and applied at the pad-soil interface.

The cask stability analysis was performed by computer methods using cask-to-pad coefficients of friction equal to 0.2 (emphasizes sliding potential) and 0.8 (emphasizes tipping potential) to bound the maximum sliding and tipping behavior of the cask. The results of the site-specific analysis show that the storage casks will not tip over or slide to the extent of impacting adjacent casks during the PFSF deterministic design earthquake.

For the limiting case with a 0.8 coefficient of friction (maximum tip), there is minimal rotation of the cask vertical centerline, except the case with a single cask on the pad. For this case, there is evidence the cask edges and rolls to a new position approximately 5 inches from its original position during the seismic event. The maximum tip, identified as the lateral motion of the cask top center point from its initial position, is approximately 13 inches for the single cask case and considerably less for all other cases. For the limiting case bounded by a 0.2 coefficient of friction (maximum slide), the maximum distance in which a cask will slide is shown to be 10.25 inches. For both coefficients of friction considered, cask motions are generally in-phase with each other. The casks are spaced on the storage pad at 15 ft center-to-center which provides 47.5 inches clear between casks (cask diameter is 132.5 inches) and provides a considerable margin of safety against impacts between casks during a seismic event. The analysis also shows that for those cases where the cask(s) tip up slightly and then "slap" back down on their base, vertical deceleration forces, up to 8.41 g, are developed, but these values are well below the design basis deceleration forces for the HI-STORM 100 system.

The site specific cask stability analysis performed by the cask vendor demonstrates that the HI-STORM storage cask will not tip over in a seismic event. The calculated cask movements are much less than the cask spacing on the storage pad and as such, the storage casks are shown not to impact one another or move off of the storage pad in a seismic event. Therefore, no radioactive material would be released from the storage system when subjected to the DE. The HI-STORM storage system thus meets the general design criteria of 10CFR 72.122(b), as it relates to earthquakes.

TranStor Cask Stability Analysis

The TranStor generic cask stability analysis is described in Section 11.2.5 of the TranStor SAR. The analysis demonstrates the storage cask will not tip over when

subjected to a seismic event characterized by Regulatory Guide 1.60 response spectra curves with a zero period acceleration of 0.75 g in two horizontal directions and 0.50 g in the vertical direction. The generic cask stability analysis is performed using a two-dimensional analysis, and as such, utilizes a horizontal response spectrum with a resultant zero period acceleration of 0.8 g and a vertical response spectrum with a zero period acceleration of 0.5 g. The horizontal response spectrum is the resultant of both 0.75 g horizontal accelerations combined using the 100-40 rule allowed by NUREG/CR-0098 (Reference 9). The analysis also concludes that with a maximum ground displacement of 20.0 inches (substantially less than 44 inches of clear space between the casks), sliding would not cause impact between adjacent casks.

The generic cask stability analysis in the TranStor SAR assumes a single cask resting on a rigid surface. The analysis does not consider soil-structure interaction, which will affect the dynamic properties and seismic response of the structural system. In order to verify the cask stability under actual conditions, a site specific cask stability analysis was performed by the cask vendor.

The TranStor storage cask was analyzed for the PFSF deterministic design earthquake characterized by site specific response curves with zero period accelerations of 0.67 g in both horizontal directions and 0.69 g in the vertical direction. The analysis considered soil-structure interaction, actual storage pad size, and a variety of cask placements on the pad.

The site specific cask stability analysis was performed using the ANSYS finite element program (discussed in TranStor SAR Section 11.2.5.2) and time history analysis. The time history is developed from the free-field seismic response spectra generated for the PFSF site. However, one of the important factors that must be considered when defining the structural response to an earthquake is that the ground input at the base of the foundation may be influenced by the presence of the structure itself. From the

dynamics viewpoint, the seismic response of a dynamic system (free field soil) may be altered due to both inertial and kinematic interaction when another system (structure) is present. As a result, the motion at the structure base may be different from the free field motion. The soil-structure interaction effects are discussed in ASCE 4 (Reference 10), Section 3.3 and the Commentary. The problem can be analyzed as two separate sub-problems: 1) analysis of soil-structure interaction to determine the seismic input at the cask base, and 2) analysis of the cask stability under that excitation.

To take into account soil-structure interaction at the PFSF site, the SUPER SASSI/PC computer program (System for Analysis of Soil-Structure Interaction) (Reference 11) is used. This program is a specialized finite-element code originally developed at UC Berkeley by Prof. J. Lysmer and modified into a PC version by Stevenson and Associates. The analysis methodology is as follows:

1. Model the structure and soil in three dimensions. Embedment of the structure is adequately represented using 3-D solid elements. The casks are represented as rigid bodies because their natural frequencies have been shown to be well beyond the seismic cutoff of 33 Hz. The site specific soil properties (established by testing) are used for the top 120 feet and an elastic half-space is modeled below that level.

2. Develop free field vertical and two horizontal time histories which envelope the provided response spectra. The duration of 20 seconds has been selected for time histories used in this evaluation. Power Spectral Density (PSD) distribution is also evaluated to assure that the energy is reasonably distributed across the spectrum. All time histories are generated to be statistically independent.

3. Provide the generated free field time histories and the soil-structure model as input for SUPER SASSI/PC.

4. Obtain translational time histories at the base of the storage cask. These time histories are used in the cask tip over analysis.

5. Repeat the procedure for different pad loading patterns to determine the configuration that produces the highest input at the cask base. One, four, and eight cask patterns are considered.

The stability of the TranStor storage cask during the PFSF deterministic design earthquake is evaluated using the ANSYS finite element program and non-linear time history analysis. Non-linearity of the system results from the fact that the storage pad is capable of producing a reaction in only one direction (upward). No hold-down restraint is available to prevent the cask from rocking. Non-linear gap elements are used in the finite element model to represent this condition. The most unfavorable cask geometry (highest center of gravity location) is conservatively used. The highest vertical and horizontal components from the different pad loading configurations are taken for the cask stability analysis.

The cask ANSYS model is the same as used to model the generic seismic event in the TranStor SAR. Since the model is two-dimensional, i.e. only one horizontal and one vertical excitation is applied, the horizontal time history must represent the geometrical sum of two independent horizontal components. This is accomplished by using the 100-40 rule allowed by NUREG/CR-0098 (Reference 9). According to this rule, due to statistical independence of two motions, the structure can be designed to take the combined effects of 100% in one particular direction and 40% of the effects corresponding to the other direction. As a result, the ZPA for the horizontal component used in the analysis is increased by the factor of $\sqrt{(1^2 + 0.4^2)} = 1.08$.

Results of the calculation demonstrate that the cask is stable under the loads of the PFSF deterministic design earthquake. Furthermore, a vertical ground displacement of approximately 5.6 feet would be required to move the center of gravity over the corner of the cask so that the cask would topple. This type of ground displacement and/or failure of the foundation is considered to be unrealistic and, hence, it is concluded that in addition to not toppling due to the kinetic energy of the earthquake, the cask will also not topple due to permanent failure and vertical movement of the foundation. Therefore, based on this analysis, it is concluded that the cask will not tip over during a seismic event.

The canister, its internals, and the concrete storage cask are very rugged and, since tip over is precluded, stresses due to the PFSF deterministic design earthquake are relatively minor and bounded by the 17.5 g load during the off-normal handling event (Section 8.1.4.3). Section 11.1.5 of the TranStor SAR determined that stresses in the canister resulting from this event are within allowable limits. Therefore, the TranStor canister will not breach or suffer damage in the event of the PFSF deterministic design earthquake.

Shell and concrete stresses in the storage cask from the seismic shear and moment forces are bounded by those resulting from the 19.8 g deceleration load from the hypothetical TranStor cask tip over analysis discussed in Section 8.2.6. Loads on the storage cask resulting from the PFSF deterministic design earthquake are included in the load combinations discussed in Chapter 4, where it is shown that stresses resulting from required load combinations are within allowable stress capacities.

The seismic analyses show that the storage cask will not tip over and no damage will be sustained by either the canisters, their internals, or the storage casks in the event of an earthquake. Therefore, no radioactive material would be released from the storage system when subjected to the PFSF deterministic design earthquake. The TranStor

storage system thus meets the general design criteria of 10CFR 72.122(b), as it relates to earthquakes.

Independent Cask Stability Analysis

An independent cask stability analysis was performed by J. D. Stevenson, Consulting Engineer, for the purpose of confirming the conclusions of the vendors' site specific cask stability analyses. The analysis considered both the HI-STORM and TranStor storage casks to determine a controlling and bounding storage cask configuration. Although both storage casks are similar in overall dimensions and weight, the cask which was selected as bounding for evaluation of seismic stability was the HI-STORM MPC-32 canister. The HI-STORM MPC-32 canister is the heaviest and the tallest loaded canister and storage cask combination with a weight of 356,521 lb. and a center of gravity of 118 in. above the base of the cask.

The cask stability analysis was performed using a two step approach. First, the cask/pad/soil system was modeled using the SUPER SASSI/PC computer program (Reference 11) to include the effects of soil-structure interaction. The results of the SUPER SASSI/PC analysis were then used in a non-linear time-history analysis using the ANSYS (Reference 12) computer program. The ANSYS analysis was for a single cask, considered essentially as a rigid body, evaluated for overturning. Additional rigid body analysis was also considered, as suggested by Housner (Reference 13), to check the effects of cask tip over and sliding as a rigid body.

The independent cask stability analysis utilized the PFSF deterministic design earthquake response spectra curves, having a zero period acceleration of 0.67 g horizontal (two directions) and 0.69 g vertical. The free field ground surface response spectra were used to develop 3 independent synthetic time histories using the SPECTRA (Reference 14) computer program. These time histories were used as input to the SUPER SASSI/PC computer analysis to evaluate the soil-structure interaction.

The model included the cask storage pad (3 ft. thick x 30 ft. wide x 64 ft. long) with eight casks in place. The casks were idealized by rigid sticks (beam elements) with translational and rotational inertia concentrated at the cask center of gravity. Rigid links were introduced to simulate the physical sizes of the cask bases. The model was excited by the three acceleration time-histories obtained from SPECTRA. The results of the SUPER SASSI/PC computer analysis were the time-history motions at the base of the cask. Computed results showed that the rocking motions of the storage pad were practically negligible in comparison with the translational motions.

The output time-history motions at the base of the cask were then applied to the non-linear ANSYS analysis using a single cask. Several analyses were performed using ANSYS to evaluate the potential for the cask to tip during the seismic event as simulated by the storage pad seismic motions. The ANSYS models used included both two-dimensional and three-dimensional beam element models. Contact elements were used to model the interface between the cask and the storage pad. The cask was modeled using rigid beam elements with a single vertical element for the cask. The cask base was modeled with two horizontal elements at the base extending to the outer edges of the cask diameter for the two-dimensional model and multiple horizontal elements at thirty degrees spacing for the three-dimensional model. The mass of the cask was lumped at the cask center of gravity located on the vertical element. The model was loaded with gravity prior to earthquake loading. Friction effects were included using a coefficient of friction of 0.5 between the cask and the pad. The cask tip over analysis was based on a constant friction force (lateral resistance at the base of the cask model) active during the stability analysis which conservatively overestimates the overturning potential of the cask. In addition, some rigid body dynamic analyses were performed to provide insight into the behavior of the ANSYS models, and also to evaluate the sliding effects. Rigid body rocking analysis indicates a maximum angle of cask rotation less than 1.5 degrees. Rigid body sliding response of the cask was calculated and found to be less than one foot.

From the analysis performed, it was determined that the most conservative results (and therefore the greatest overturning response) were obtained with the two-dimensional model with the displacement motion applied only at the center base node of the model. The results of the two-dimensional analyses indicate that the rotation at the base of the cask model is approximately 7.2 degrees, which will cause the storage cask top to move laterally approximately 29.0 inches. For the three-dimensional model, a maximum cask rotation of approximately 1.3 degrees was obtained, which will cause the storage cask top to move laterally approximately 5.2 inches. It should be noted that for the three-dimensional model, two components of displacement loading were applied simultaneously. Whereas, for both the two-dimensional and three-dimensional models, the vertical earthquake effects were included as an acceleration time history.

The site specific cask stability analysis performed by J. D. Stevenson, Consulting Engineer, demonstrates that a bounding storage cask configuration, enveloping both the HI-STORM and TranStor storage casks, will not tip over or slide excessively in an earthquake. The computed tip angle of the cask was 7.2 degrees for the two-dimensional model and 1.3 degrees for the three-dimensional model for the prescribed seismic criteria and soil conditions. Lateral displacements of the top of the cask are well below the point at which the cask would tip over (approximately 28.85 degrees). And, using a coefficient of friction of 0.5, sliding is less than one foot. These cask movements are within the clear distances between casks (47.5 inches) and will preclude impact of adjacent casks.

Canister Transfer Operations

Canister transfer operations are performed in the Canister Transfer Building and described in Chapter 5. The overhead bridge crane, located inside the building, is used to handle the shipping casks and transfer casks of both vendors, and can also be used to handle the TranStor canister. The semi-gantry crane is used to handle the transfer

casks of both vendors and the TranStor canister. The canister transfer operations of the two vendors are essentially the same, with the exception that the HI-STORM operation uses the canister downloader and the TranStor operation uses a crane to raise and lower the canister. The canister downloader is a hydraulically powered lifting device that is bolted onto the top of the HI-TRAC transfer cask.

The overhead bridge crane and the semi-gantry crane are designed to withstand the PFSF deterministic design earthquake, as is the Canister Transfer Building which provides the structural support for the cranes. As discussed in Section 4.7.2, the overhead bridge crane and semi-gantry crane are designed to meet the criteria for single-failure-proof lifting devices. The canister downloader is also a single-failure-proof lifting device (Section 4.7.3.5.1), which is designed to withstand the PFSF deterministic design earthquake. The overhead bridge crane, semi-gantry crane, and canister downloader are capable of withstanding the PFSF deterministic design earthquake during the critical lift without toppling or dropping the load. Therefore, the PFSF deterministic design earthquake will not cause a load drop accident during lifting of either vendor's shipping cask, transfer cask, or a canister.

As discussed in Chapter 5, the overhead bridge crane lifts either a HI-STAR or a TranStor shipping cask off the heavy haul trailer or rail car and moves it into one of the canister transfer cells, where it is placed upright on its bottom end in preparation for the canister transfer operation. Prior to disconnecting the crane and unbolting the lid, the shipping cask is secured in place by attaching seismic support struts between the cask and the transfer cell walls. The seismic support struts are designed to resist forces resulting from the PFSF deterministic design earthquake and maintain the shipping cask in its upright position. Once the lid is unbolted and removed, the canister is accessible through the top of the shipping cask. Canister lifting attachments and hoist

slings are installed on the canister lid and the transfer cask placed onto the shipping cask by means of the overhead bridge crane or semi-gantry crane. This stacked cask configuration that occurs during the canister transfer process, with a transfer cask resting on either a shipping or storage cask, was evaluated for stability in the event of a PFSF deterministic design earthquake (see Section 4.7.4.5.1).

In the HI-STORM transfer operation, the HI-TRAC transfer cask, resting on either the HI-STAR shipping cask or the HI-STORM storage cask, is connected to either the overhead bridge crane or the semi-gantry crane throughout the transfer operation. Continuous connection of the crane to the transfer cask provides assurance that the transfer cask cannot topple in the event of forces associated with a PFSF deterministic design earthquake.

In the TranStor transfer operation, during the stacked cask portion of the operation, the transfer cask is disconnected from the crane when the crane is used to lift the canister out of the shipping cask into the transfer cask, and when the crane is used to lower the canister from the transfer cask into the storage cask, as described in Chapter 5. In order to assure cask stability in the event of an earthquake, the crane is not disconnected from the transfer cask until seismic support struts are attached to the transfer cask, as discussed in Section 4.7.4.5.1. The seismic support struts are physically connected to the walls of the transfer cell and are designed to resist forces resulting from the PFSF deterministic design earthquake and maintain the transfer cask in its upright position. Therefore, the stacked cask configuration of each vendor is stable and will withstand the forces associated with a PFSF deterministic design earthquake without a drop accident.

8.2.1.3 Accident Dose Calculations

The PFSF deterministic design earthquake is not capable of damaging the canisters or storage casks during canister storage operations. Since accelerations associated with this seismic event are significantly higher than those associated with the PFSF design basis ground motions (Section 3.2.10.1.1), the canisters and storage casks will also safely withstand the design basis ground motion. The Canister Transfer Building structure is designed to withstand the PFSF deterministic design earthquake. 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 a PFSF deterministic design earthquake 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 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.

localized damage to the radial concrete shield and outer steel shell where the cask impacts the pad could result in an increased surface dose rate due to the damage. However, this would not produce a noticeable increase in the dose rates at the RA fence or OCA boundary because the affected area would likely be small (HI-STORM SAR, Section 11.2.3). The maximum concrete crush depth of 2 inches calculated for the TranStor storage cask would approximately double the dose rates in the localized area, but would not significantly affect the overall dose rates from the storage cask (TranStor SAR Section 11.2.10).

In the hypothetical event of a storage cask tipover / drop accident that is postulated to result in damage to a storage cask, the PFSF staff would evaluate the extent of damage and if needed would remove a canister from the damaged storage cask and transfer the canister to a new storage cask in the Canister Transfer Building utilizing a transfer cask to provide canister shielding and a single-failure-proof crane.

8.2.7 Canister Leakage Under Hypothetical Accident Conditions

The leakage of a canister under hypothetical accident conditions wherein cladding of 100% of the fuel rods is postulated to have ruptured is classified as Design Event IV as defined by ANSI/ANS-57.9. This is not a credible accident at the PFSF.

8.2.7.1 Cause of Accident

The HI-STORM and TranStor canisters are totally sealed, integrally welded pressure vessels, designed to Section III of the ASME BPVC. There are no gaskets, mechanical seals, or packing that could provide a potential leakage path for the radioactive fission products contained within the fuel cladding. The canisters are provided with multiple closures to confine the radioactive fuel. Following welding of the closures, the canisters are tested to verify their leaktight integrity. No components are required to penetrate the sealed canisters after helium backfilling is completed and the outer closure is welded in place. The postulated failure of the cladding of all fuel rods in a canister and release of gases normally contained in the fuel rod cladding under pressure would not challenge the integrity of the canisters (Section 8.2.10). Maximum canister leakage under conditions wherein cladding of 100% of the fuel rods is postulated to have ruptured is considered to be a non-credible event, which will not occur over the life of the PFSF. Nevertheless, this accident is hypothesized and analyzed below. Doses resulting from the canister leakage under hypothetical accident conditions were calculated in accordance with Interim Staff Guidance-5 (ISG-5, Reference 31).

8.2.7.2 Accident Analysis

In this accident analysis, it is postulated that a canister leaks at the maximum rate permitted by the closure helium leakage test acceptance criteria. Such a leak would require a significant defect in each of two redundant closure welds. In this hypothetical

accident condition, it is assumed that cladding of 100% of the fuel rods stored in the canister has ruptured. Failure of the cladding of all the rods in a canister is not a credible scenario for the reasons discussed in Section 8.2.10. The spent fuel is stored in a manner that complies with the general design criteria 10 CFR 72.122(h), in that the spent fuel cladding is protected during storage against degradation that could lead to gross ruptures. The space internal to the confinement boundary is filled with an inert gas (helium) without the presence of air or moisture that might produce the potential for long term degradation of the spent fuel cladding. The spent fuel storage systems are designed to assure that fuel is maintained at temperatures below those at which fuel cladding degradation occurs, under normal, off-normal, and accident conditions. It is therefore highly unlikely that a spent fuel assembly with intact fuel cladding will undergo cladding failure during storage, and the assumption of complete cladding failure of all rods in a canister is extremely conservative.

This postulated cladding rupture results in the escape of rod fill gas and fission product gases from the rods to the canister innerspace, with consequent pressure increase of the canister innerspace. In addition, it is conservatively assumed that the canister is at its maximum storage temperature, thus maximizing the canister internal pressure and the flow rate of gas through the leak path to the atmosphere.

For the HI-STORM canister, the hypothetical accident conditions of temperature and pressure result in the canister leak rate increasing from Holtec's test condition leak rate of 7.5 E-6 scc/sec (5.0 E-6 scc/sec leakage test acceptance criteria plus 2.5 E-6 scc/sec minimum sensitivity) to a calculated leak rate of 1.58 E-5 scc/sec. The results of this leak rate calculation, performed using the equations provided in ANSI N14.5-1977 (Reference 32), are included in the HI-STAR Storage SAR (Reference 33). A leak rate of 1.0 E-4 cc/sec was used for the TranStor canister under hypothetical accident conditions. Leakage rate test criteria assure that the leak rate from these canisters will not exceed 1.0 E-4 cc/sec under the hypothetical accident conditions.

Both storage system vendors used the SAS2H and ORIGEN-S modules of the SCALE4.3 system to generate the gaseous, volatile, and fuel fine radionuclide inventories associated with PWR and BWR spent fuel assumed to have conservative burnup, cooling time and enrichment. Nuclides that contribute greater than 0.1% of the total curie inventory for a fuel assembly, as well as iodine, were considered. The Co-60 crud inventory was determined using the 140 $\mu Ci/cm^2$ crud surface activity for PWR rods and the 1254 $\mu Ci/cm^2$ crud surface activity for BWR rods guidance provided in NUREG/CR-6487 (Reference 34). These surface concentrations were multiplied by the surface area per assembly (3 E5 cm^2 and 1 E5 cm^2 for PWR and BWR rods respectively), also provided in NUREG/CR-6487. The Co-60 source terms were then decay-corrected to account for the cooling time, using the half life of Co-60. It was determined that maximum dose rates are associated with postulated leakage of a TranStor canister containing 61 design basis BWR fuel assemblies. While the HI-STORM MPC-68 canister contains seven more BWR fuel assemblies and a slightly more conservative burnup/cooling time combination (Holtec used 40 GWd/MTU burnup and 5 years cooling time compared to 40 GWd/MTU burnup and 6 years cooling time assumed for the TranStor BWR canister), the calculated leak rate for the HI-STORM MPC-68 under bounding accident conditions of temperature and pressure (1.58 E-5 cc/sec) is substantially less than the 1.0E-4 cc/sec for the TranStor canister.

The radionuclide inventory for the TranStor BWR canister was based on 61 design basis BWR fuel assemblies (GE 8X8) with a burnup of 40,000 MWd/MTU, 6 years cooling time, and 2.95% enrichment. This is conservative, since fuel with these characteristics is too "hot" for shipment to the PFSF. This ensures that the inventory used in the calculation exceeds that of fuel authorized for storage at the PFSF.

The radionuclide release fractions are based on Table 4-1 of NUREG-1617 (Reference 35) for hypothetical accident conditions, in accordance with ISG-5, and are as follows:

Nuclides	Release Fractions
Gases (Includes H-3, Kr-85, I-129)	0.30
Crud (Includes Co-60)	1.0
Volatiles (Includes Sr-90, Ru-106, Cs-134, Cs-137)	2.0 E-4
Fuel Fines (Includes Y-90, Sb-125, Te-125m, Ce-144, Pr-144, Pm-147, Eu-154, Eu-155, Pu-238, Pu-239, Pu-240, Pu-241, Am-241, Cm-244)	3.0 E-5

The activity inventory of each radionuclide released from the fuel into the canister was calculated by multiplying the total activity of the radionuclide associated with 61 BWR fuel assemblies times the above release fraction. For conservatism no credit was taken for holdup of particulates and volatiles released from the fuel inside the canister.[1] Therefore, 100 percent of these radionuclides and 100 percent of the H-3, Kr-85 and I-129 are assumed to be available for release from the canister. The release rate for each radionuclide was calculated by taking the TranStor canister gaseous leak rate under the hypothetical accident conditions (1.0E-4 cc/sec) divided by the minimum free volume of a TranStor canister loaded with BWR fuel assemblies (accounting for the fuel rod plenum volumes), multiplied by the activity of each radionuclide available for release.

[1] Based on Table XIX of Reference 25, 90 percent of particulate and volatile fission products would be subject to plateout or deposition within the leaking canister following release from the fuel rods, and would not be available for release to the atmosphere. However, for conservatism no credit has been taken for holdup of particulates and volatiles released from the fuel inside the canister.

8.2.7.3 Accident Dose Calculations

The nearest distance from a PFSF storage pad to the OCA fence (site area boundary) is 646 meters, and the nearest distance from the Canister Transfer Building to the OCA fence is 500 meters. A χ/Q of 1.94 E-3 sec/cubic meter was calculated in accordance with Regulatory Guide 1.145 (Reference 6), assuming a distance of 500 meters from the release source to the dose receptor, a wind speed of 1 meter/sec, and atmospheric stability class F, with no consideration for plume meander.

The dose conversion factors for internal doses due to inhalation, the Committed Effective Dose Equivalent (CEDE) and Committed Dose Equivalent (CDE) to organs, were obtained from the EPA Federal Guidance Report No. 11 (Reference 7). An adult breathing rate of 3.3 E-4 cubic meters per second was assumed (Reference 7). For conservatism no credit was taken for a respirable fraction, and internal doses were calculated assuming that 100% of radionuclides released from the leaking canister are of respirable size.[2] In addition to internal doses, doses due to external radiation from submersion in the plume (deep dose equivalent) were also evaluated. Dose conversion factors for submersion were obtained from EPA Federal Guidance Report No. 12 (Reference 30). In accordance with ISG-5, a canister leakage duration of 30 days was assumed for this hypothetical accident condition. Dose calculations were conservatively based on the assumption that an individual is continuously present at the location nearest the canister transfer building on the OCA boundary for the 30 day leakage duration, and the wind constantly blows in this direction for 30 days.

[2] Based on Table XX of Reference 25, 95 percent of particulates released from inside the fuel rod due to cladding breach are greater than 10 microns aerodynamic diameter and are non-respirable. However, for conservatism no such credit has been taken and the respirable fraction is assumed to be 1.

Calculation determined a CEDE of 75.7 mrem due to inhalation and an external dose due to submersion in the plume of 0.155 mrem, for a Total Effective Dose Equivalent (TEDE) of 75.7 + 0.155 = 75.9 mrem.[3] The maximum organ dose is the CDE to the bone surface plus the submersion dose, calculated to be 824 + 0.155 = 824 mrem. The skin dose was calculated to be 0.28 mrem, which serves as a reasonable approximation of the dose to the lens of the eye. 10 CFR 72.106(b) requires that any individual located on or beyond the nearest boundary of the controlled area may not receive from any design basis accident the more limiting of a total effective dose equivalent of 5 rem, or the sum of the deep-dose equivalent and the committed dose equivalent to any individual organ or tissue (other than the lens of the eye) of 50 rem. The lens dose equivalent shall not exceed 15 rem and the shallow dose equivalent to skin or to any extremity shall not exceed 50 rem. Based on the above TEDE and organ doses, the bounding leaking canister accident, involving maximum leakage of a TranStor canister containing failed BWR fuel, does not exceed the limits specified in 10 CFR 72.106(b). Note that although the consequences have been evaluated, this is not considered to be a credible event for the PFSF.

As an evaluation of the potential doses from environmental pathways following deposition of material in the plume, a pathway analysis using the RESRAD computer code (Reference 36) was next conducted. The first step of this evaluation was to estimate the amount of material deposited on the ground from the plume. This estimate was made assuming that the effluent concentration in a given sector is uniform across the sector at a given distance, as described in Regulatory Guide 1.111 (Reference 37).

[3] Although no such credit has been taken, if credit were taken for a 10% canister release fraction of volatiles and particulates (with 90% of volatiles and particulates released from the fuel rods assumed to be held up in the canister by plateout/deposition and unavailable for release), based on Table XIX of Reference 25, and if credit were also taken for the 5% respirable fraction of particulate fission products released from inside the fuel rods in accordance with Table XX of Reference 25, then the TEDE would be 2.70 mrem to the individual at the owner controlled area fence. The CDE to the maximally exposed organ, the lungs in this case, would be 14.7 mrem.

Using a straight-line trajectory model, this approach requires that the relative deposition rate should be divided by the arc length of the sector at the given downwind distance being considered to estimate deposition. The value of relative deposition (m^{-1}) was obtained from Figure 6 of Regulatory Guide 1.111, with the resulting value of 8.0 E-5 m^{-1} at 500 meters downwind. Deposition estimates were made for each of the radionuclides in the source term. These values, in units of pCi/m^2, were next modified to units of pCi/g to match the input requirements of the RESRAD code, by assuming a soil density of 1.5 E+6 g/m^3 and uniform contamination of the soil to a depth of 1 cm.

The exposure scenario considered in the RESRAD analysis includes direct exposure to contaminated ground, inhalation of resuspended radioactive material, ingestion of milk and beef following grazing, and ingestion of soil. This scenario is considered to be a conservative representation of the land use conditions and environment of the land surrounding the PFSF. 2,000 hours/year occupancy time was assumed at the 500 meter distance along the owner controlled area fence. Although natural vegetation is quite sparse, it is conservatively assumed that the RESRAD default values for fodder intake are met both for the dairy and beef cattle. Default values for human consumption provided in RESRAD for air, milk, beef, and soil were assumed (with the inhalation value reduced from the default value by a factor of 0.228 (2000 hrs / 8760 hrs) to account for partial occupancy). The default values include inhalation of 1,918 m^3 of air with a mass loading factor for air of 2.0 E-4 g/m^3, ingestion of 92 liters of milk, ingestion of 63 kg of beef, and ingestion of 36.5 g soil. The resulting TEDE for the accident case was 2.70 mrem/yr at 500 meters downwind. This dose is a small fraction of the inhalation plus submersion doses identified above, and well below the 5 rem TEDE accident limit imposed by 10 CFR 72.106(b). The dominant exposure pathway was determined to be external exposure to contaminated land and the radionuclide with the largest contribution to the dose was Co-60.

8.2.7.4 Recovery Plan for a Hypothetical Canister Breach

This section has been removed in accordance with the NRC's Interim Staff Guidance-3 (ISG-3, Reference 38), which indicates that recovery from non-mechanistic failures of the confinement boundary by such means as over-packs or dry transfer systems would not be considered and evaluated in the licensing process.

8.2.8 100% Blockage of Air Inlet Ducts

Complete blockage of the air inlet ducts is classified as Design Event IV as defined by ANSI/ANS-57.9.

8.2.8.1 Cause of Accident

This event involves postulated complete blockage of all four storage cask air inlet ducts. Heat is normally removed from the canister shell by natural convection, and the heated air flows up the annulus by natural convection to four top outlet ducts, where the hot air exits the storage cask.

Since the HI-STORM storage casks have four air inlet ducts 90° apart and the TranStor storage casks have four air inlet ducts, with two located on opposing sides of the cask, it is highly unlikely that all air inlet ducts could become blocked by blowing debris, snow, rodents, or other material. A severe windstorm could possibly blow debris against the bottom of the storage casks and possibly clog one or two of the inlet screens exposed to the wind, but the inlets on the leeward side of the cask would be expected to remain relatively free of dirt and debris. If a large sheet of plastic or a tarpaulin were to blow against a storage cask (which is unlikely since the RA is surrounded by two 8-ft high chain link fences that would be expected to catch such items), it could wrap partially around the storage cask and block, or partially block, the air inlet ducts on the windward side, but ducts on the opposite side would be expected to remain open.

One means of cutting off normal convection airflow would be a flood in which the height of the water exceeded the tops of the air inlet ducts. However, since the PFSF location and design assures that the upper surfaces of the storage pads are at an elevation above the elevation of the probable maximum flood in this area, blockage of the inlet ducts by flooding is not credible.

8.3 SITE CHARACTERISTICS AFFECTING SAFETY ANALYSIS

Site characteristics have been considered in the formation of the bases for these safety analyses. The PFSF site layout was considered in determining conservative χ/Q atmospheric dispersion factors to estimate doses from accidents involving postulated and hypothetical releases of radioactivity to a hypothetical individual located at the closest point of the OCA boundary to the source of radioactivity for the duration of the releases. The site location, relative to the nearest major highway, was considered in the assessment of effects of postulated explosions resulting from transportation accidents.

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 day/night average maximum temperatures that could occur over a period of several days (Reference 4).

Regional and site geology and seismology were used to define the design basis ground motion. Regional meteorology was considered in the determination of the design basis tornado parameters (Reference 15). The evaluation of the potential for fires is based on characteristics of the area surrounding the concrete storage pads, as well as the systems that will be used to transfer canisters and storage casks.

Information associated with aircraft flights in the vicinity of the PFSF, presented in Section 2.2 of this SAR, is based on data obtained from the Dugway Proving Grounds and its associated Michael Army Air Field and on flight path data issued by the Federal Aviation Administration (FAA) and the National Oceanic and Atmospheric Administration (NOAA). As discussed in Section 2.2, the calculated probability of an aircraft impacting the PFSF is below the applicable guidance and therefore is not considered to be a credible event.

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17. Regulatory Guide 1.91, Evaluations of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants, Revision 1, U.S. NRC, February 1978.

18. IAEA Safety Standards, Regulations for the Safe Transport of Radioactive Material, IAEA Safety Series No. 6, 1985.

19. Gregory, J.J., et. al., Thermal Measurements in a Series of Large Pool Fires, SAND 85-1096, Sandia National Laboratories, August, 1987.

20. Topical Safety Analysis Report for the Holtec International Storage, Transport, and Repository Cask System, (HI-STAR 100 Cask System), Holtec Report HI-951251, Docket 71-9261, Revision 4, September 1996.

21. UCID-21246, Dynamic Impact Effects on Spent Fuel Assemblies, Lawrence Livermore National Laboratory, Chun, Witte, Schwartz, October 20, 1987.

22. ACI-349, Code Requirements for Nuclear Safety-Related Concrete Structures, American Concrete Institute, September 1985.

23. TranStor Basket Handling and Dead Weight Load Analysis, Sierra Nuclear Corporation Design Calculation No. TSL-1.10.06.60, Rev. 1; March 6, 1997.

24. NUREG-1536, Standard Review Plan for Dry Cask Storage Systems, Final Report, January 1997.

25. SAND80-2124, Transportation Accident Scenarios for Commercial Spent Fuel, Sandia National Laboratories, February 1981.

26. Passive/Evolutionary Regulatory Consequence Code (PERC2), Version 0, Level 1; Computer Code Designator NU-226.

27. Trojan ISFSI Safety Analysis Report, Trojan Nuclear Plant, Portland General Electric Company, Revision 0, Docket No. 72-17.

28. Geomatrix Consultants, Inc, Deterministic Earthquake Ground Motions Analysis, Private Fuel Storage Facility, Skull Valley, Utah, prepared by Geomatrix Consultants, Inc. and William Lettis & Associates, Inc., GMX#3801.1 (Rev. 0), March 1997.

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

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.

36. RESRAD Computer Code, Version 5.82 for Windows.

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.

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facilities of the individual nuclear power plant licensees.

Over 50 engineers have been involved in one or more phases of the project. They include nuclear engineers up to the Ph.D. level, several with 20 or more years of industry experience; operations personnel with direct fuel handling experience; and health physics staff with plant experience to the level of Radiation Protection Manager.

Security and mechanical design as well as instrumentation design oversight has been provided by staff at several member utilities. Safety Analysis Report preparation and review were a joint effort between these staff members and the A/E. Resumes of PFSLLC support personnel are on file with the PFSLLC.

9.1.2 Operating Organization, Management, and Administrative Control System

The following section describes the PFSLLC organization during the operational phase of the project.

9.1.2.1 On-Site Organization

Figure 9.1-3 details staff composition and job functions. During the operational phase, the Board of Managers has overall responsibility for safe operation of the PFSF, and the authority to ensure continued safe operation. The General Manager shall also function as the Chief Operating Officer during the operational phase, responsible to the Board of Managers for managing the PFSF in a manner that ensures safe and efficient operations and maintenance activities. The functions represented by the PFSLLC organization have the authority to control various aspects of the PFSF including engineering and design, quality assurance, fuel accountability, maintenance, radiation protection, training, operations, and decommissioning. This organization will ensure the

continued safe operation of the PFSF during all normal, off-normal, and accident conditions.

Staff members in the various departments provide backup for the lead person in their respective department. Each specialty has more than one qualified individual and they are responsible for performing the lead responsibilities during the absence of the principal. The General Manager/Chief Operating Officer shall designate a lead person as a backup during his/her absence and rotate this responsibility among various leads to develop senior capability for site direction. Training will be available to General Plant Workers for the various technical specialties should an opening occur.

9.1.2.1.1 Safety Review Committee

As specified in the PFSF Technical Specifications, the PFSF Safety Review Committee is responsible for reviewing and advising the Board of Managers on all matters relating to Structures, Systems, and Components (SSCs) Important to Safety. The Safety Review Committee is totally independent of the Operational Review Committee, with approval authority and responsibilities which include, but are not limited to, the review and approval of:

- Safety evaluations for procedures and changes thereto,
- Changes to SSCs classified as Important to Safety,
- Tests or experiments involving SSCs classified as Important to Safety,
- Review of Quality Assurance Audits related to safety
- Proposed changes to the technical specifications or the license, and
- Violations of codes, regulations, orders, license requirements, or internal procedures/instructions which pertain to SSCs classified as Important to Safety.

(EP). This includes responsibility for ensuring the adequacy of EP implementing procedures, that PFSF site personnel are adequately trained in emergency response, that emergency response facilities and equipment are maintained in a state of readiness, and for coordination with offsite agencies and support organizations who may be called upon to provide assistance in the event of an emergency. This position will coordinate drills and exercises, as discussed in the EP, in which individuals demonstrate the ability to perform assigned emergency response functions.

In addition to emergency preparedness, this position is also responsible to the General Manager/Chief Operating Officer for conduct of the overall PFSF site training program, including general orientation training, training on operating/maintenance procedures, and radiological training. The Emergency Preparedness Coordinator is a qualified radiation protection technician, capable of providing backup support to Radiation Protection with radiological monitoring functions on an as-needed basis.

9.1.3 Personnel Qualification Requirements

The staff individual qualification requirements are drafted for the minimum levels. Actual functional resumes will be available once staff is selected during construction in advance of operation under the License.

9.1.3.1 Minimum Qualification Requirements

9.1.3.1.1 General Manager/Chief Operating Officer

At the time of appointment to the active position, the General Manager/Chief Operating Officer shall have ten years of responsible experience within the nuclear power industry. A maximum of four years of the ten years may be fulfilled by academic

training on a one-for-one basis. To be acceptable, this academic training shall be in an engineering or scientific field generally associated with the nuclear industry.

The General Manager/Chief Operating Officer shall have a recognized Baccalaureate (Bachelor) or higher degree in an engineering or scientific field generally associated with nuclear power production, fuel storage or radiation protection.

9.1.3.1.2 Radiation Protection Manager

At the time of appointment to the active position, the responsible person shall have a minimum of ten years experience in radiation protection within the nuclear power industry. A maximum of four years of this ten years experience may be fulfilled by related technical or academic training on a one-for-one basis. This person shall have a Bachelor or higher degree in radiation protection or a related field.

9.1.3.1.3 Radiation Protection Technicians

Technicians in responsible positions shall have a minimum of four years of working experience in radiation protection. These personnel should also have a minimum of one year of related technical training.

9.1.3.1.4 Lead Mechanic/Operator

At the time of appointment to the active position, this person shall have a high school diploma or equivalent and a minimum of six years of experience in mechanical maintenance. A Lead Mechanic/Operator will also become a certified storage facility operator prior to facility operation, and a certified welder.

9.1.3.1.5 Mechanics

Mechanics shall hold a high school diploma or equivalent and a minimum of four years experience in mechanical maintenance. All mechanics shall become certified facility operators prior to operating cask handling equipment, and shall become licensed locomotive operators if a rail line is developed to the site. One mechanic must become a certified welder to provide backup for the Lead Mechanic/Operator. Mechanics should possess a high degree of manual dexterity and the ability to learn and apply new skills in maintenance operations as they are developed and incorporated into facility operations.

9.1.3.1.6 Lead Instrument and Electrical Technician

At the time of appointment to the active position this person shall have a high school diploma or equivalent and a minimum of six years of experience in instrumentation and electrical work. This person should possess a two-year associate degree in the instrumentation, control and electrical field.

9.1.3.1.7 Instrument and Electrical Technicians

Technicians shall have a high school diploma or equivalent and a minimum of four years of working experience in this field. They should have minimum of one year of related technical training in addition to their experience.

9.1.3.1.8 Lead Quality Assurance Technician

This person shall have a high school diploma or equivalent and a minimum of six years experience within the nuclear power industry in a quality assurance position. This

person should also have two years technical education in this area.

9.1.3.1.9 Quality Assurance Technician and Quality Assurance Auditor

The Quality Assurance Technicians shall have a high school diploma or equivalent and a minimum of four years experience in the quality assurance field within the nuclear power industry.

9.1.3.1.10 Lead Nuclear Engineer

This person shall have a minimum of a Bachelor degree in nuclear engineering and four years experience in the nuclear power industry.

9.1.3.1.11 Nuclear Engineers

This person shall have a Bachelor degree in Nuclear Engineering.

9.1.3.1.12 Security Captain

This person shall be a High School graduate, a qualified weapons instructor (NRA or equivalent), and have received supervisory skills training. Should have 6 years experience in security at least three of which must be at a nuclear facility.

9.1.3.1.13 Emergency Preparedness Coordinator

This person shall have a high school diploma or equivalent and a minimum of two years experience in emergency preparedness. This individual shall also have a minimum of four years of working experience in radiation protection. In addition, this person shall have experience in providing training, including conducting emergency preparedness

training and drills.

9.1.3.2 Personal Qualification Requirements

Each member of the site staff involved with important safety activities will be required to meet the minimum qualifications of the License. Programs for additional site familiarization training and ongoing training and retraining will be maintained to provide a continuously qualified staff. The training program as coordinated by the Emergency Preparedness Coordinator is under the overall direction of the General Manager/Chief Operating Officer.

9.1.4 Liaison with Outside Organizations

During the pre-licensing phase, most of the outside technical support for the PFSF engineering and licensing was employed through the A/E. Oversight was provided by a nuclear engineer on the Technical Committee. The outside organizations providing technical expertise on site selection were directed by the Project Manager; the Chairman of the Board monitored the interface.

The outside organizations supplying the cask storage systems are directed by the Technical Committee and the PFSLLC Project Manager. Their review is performed in accordance with the Quality Assurance Program and is audited by the Quality Assurance Committee.

During construction, the outside organization for installation and construction and the A/E are overseen by the PFSLLC Project Manager. The system to monitor the design includes Technical Committee review and Quality Assurance Committee audits as well as routine telephone conferences.

During the operational phase, the General Manager/Chief Operating Officer shall be responsible for day-to-day contacts with the U.S. Nuclear Regulatory Commission and other regulatory bodies. The authority to hire necessary consulting staff within the guidelines approved by the Board will rest with the General Manager/Chief Operating Officer after consulting the Board Chairman. The acquisition of outside consulting expertise or services will be done in full accordance with the standards outlined in the Quality Assurance Program. All work performed on SSCs that are Important-to-Safety, will be strictly governed by the Quality Assurance Program.

Oversight of the outside organizations which manufacture canisters is provided by the General Manager/Chief Operating Officer and the Nuclear Engineering staff, who will conduct oversight activities in accordance with the Quality Assurance Program. Fabrication of canisters to appropriate standards and storage, transfer and transportation technology is monitored by the nuclear engineering staff. The oversight of outside organizations is audited periodically by the Quality Assurance staff."

9.3 TRAINING PROGRAM

The purpose of the training program is to provide the training necessary to ensure the safe, reliable, and cost effective operation of the storage site and to protect the health and safety of site personnel and the general public.

9.3.1 Program Description

The training program will be developed using a "Systematic Approach to Training" which is based upon analysis of the job performance requirements to establish the knowledge level and skills that are required for each position. Explicit learning objectives and performance measures are generated from this analysis. Training plans are then developed which identify training settings, sequences, and materials required. The training program is implemented by conducting the training activities, documenting the training and evaluating the program's effectiveness.

Job descriptions will detail the training, education, and experience requirements for each position. An individual assessment of the employee's needs will be conducted relative to the identified training requirements for each of the respective positions. Training will consist of classroom and on-the job training (OJT), as appropriate, for all individuals, commensurate with their job duties and responsibilities. Training will be structured to meet specific needs of various management and support groups.

It is the intent to hire individuals with the training, education and experience which enable them to perform the assigned tasks, and to provide additional training, as appropriate. There will be an adequate complement of trained and certified personnel prior to the receipt of spent fuel for storage, and throughout the period of the NRC operating license.

The training program depends on a constant evaluation of the job or task to be performed, the work environment, and the training provided, to determine whether the program is effective in producing and maintaining competent employees. Data from these evaluations are used to identify and correct deficiencies and to accommodate changing needs. Additionally, the effectiveness of the training, and the training program will be evaluated by reviewing written test performances, performance on walk through evaluations, on-the-job training, and feedback from trainees, supervisors, and instructors.

The training program will consist of initial training and continuing or refresher training at a frequency of not less than every two years. The training program will be implemented prior to the receipt of spent fuel at the PFSF and during the life of the facility, until termination of the NRC license.

9.3.2 Initial Training

Initial training will consist of general employee training (GET) and job-specific training to provide individuals with the skill and knowledge required to perform their particular duties and responsibilities. For example: it is anticipated that the mechanics will become licensed locomotive operators, and therefore this training will need to be completed prior to operating the locomotives.

9.3.2.1 General Employee Training (GET)

The following topics will be addressed in GET:

- Facility operations and design,
- Instrumentation and controls,

- Emergency Plan and Procedures,
- Security Plan and Procedures,
- Radiation Control Procedures and Practices, including: The nature and sources of radiation and contamination, interactions of radiation with matter, biological effects of radiation, methods of detecting and controlling radiation and contamination, ALARA concepts, facility access and visitor controls, decontamination procedures, use of monitoring and personal protective equipment, regulatory and administrative exposure and contamination limits, and site specific hazards,
- Environmental Protection,
- Quality Assurance,
- Administrative Procedures,
- Normal and Off-Normal Procedures,
- Safety, and
- Fire Brigade.

9.3.2.2 Job Specific and Certification Training

Individuals who operate equipment and controls that have been identified as "important to safety" in the Safety Analysis Report and in the NRC license must be trained and certified. Supervisory personnel who direct the operation of equipment and controls that are "important to safety" must also be certified. These individuals shall be certified as specified in their respective job descriptions. The Operator Training Program will address the following:

- Canister transfer system design and operations,
- Canister transfer system normal and off-normal procedures,

- Storage facility normal and off-normal procedures,
- Normal and off-normal transportation procedures for on-site transportation package positioning prior to lifting of the shipping cask from the transport vehicle,
- Maintenance,
- Storage cask temperature monitoring system,
- Radiation detection, monitoring, sampling and survey instruments,
- Facility layout and functions,
- Operator responsibility and authority,
- Technical Specifications,
- Normal and emergency communications,
- Transportation,
- Topics covered in General Employee Training (GET), addressed with specific emphasis on operations.

Other tasks that will be unique to this facility may require special training prior to performing them. Some of this training may have to be contracted, either on site or at a vendor's facility.

Whenever additional or job-specific training is required, the "Systematic Approach to Training" (SAT) shall be used to develop the necessary training materials. Exceptions to the use of the SAT method shall be approved on a case-by-case basis by the emergency preparedness coordinator. Training materials will be developed by the site personnel qualified on those particular tasks. Training will also be delivered by individuals qualified on the particular tasks, or by appropriate contractors.

Data collected from test results, job performance results, instructor and trainee critiques will be evaluated and necessary adjustments made to ensure program effectiveness.

9.3.3 Continuing Training

All site personnel will receive GET retraining at a frequency of not more than every two years. The topics selected for GET retraining will include, at a minimum, the subjects covered in initial GET, which appear in Section 9.3.2.

Job Specific and Certification retraining will be provided at a frequency of not more than every two years. Topics for this continuing training may be selected from initial training, NRC bulletins and information notices, major equipment and procedure changes, relevant industry events, and topics designed by the General Manager or requested by other site personnel. All facility processes that could affect the training program should be monitored and analyzed for impact, and the training program adjusted accordingly.

9.3.4 Administration and Records

The Emergency Preparedness Coordinator is responsible for the administration of the training program and for maintaining the training records. The Emergency Preparedness Coordinator will be the primary instructor for GET, and will ensure appropriate, qualified instructors from on site or contracted training services conduct all training activities.

Training records include: dates and hours of training received, outline of subjects (i.e., lesson plan), job performance statements, copies of written examinations, information pertaining to walk-through examinations, practical factors and OJT completed, and re-testing information. Training records will be maintained during the period of an individual's employment and for two years thereafter.

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9.4 NORMAL OPERATIONS

9.4.1 Procedures

Operations important to safety will be conducted at the PFSF in accordance with detailed written and approved procedures. All pre-operational, normal operating, maintenance, testing, surveillance and emergency procedures will be in effect prior to operation of the PFSF. These procedures and changes thereto will be reviewed and approved by the Health Physics amd Quality Assurance organizations, independent of the operating function. Procedures will contain sufficient detail to allow qualified and trained personnel to perform the actions without incident or abnormal event.

9.4.1.1 Categories Of Procedures

9.4.1.1.1 Administrative Procedures

Administrative procedures will provide rules and instructions to all PFSF personnel to provide a clear understanding of operating philosophy and management policies. These procedures will include instructions pertaining to personnel conduct and procedures to develop, review, change and approve the other facility procedures. Administrative procedures include activities to ensure that personnel safety, the working environment, procurement, and other general activities of the facility are carried on at a high degree of readiness, quality and success.

Medical evaluations will be conducted on PFSF staff members who are certified for operations important to safety. NRC Form 396 will be used for the performance of the physical examination and guidance contained in ANSI/ANS 3.4-1983 or later will be followed to certify that the individual's fitness and the applicant's physical condition and

general health are such that they might not cause operational errors endangering public health and safety

9.4.1.1.2 Radiation Protection Procedures

Radiation protection procedures are used to implement the radiation control program. Radiation protection procedures will assure compliance with 10 CFR 20 and ALARA principles. The radiation protection procedures will describe the acquisition of data, use of equipment, and qualifications and training of personnel to perform radiation surveys, measurements, and evaluations for the assessment and control of radiation hazards associated with the PFSF.

Entrance to and work performed inside the PFSF restricted area will require a radiation work permit and will be reviewed and controlled by radiation protection personnel. Existing radiation protection procedures from the utilities utilizing the PFSF will be used in developing PFSF specific procedures. The radiation protection procedures will ensure safety of personnel and operations at the PFSF.

The operation and use of radiation monitoring instrumentation at the PFSF, including area monitors, personnel monitoring equipment, air sampling, and measurement and sampling techniques will be described in written procedures.

9.4.1.1.3 Maintenance and Surveillance Procedures

Procedures will be established for performing preventative and corrective maintenance and for surveillance of PFSF equipment and instrumentation. Preventative maintenance and surveillance, including calibrations, will be performed on a periodic basis to preclude the degradation of PFSF systems, equipment, and components.

Corrective maintenance will be performed to rectify any unexpected system, equipment, or component malfunction, and will be initiated as the need arises.

Procedures will describe the expertise or training required to perform tasks important to safety, special equipment needed, and operational controls. Any projected radiation exposure will be identified, along with the ALARA principles to be applied to minimize such exposure.

9.4.1.1.4 Operating Procedures

The operating procedures will provide instructions for all routine and projected contingency (off-normal) operations, including handling, loading, transporting, and storing of spent fuel, and for all other operations important to safety. Operating procedures will include chemical safety, off-normal occurrences, operation of the cask temperature monitoring system and all operations identified in the technical specifications. The requirements for certification of personnel operating equipment and controls important to safety will be specified in the operating procedures.

9.4.1.1.5 Quality Assurance Procedures

Quality assurance procedures will prescribe the necessary elements of quality oversight to ensure activities important to safety are conducted in a controlled manner, in accordance with 10 CFR 72.122 (a) and all applicable regulations, the PFSF license and technical specifications, the radiation protection program, and approved procedures. The quality assurance procedures will clearly communicate that the responsibility for quality rests with each individual, employee or visitor, who enters the facility.

9.4.1.2 Procedure Preparation

Procedures will be generated for all activities important to safety, and will include the following format and depth of coverage:

- Purpose and role in broader scope function
- Personnel required per shift by staff position and general function (e.g., function performance, QA, radiation monitoring)
- Continuous or single (or double) shift operation
- Prerequisites for readiness, such as
  - calibrations to be performed or checked
  - instrumentation to be on hand
  - tools and special equipment to be on hand
  - notifications (with lead times)
  - check/set equipment controls (e.g., physical travel limits for overhead crane)
  - check environmental or other monitors for acceptable range
  - identification of subject(s) of function (e.g., canister to be transferred, cask to be retrieved)
  - log and forms to be completed on hand
  - preceding function
- Series of operations, including results, projected times, projected instrument and gauge readings, controls to be used in performance (e.g., torque, time at pressure, and threshold limits requiring contingency actions (such as hold, initiating a contingency sequence, notification))
- Records to be completed during operation and distribution
- Record and notification upon Completion
- Identification of following function

9.4.1.3 Training On Procedures

All personnel involved in activities important to safety will be trained on the associated procedures prior to conducting the activity. Formal training of personnel on facility procedures will be substantially complete prior to the receipt of radioactive materials at the PFSF. Personnel performing activities important to safety will be certified to perform such functions and will undergo refresher training and testing a minimum of every two year

9.4.2 Records

9.4.2.1 Records Management System

Records relating to the historical operation of the facility will be maintained by the Administrative organization, under the responsibility of the Administrative Assistant. Records will be stored in the Administration Building, with copies of records required to be maintained in duplicate, as noted below, maintained in the Security and Health Physics Building. Unless otherwise noted, records will be maintained until termination of the facility license by the NRC.

9.4.2.2 Records To Be Maintained

The following PFSF records will be maintained in accordance with procedures developed specifically for the PFSF. The regulatory reference for each category of records is provided in parentheses.

1. Radiation protection program and survey records(10 CFR 20.2101 to 20.2110);

2. Records associated with reporting of defects and noncompliance (10 CFR 21.51);
3. Records important to decommissioning (10 CFR 72.30(d)), consisting of:
   - Records of spills or off-normal occurrences involving the spread of contamination,
   - As-built drawings and modifications of structures and equipment involved in the use and/or storage of radioactive materials, and locations of possible inaccessible contamination,
   - A document, which is updated a minimum of every 2 years, containing a list of all areas designated at any time as restricted areas as defined in 10 CFR 20.1003, and a list of all areas outside of restricted areas involved in a spread of contamination,
   - Records of decommissioning cost estimates and the funding method used.
4. Records of changes to the physical security plan made without prior NRC approval, which must be maintained for a period of three years from the date of the change (10 CFR 72.44(e) and 72.186(b));
5. Records of changes, tests and experiments, and of changes to procedures described in the SAR (10 CFR 72.48(b)(1));
6. Records showing the receipt, inventory, location, disposal, acquisition and transfer of all spent fuel (10 CFR 72.72(a)), which must be maintained in duplicate as long as the spent fuel is stored at the PFSF and for five years after it is removed or transferred from the PFSF;
7. A copy of the current inventory of all spent fuel in storage at the PFSF (10 CFR 72.72(b), which must be kept in duplicate;
8. A copy of the current material control and accounting procedures (10 CFR 72.72(c));

9. Other records required by license conditions or by NRC rules, regulations or orders (10 CFR 72.80);
10. Records of the occurrence and severity of important natural phenomena that affect the PFSF design must be retained until the license is issued (10 CFR 72.92(b));
11. Quality assurance records, including records pertaining to the design, fabrication, erection, testing, maintenance and use of structures, systems and components important to safety, and results of reviews, inspections, tests, audits, monitoring of work performance, and material analyses (10 CFR 72.174);
12. A copy of the current physical security plan, plus any superseded portions of the plan, which must be maintained for three years after a change is made (10 CFR 72.180);
13. A copy of the current safeguards contingency plan procedures, plus any superseded portions of the procedures, which must be maintained for three years after a change is made (10 CFR 72.184);
14. Operating records, including maintenance, alterations or additions made;
15. Records of off-normal occurrences and events;
16. Environmental survey records;
17. Records of employee qualifications and certifications, including NRC Form 396 physical examinations" as required for those PFSF staff members who are certified for operations important to safety;
18. Record copies of:
    - SAR and updates,
    - Reports of accidental criticality or loss of special nuclear material,
    - Material status reports,
    - Nuclear material transfer reports,
    - Reports of pre-operational test acceptance criteria and results,

- Procedures,
- Environmental Report, and
- Emergency Plan.

19. Construction Records

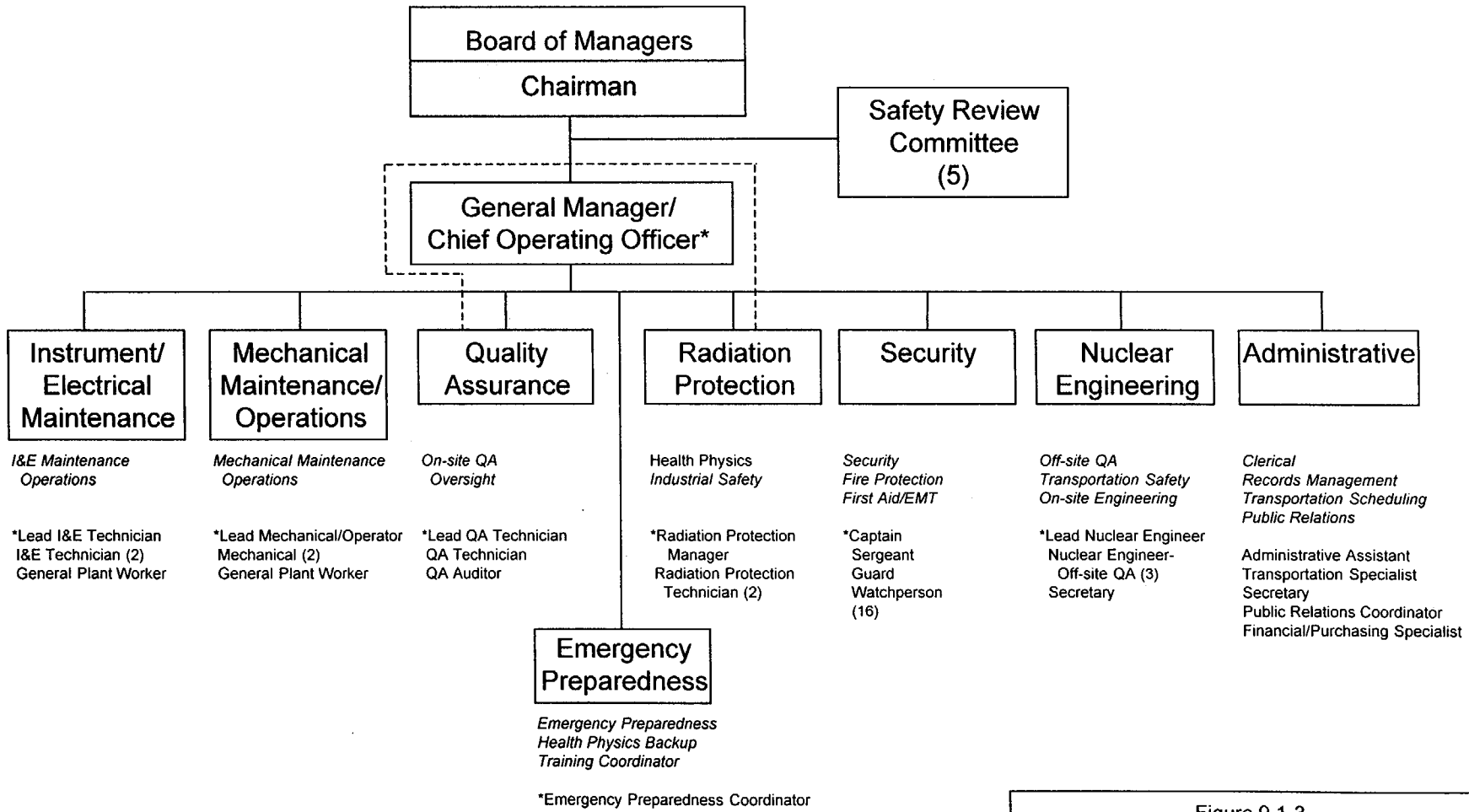

Figure 9.1-3

OPERATIONAL ORGANIZATION

PRIVATE FUEL STORAGE FACILITY
SAFETY ANALYSIS REPORT

D39903 - Rev 3