Private Fuel Storage, L.L.C.

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April 16, 2001

ERRATA TO CORRECT CHAPTER 2 OF THE PFSF LICENSE APPLICATION
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: PFS letter, Parkyn to U.S. NRC, License Application Amendment No. 22, dated March 30, 2001

The above reference transmitted Amendment No. 22 to the Private Fuel Storage (PFS) License Application. PFS has identified several minor corrections/clarifications that need to be made to Chapter 2 of the SAR for consistency and has developed an errata package. The enclosed errata package consists of minor changes to 5 pages of text and 2 tables. The changes are described briefly below:

- Page 2.6-65 was corrected to indicate that the factor of safety for sliding stability of the storage pads is 2.3 for Load Case III and 1.25 for Load Case IV
- Page 2.6-71, calculated displacements of the storage pads during various load combinations were corrected to indicate a range of 1.9 to 2.2
- Page 2.6-74, the gross allowable bearing capacity of the Canister Transfer Building for static loads for the soil strength was corrected to >50 ksf
- Page 2.6-76, the minimum factor of safety against a dynamic bearing capacity failure for the Canister Transfer Building was corrected to 2.4 ksf
- Page 2.6-109 was corrected to indicate that the soil cement directly beneath the storage pads will be designed and constructed to have a static elastic modulus of less than approximately 75,000 psi
- Table 2.6-7, values were corrected for Case IVC in regards to allowable bearing capacity of cask storage pads based on the design basis ground motions
- Table 2.6-9, several values were corrected for Case IB (static effective strength) in regards to allowable bearing capacity of the Canister Transfer Building based on static loads

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We apologize for any inconvenience caused by these errors. If you have any questions, please contact me at 303-741-7009.

Sincerely,

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

Enclosure
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dynamic lateral earth pressure.

The value of c is based on the results of the direct shear tests that were performed on specimens obtained from a depth of 5 to 6.3 ft from Sample U-1 in Boring C-2, which was drilled in the pad emplacement area. These test results are consistent with the results obtained from the direct shear tests that were performed on samples obtained from within the Canister Transfer Building area (Borings CTB-6 and CTB-S). All of these direct shear test results are reported in Attachments 7 and 8 of Appendix 2A.

The sliding stability was checked for Load Cases III and IV, conservatively assuming that 100% of the dynamic forces due to the earthquake act in both the N-S and Vertical directions at the same time. In determining the resisting forces in these analyses, no credit is taken for passive resistance acting on the embedded pad. The resulting factor of safety was ~2.3 for Load Case III and ~1.25 for Load Case IV, which provides an adequate margin against sliding.

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

The horizontal driving force in these analyses includes the inertial forces of the pad and the maximum cask driving forces reported in Calculation 05996.02-G(PO17)-2 (CEC, 2001) that are due to the PSHA 2,000-yr return period earthquake. The

dynamic loads due to soil pressures acting on the embedded pad were also included, calculated based on the Mononobe-Okabe method, as described in Seed and Whitman (1970). The driving forces were calculated based on the peak vertical and peak horizontal accelerations; i.e., no credit was taken for the fact that these peaks are not expected to occur at different times.

The driving force due to dynamic active earth pressures acting on the pad in the E-W direction is greater than that acting in the N-S direction, because the dimension of the pad in the N-S direction (67 ft) is greater than twice the width in the E-W direction (30 ft). Therefore, ignoring passive resistance, sliding will be more critical in the E-W direction. The maximum dynamic cask driving force, however, acts in the N-S direction. To be conservative, this analysis assumed that the driving force due to dynamic active earth pressures calculated for the E-W direction acts in the N-S direction. However, the maximum horizontal force that can be applied to the top of the pad by the casks is limited to the maximum value of the coefficient of friction between the cask and the top of the pad, which equals 0.8, multiplied by the cask normal force.

As indicated above, these analyses are very conservative for a number of reasons. They combine the maximum horizontal and vertical forces of the earthquake, rather than using reduced values to account for the fact that the peaks in these motions are not expected to occur at the same time. They also conservatively use the shear strength measured in the static direct shear tests; i.e., no credit is taken for the increase in this strength that is applicable for dynamic loadings, as discussed in Section 2.6.1.11.5, "Dynamic Strength of Cohesive Soils." Therefore, these analyses yield lower-bound factors of safety against sliding where the pads are supported on clayey soils.

These analyses illustrate that if the cask storage pads were constructed directly on the silty clay/clayey silt layer, they would have an adequate factor of safety against sliding

LOAD COMBINATION				DISPLACEMENT
Case IIIA	40% N-S,	-100% Vertical,	40% E-W.	2.2 inches
Case IIIB	40% N-S,	-40% Vertical,	100% E-W.	1.9 inches
Case IIIC	100% N-S,	-40% Vertical,	40% E-W.	1.9 inches

For this hypothetical, worst case scenario, the estimated relative displacement of the cask storage pads ranges from ~1.9 inches to 2.2 inches. Because there are no connections between the pads or between the pads and other structures, displacements of this magnitude, were they to occur, would not adversely impact the performance of the cask storage pads. There are several conservative assumptions that were made in determining these values and, therefore, the estimated displacements represent upper-bound values.

The soils in the layer that are assumed to be cohesionless (the one ~10 ft below the pads) are clayey silts and silts, with some sandy silt. To be conservative in this analysis, these soils are assumed to have a friction angle of 30°. However, the results of the cone penetration testing (Appendices D & F of ConeTec, 1999) indicate that these soils have ϕ values that generally exceed 35 to 40°. These high friction angles likely are the manifestation of the cementation that was observed in many of the specimens obtained in split-barrel sampling and in the undisturbed tubes that were obtained for testing in the laboratory. Possible cementation of these soils is also ignored in this analysis, adding to the conservatism.

In addition, this analysis postulates that cohesionless soils exist directly at the base of the pads. In reality, the surface of these soils is 10 ft or more below the pads, and it is not likely to be continuous, as the soils in this layer are intermixed. For the pads to

slide, a surface of sliding must be established between the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft below the pads, through the overlying clayey layer, and daylighting at grade. As shown in the analysis preceding this section, the overlying clayey layer is strong enough to resist sliding due to the earthquake forces. The contribution of the shear strength of the soils along this failure plane rising from the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft to the resistance to sliding is ignored in the simplified model used to estimate the relative displacement, further adding to the conservatism.

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

2.6.1.12.2 Stability and Settlement Analyses—Canister Transfer Building

Stability and settlement analyses of the Canister Transfer Building were performed to confirm the adequacy of the structure and its foundation. Calculation 05996.02-G(B)-13 (SWEC, 2001c) evaluated the stability of the Canister Transfer Building and determined it is stable with respect to bearing capacity, overturning, and sliding due to static and dynamic load conditions. Calculation 05996.02-G(C)-14 (SWEC, 2000b) evaluated the soil settlements due to static load conditions and found the resulting building settlements to be uniform and small and to have little effect on the structure. Calculation 05996.02-G(B)-11 (SWEC, 2000a) evaluated the soil settlements due to dynamic load conditions and also found the resulting building settlements to be small and to have little effect on the structure. In summary, the stability and settlement analyses of the Canister Transfer Building indicate that the building is stable and will

reinforced concrete mat foundation 5'-0" thick. The interior partitions that make up the low level waste holding area will be constructed of concrete or concrete masonry. The equipment and office areas on the east side of the building will utilize steel-framed partition walls covered with gypsum board. The total weight (static load) of the building and foundation is approximately 89,400 kips (Calculation 05996.02-SC-5, SWEC, 2001d) or 44,700 tons.

Bearing Capacity of the Canister Transfer Building

The bearing capacity of the Canister Transfer Building foundation was determined using the general bearing capacity equation and associated shape, depth, and inclination factors, as presented in Winterkorn and Fang (1975). Refer to Calculation 05996.02-G(B)-13 (SWEC, 2001c) for details. These analyses are based on the strength parameters for the silty clay/clayey silt layer directly underlying the mat. Conservatively ignoring the presence of the denser layers, which start at a depth of ~25 to 30 ft, these analyses demonstrate that there is an adequate factor of safety against a bearing capacity failure for both static and dynamic loadings. They included determination of factors of safety against a bearing capacity failure of the foundation due to static loads and due to static plus dynamic loads from the design basis ground motion (PSHA 2,000-yr return period earthquake). The dynamic bearing capacity analyses are discussed in detail in the section below titled *"Dynamic Bearing Capacity of the Canister Transfer Building."*

Static Bearing Capacity of the Canister Transfer Building

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

Case IA Static using undrained strength parameters (ϕ = 0° & c =3.18 ksf).
Case IB Static using effective-stress strength parameters (ϕ = 30° & c = 0).

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 6.5 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and c = 3.18 ksf, to model the end of construction. This is the average undrained strength of the soils in the upper ~25 to ~30 ft, as discussed in Section 2.6.1.11.2. Using the estimated effective-stress strength of $\phi = 30°$ and c = 0, results in higher allowable bearing pressures. As shown in Table 2.6-9, the gross allowable bearing capacity of the Canister Transfer Building for static loads for this soil strength is greater than 50 ksf.

Settlement of the Canister Transfer Building

Analyses were performed to estimate the settlement of the Canister Transfer Building for the static dead and live loads in Calculation 05996.02-G(C)-14 (SWEC, 2000b). A total building settlement of approximately 3 inches is estimated over the life of the building. The settlement will be generally uniform. Of the total building settlement, approximately 1.9 inches will occur within a few years after construction and an additional 1.1 inches will occur during the life of the building. These analyses were performed using the results of the consolidation tests that are included in Attachment 2 of Appendix 2A.

As indicated in Section 2.6.1.12.1 regarding the settlement analyses of the storage pads, this settlement represents an upper-bound estimate of the settlement, because it was developed assuming that the consolidation characteristics that were measured for the clayey soils at a depth of about 10 ft are applicable for the entire upper layer. The SPT data from the borings and the CPT results indicate that the soils become stiffer within the 10 to 20 ft depth zone. Additional consolidation tests performed on samples

obtained from depths of about 25 ft in the Canister Transfer Building area, reported in Attachment 6 of Appendix 2A, indicate that the soils at that depth are less compressible than those used to estimate these settlements.

Dynamic Bearing Capacity of the Canister Transfer Building

The dynamic bearing capacity was analyzed in Calculation 05996.02-G(B)-13 (SWEC, 2001c) using the dynamic loads for the building that were developed in Calculation 05996.02-SC-5, (SWEC, 2001d). The development of these dynamic loads is described in Section 4.7.1.5.3. As in the structural analyses discussed in Section 4.7.1.5.3, the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions.

Table 2.6-10 presents the results of the bearing capacity analyses for the following cases, which include static loads plus dynamic loads due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the average undrained strength of the soils in the upper ~25 to ~30 ft, as discussed in Section 2.6.1.11.2 (ϕ = 0° and c = 3.18 ksf). As indicated above, for these cases including dynamic loads from the design basis ground motion, the minimum acceptable factor of safety is 1.1.

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

Table 2.6-10 indicates the minimum factor of safety against a dynamic bearing capacity failure was obtained for Load Case II, the load combination of full static, 100% of the seismic forces acting in the N-S direction and the E-W direction and 0% in the upward direction. This load case resulted in an actual soil bearing pressure of 2.4 kips per square foot (ksf), compared with an ultimate bearing capacity of 13.2 ksf. The resulting factor of safety against a bearing capacity failure for this load case is ~5.5, which is much greater than the minimum allowable factor of safety for seismic loading cases. Therefore, the Canister Transfer Building has an adequate factor of safety against a dynamic bearing capacity failure.

Overturning Stability of the Canister Transfer Building

The overturning stability of the Canister Transfer Building was analyzed in Calculation 05996.02-G(B)-13 (SWEC, 2001c) using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads are listed in Table 2.6-11, and they were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (SWEC, 2001d), which is described in Section 4.7.1.5.3. The masses and accelerations of the joints used in the model of the Canister Transfer Building in Calculation 05996.02-SC-5 are listed on the left side of Table 2.6-11, and the resulting inertial forces and associated moments are listed on the right. Based on building geometry and the forces and moments shown in Table 2.6-11, overturning is more critical about the N-S axis (~279.5 ft) than about the E-W axis (~240 ft).

The factor of safety against overturning is defined as:

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

The resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The weight of the building

the silty clay/clayey silt layer. The soil cement around the storage pads and Canister Transfer Building will be designed and constructed to have a minimum unconfined compressive strength of 250 psi (and possibly as high as 340 psi based on Section 2.6.1.12.1) and quality assurance testing will be performed during construction to demonstrate that this minimum strength is achieved. The soil cement directly beneath the storage pads will be designed and constructed to have an unconfined compressive strength of ~100 psi with static elastic modulus of less than ~75,000 psi. Therefore, the resistance to sliding due to loadings from the design basis ground motion will be enhanced by constructing the cask storage pads on a properly designed and constructed soil-cement subgrade. See the section titled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement" in 2.6.1.12.1 for additional details.

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

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

potentially liquefiable sandy soils under the foundations of two 900-MW nuclear power plants in Koeberg, South Africa (Dupas and Pecker, 1979). The strength of soils can be improved markedly by the addition of cement. The eolian silt at the site is similar to the soils identified as Soil A-4 in Nussbaum and Colley (1971), Soils 7 and 8 in Balmer (1958), and Soil 4 in Felt and Abrams (1957). As indicated for Soil A-4 in Table 5 of Nussbaum and Colley (1971), the addition of just 2.5% cement by weight to the silt increased the cohesion from 5 psi (720 psf) to 30 psi (4,320 psf). The cohesion for Soils 7 and 8 also were increased significantly by the addition of low percentages of cement, as shown on Tables VI and VII of Balmer (1958). Figure 10 in Felt and Abrams (1957) illustrates the continued strength increase over time for these soil-cement mixtures. Other examples of soil-cement strength increases over time are presented in Figure 4.3 of ACI (1998), Table 6 of Nussbaum and Colley (1971), and Figures 6 and 7 of Dupas and Pecker (1979). Therefore, the soil cement will be much stronger than the underlying silty clay/clayey silt and the strength will increase with time, providing an improved foundation material. This will provide additional margin against sliding compared to the original plan to construct the pads directly on the silty clay/clayey silt layer.

As shown in the section titled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement" in Section 2.6.1.12.1 above, the shear resistance required at the base of the pads can be provided easily by the passive resistance of the soil cement acting against the vertical side of the foundation and by bond between the pad foundation and soil-cement contact and the cohesive strength of the soil cement. Shear resistance will be transferred through the approximately 2-ft thick soil-cement layer and into the underlying silty clay/clayey silt subgrade. Additional resistance will be provided by the continuous layer of soil cement under and between the pads; therefore, shear resistance requirements within the silty clay/clayey silt layer will be less with the soil-cement layer compared to the original plan to construct the pads directly on the silty clay/clayey silt without the proposed soil-cement layer.

TABLE 2.6-7

SUMMARY – ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Forces Due to Design Earthquake: PSHA 2,000-Yr Return Period

Case	F_V	$EQ_{H\ N-S}$	$EQ_{H\ E-W}$	$\Sigma M_{@N-S}$	$\Sigma M_{@E-W}$	β_B $EQ_{H\ E-W}$	β_L $EQ_{H\ N-S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
II	3,757	2,671	2,671	26,982	26,982	35.4	35.4	5.34	**4.85**	7.2	7.2	15.6	52.6	**4.56**	**1.2**
IIIA	1,146	953	953	10,793	10,793	39.8	39.8	9.03	**8.21**	9.4	9.4	11.2	48.2	**2.13**	**4.2**
IIIB	2,712	1,068	2,291	26,982	10,793	40.2	21.5	5.26	**4.78**	9.9	4.0	10.1	59.0	**4.55**	**1.2**
IIIC	2,712	2,291	1,068	10,793	26,982	21.5	40.2	9.45	**8.59**	4.0	9.9	22.0	47.1	**2.61**	**3.6**
IVA	6,368	1,068	1,068	10,793	10,793	9.5	9.5	11.57	**10.51**	1.7	1.7	26.6	63.6	**3.76**	**3.1**
IVB	4,801	1,068	2,671	26,982	10,793	29.1	12.5	8.51	**7.73**	5.6	2.2	18.8	62.5	**4.09**	**2.1**
IVC	4,801	2,671	1,068	10,793	26,982	12.5	29.1	10.05	**9.13**	2.2	5.6	25.5	55.8	**3.38**	**3.0**

c =	**2,200**	Undrained strength (psf)
ϕ =	**0.0**	Friction angle (deg)
γ =	**80**	Unit weight of soil (pcf)
B =	**30**	Footing width (ft)
L =	**67**	Footing length (ft)
D_f =	**3.0**	Depth of footing (ft)
γ_{surch} =	**100**	Unit weight of surcharge (pcf)
FS =	**1.1**	Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)

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

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

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

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

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

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

TABLE 2.6-9

SUMMARY - ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Based on Static Loads

Case	F_V k	$EQ_{H\,N\text{-}S}$ k	$EQ_{H\,E\text{-}W}$ k	$\Sigma M_{@N\text{-}S}$ ft-k	$\Sigma M_{@E\text{-}W}$ ft-k	β_B $EQ_{H\,E\text{-}W}$ deg	β_L $EQ_{H\,N\text{-}S}$ deg	GROSS q_{ult} ksf	GROSS q_{all} ksf	e_B ft	e_L ft	EFFECTIVE B' ft	EFFECTIVE L' ft	EFFECTIVE q_{actual} ksf	FS_{actual}
IA - Static Undrained Strength	97,749	0	0	0	0	0.0	0.0	19.63	**6.54**	0.0	0.0	240.0	279.5	**1.46**	**13.47**
IB - Static Effective Strength	97,749	0	0	0	0	0.0	0.0	169.92	**56.64**	0.0	0.0	240.0	279.5	**1.46**	**116.61**

c = **3,180** Undrained strength (psf), ϕ=0.

ϕ = **30** Effective stress friction angle (deg), c=0.

γ = **90** Unit weight of soil (pcf)

B = **240** Footing width (ft)

L = **279.5** Footing length (ft)

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

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

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

F_V = Vertical load (Static + EQ_V)

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

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

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

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

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

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