Private Fuel Storage, L.L.C.

P.O. Box C4010, La Crosse, WI 54602-4010
Phone 303-741-7009 Fax: 303-741-7806
John L. Donnell, P.E., Project Director

March 28, 2000

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555-0001

COMMITMENT RESOLUTION LETTER #28
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In accordance with our March 23, 2000 conference call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA questions and comments regarding geotechnical issues for the Private Fuel Storage Facility (PFSF). The NRC questions/comments are documented below followed by the PFS response.

NRC Questions and Comments

1. Due to the high void ratios of some of the in situ soils and their weakly cemented nature, there is the potential that these soils may be collapsible soils, which could settle dramatically due to wetting caused by the PMF flood or due to vibrations from the design earthquake. PFS should provide more detailed information demonstrating that the in situ soils are not collapsible due to wetting. This response should also include discussion of why the soils will not be wetted from the PMF flood.

PFS RESPONSE

The ***collapse potential*** of the high void ratio soils was determined in accordance with the requirements ASTM D5333 – 92, "*Standard Test Method for Measurement of Collapse Potential of Soils*." As indicated in Section 5.1 of this ASTM, "collapsible soils" are subject to "*... sudden and often large induced settlements when these soils are saturated* ...". The test method consists of performing consolidation tests, wherein the tests are initiated with the specimens at the natural water content and, at a predetermined vertical stress, inundating the specimens with distilled water to determine their proclivity to collapse upon wetting.

PFS reviewed the consolidation testing that has been performed to-date on samples obtained from the site. These data are summarized on the attached table. As indicated, PFS performed ten consolidation tests. Five of these were inundated with water after the primary consolidation had occurred for pressure increments that were less than the static load expected underneath the cask storage pads and just slightly greater than the static load expected underneath Canister Transfer Building. Four of the five consolidation specimens that were inundated were samples representative of the high void ratio soils. These specimens had void ratios that ranged from 1.95 to 2.51.

Following inundation, these specimens were kept inundated throughout the remainder of the consolidation tests. If susceptible to collapse, their collapse would have been manifested at some point during the performance of the consolidation tests. All of the inundated samples acquired degrees of saturation greater than 96%, which is in excess of the typical degree of saturation (~80% per Dudley, 1970) necessary to produce collapse in most collapsible soils. However, none of these specimens exhibited vertical displacements that would be interpreted as collapse in response to wetting.

NAVFAC DM-7.1 (1982) defines "collapse potential" as the additional strain induced by inundation; i.e.,

$$CP = \frac{\Delta H_c}{H_0}, \text{ expressed as a percentage.}$$

where ΔH_c is the change in height of the consolidation test specimen upon wetting and H_0 is the initial height of the specimen.

The inundation of the specimens tested by PFS typically resulted in less than ~0.1% additional vertical strain for sustained loadings of more than 800 minutes. This additional vertical strain is believed to be due to secondary compression and not soil collapse. However, even if this is considered to be collapse, the collapse potential equals only 0.1%. Figure 6 of Chapter 1 (Page 7.1-41) of NAVFAC DM-7.1 (1982), entitled "Typical Collapse Potential Results," indicates that the "Severity of Problem" due to potential for collapse of soils with collapse potential of 0 to 1% is described as "No problem". Thus, these soils are not collapsible soils.

These specimens did not collapse at any of the stress levels imposed during these tests, including those as high as 16 ksf, which is greatly in excess of the stresses (< 2 ksf) to be imposed due to the foundation loads. Comparison of the stress-strain plots of the specimens that were inundated with those that were not inundated, shows that they are nearly the same. If these soils had a tendency to collapse, this would not be the case. The inundated specimens would show increased vertical displacements if they collapsed. Therefore, based on the industry accepted method of determining the collapse potential of soils due to wetting, these soils are not "collapsible soils".

All of the inundated specimens were obtained from the upper silty clay/clayey silt layer shown in the foundation profiles, SAR Figures 2.6-5, Sheets 1 through 14 for the pad emplacement area, and Figures 2.6-21 through 2.6-23 for the Canister Transfer Building area. This is the stratigraphic unit at the site that exhibited the high void ratios and, hence, the low unit weights.

The attached plot of dry densities of the subsurface soils at the site illustrates that the upper silty clay/clayey silt layer has the lowest unit weights of the three clayey layers that exist beneath the site. Therefore, the soils in the upper layer are more likely to be collapsible soils than those in the underlying layers. As discussed above, the results of the consolidation tests performed on the upper layer indicate that they are not collapsible; therefore, the soils in the underlying layers are not collapsible, as well.

Further, these underlying soil layers are sufficiently removed from the surface (depths are >10 to 12 ft) of the site that it is extremely unlikely they could ever become wetted due to surface waters. The overlying soils are fine-grained silty clay/clayey silt, which have very low permeabilities. In addition, the upper layer of silty clay/clayey silt will be capped by a layer of engineered soil cement. As shown in the pad emplacement area foundation profiles (SAR Figure 2.6-5), the soil cement typically will extend 3 to 5 ft below most of the pads, making it approximately 5 to 7 ft thick. The permeability of the compacted soil cement is expected to be lower than that of the underlying silty clay/clayey silt. In addition, the site is pitched to the north and the site drainage is designed to direct rain falling on the site to the Detention Pond at the northern end of the PFSF. Therefore, surface water will flow off the site to the north, along the top of the soil-cement layer, and it will not wet the underlying soils.

PFS also reviewed the results of the Probable Maximum Flood (PMF) analyses (PFS Calculation 0599602-G(B)-17, Rev 1, "PFSF Flood Analysis with Proposed Access Road and Railroad," SWEC, 1999). In the discussion that follows, note that the elevations of the tops of the cask storage pads range from a low of Elevation 4463 ft at the northern end of the pad emplacement area, to a high of Elevation 4475 ft at the southern end. These are shown in Pad Emplacement Area Foundation Profiles 7-7' through 12-12', presented as Sheets 9 through 14 of SAR Figures 2.6-5. The locations of these profiles are presented on SAR Figure 2.6-19.

Figure 1 of this calculation (SWEC, 1999) identifies the locations of Drainage Basins A and B, which are also shown on SAR Figure 2.4-1. Figure 8 of this calculation presents a plan view of the PMF flood boundaries, and it indicates that the PMF boundaries for both Basins A and B do not reach the site. Figure 3, which presents the PMF water surface profile from Drainage Basin A, illustrates that although the flood waters pass over the top of the Access Road approximately 5,000 ft east of the site and floods the area to the northeast of the site, the surface of the flood is much lower in the vicinity of the PFSF.

Figure 8 illustrates that in the northeast corner of the pad emplacement area, the maximum flood water level is Elevation 4456.74 ft. The tops of the closest (and lowest) cask storage pads in this area are at Elevation 4463 ft. Thus, ***the elevation of the pads at***

this point is greater than 6 ft above the PMF elevation. This is also illustrated in Figure 5 of the calculation, which presents a cross-section view of the PMF water level near the northeast corner of the PFSF site. Figure 8 also illustrates that, at the entrance of the Access Road to the PFSF, which is east of the Canister Transfer Building, the maximum elevation of the PMF is only 4466.39 ft. The final grade in this area of the PFSF is Elevation 4475, which is also the elevation of the tops of the closest cask storage pads. Thus, ***the elevations of the closest pads are greater than 8 ft above the PMF*** elevation in this area of the PFSF. Therefore, the site will not be inundated by the maximum water levels due to the PMF occurring within Drainage Basin A.

Figures 6 and 7 of the calculation present similar information for the PMF occurring within Drainage Basin B, which impacts the railroad west of the PFSF. The boundary of the Basin B PMF is also shown in plan view on Figure 8. As indicated in these figures, the maximum elevation of the PMF near the PFSF site is 4478.22 ft, which is higher than the tops of the cask storage pads in the south and west corner of the pad emplacement area. However, this floodwater is precluded from reaching the pad emplacement area by the PMF Diversion Berm, which has a top elevation of 4480 ft. Where the PMF Diversion Berm ends near the middle of the western edge of the pad emplacement area (SAR Figure 2.6-2, Sheet 1), the PMF water level is only as high as Elevation 4464.83 ft. The tops of the cask storage pads east of that area are at Elevation 4469 (shown in Pad Emplacement Area Foundation Profile 7-7', Sheet 9 of SAR Figure 2.6-5), ***which is greater than 4 ft above the PMF elevation***. Therefore, the site will not be inundated by the maximum water levels due to the PMF occurring within Drainage Basin B.

This discussion of flooding due to the PMF illustrates that there is no opportunity for water due to the PMF to pond within the pad emplacement area. Therefore, even if the soils were collapsible, they would not be subject to collapse due to wetting caused by the PMF.

The soils at the site have a different depositional history than the collapsible soils that are present in other parts of Utah (Rollins and Williams, 1991, "Collapsible Soil Hazard Mapping for Cedar City, Utah"). As indicated in SAR Section 2.6.1.1, the unconsolidated deposits at the site are sediments laid down in and by Lake Bonneville. The collapsible soils referred to in Rollins and Williams (1991) are deposits that are formed as alluvial-fan and debris-flow sediments and in some wind-blown silts. These soils can be very susceptible to collapse upon wetting, and sometimes collapse from activities as seemingly benign as lawn watering. Note, Cedar City, Utah is located in the southwestern part of Utah, which is very far removed from the site.

In addition, there is no history or evidence of this phenomenon occurring in Skull Valley. As indicated in SAR Section 2.6.1.1, soils at the site are described in the County Soil Report (USDA, unpublished report), which includes descriptions of suitability of various soil types for construction of septic systems, dwellings, and roads; however, there is no mention of collapsible soils. It is reasonable to expect that if collapsible soils of the type found in Cedar City, Utah were present in the vicinity of the site, they would be mentioned in the County Soil Report. At the minimum, there would be cautionary statements regarding the design and installation of septic systems, which discharge water to subsurface soils, and which would be subject to damage if they were constructed within or above collapsible soils.

2. PFS needs to provide more detailed information demonstrating that the soils will not collapse due to vibrations caused by the design earthquake. In this response, PFS needs to address how the presence of negative pore pressures existing in the unsaturated fine-grained soils affect the results of the cyclic triaxial tests.

PFS RESPONSE

The ***collapse potential*** of the high void ratio soils due to shaking caused by the design earthquake was demonstrated to be nonexistent by the results of the cyclic triaxial tests that are presented in Appendix 2A of the SAR. These tests are discussed on Page 2 of Attachment 6 of Appendix 2A.

These soils will not be saturated during the life of the facility; therefore, in accordance with Section 21.7 of Lambe and Whitman, Soil Mechanics, John Wiley & Sons, New York, NY, 1969, "Partially Saturated Soils," which states:

> *"The best procedure to estimate the strength (of partially saturated soils) is to run tests that duplicate the field conditions as closely as possible: same degree of saturation, same total stress and, if possible, the same pressure in the liquid phase."*

These tests were performed on samples at their natural water content, using confining stresses that emulate conditions expected under the structures prior to the earthquake.

These unsaturated, fine-grained soils are expected to have some negative pore pressures due to the internal menisci of the pore water. Because of this, the vertical effective stresses between the soil grains will always be greater than the calculated vertical total stresses for the partially saturated fine-grained soils. The cyclic stress ratio used to determine the cyclic axial load to apply when performing the cyclic triaxial tests was determined as indicated on Page 18 of Attachment 6 of Appendix 2A of the SAR, conservatively ignoring the presence of negative pore pressures. The vertical effective stress, which appears in the denominator of the equation used to calculate the cyclic stress ratio, would be larger if the negative pore pressures were included. If the denominator was increased to include the negative pore pressures, the cyclic stress ratio for the laboratory samples would be lower. Therefore, the higher cyclic stress ratio used for the cyclic triaxial tests presented in Attachment 6 of Appendix 2A of the SAR actually overestimates the cyclic deviator stress to apply during the test to emulate the shear stresses to be imposed on these soils due to shaking caused by the design earthquake.

Further, the shear stresses due to the design earthquake were determined for the PFSF deterministic earthquake, which had a peak horizontal acceleration, a_H, of 0.67g. The design earthquake for the PFSF was subsequently changed to the PSHA 2,000-yr return period earthquake, for which $a_H = 0.528g$. Therefore, conservatively ignoring the negative pore pressures in these partially saturated fine-grained soils and estimating the shear stresses based on the PFSF deterministic earthquake, the cyclic axial load applied to these specimens far exceeded that which is expected due to the design earthquake. The strip chart plots, included in Attachment 6 of Appendix 2A of the SAR, showing the displacements measured during the cyclic triaxial tests demonstrate that even with this conservatively high cyclic axial load, the high void ratio soils showed no tendency to collapse. Therefore, these soils will not collapse due to shaking caused by the design earthquake.

References:

ASTM D5333, "Standard Test Method for Measurement of Collapse Potential of Soils," American Society for Testing and Materials, Philadelphia, PA, 1992.

Dudley, J. H., "Review of Collapsing Soils," Proceedings of the ASCE, Journal of the Soil Mechanics and Foundation Division, Vol 96, No. SM3, March 1970, pp 925-947.

Lambe, T. W., and Whitman, R. V., Soil Mechanics, John Wiley & Sons, New York, NY, 1969, p 316.

NAVFAC (1982), DM 7.1, "Soil Mechanics,", Dept of the Navy, Naval Facilities Eng'g, Command, Alexandria, VA.

Rollins, K. M., and Williams, T., "Collapsible Soil Hazard Mapping for Cedar City, Utah," Proceedings of the 1991 Annual Symposium on Engineering Geology & Geotechnical Engineering, No. 27, Pocatello, Idaho State University, 31 1 (1991).

SWEC (1999), PFS Calculation 0599602-G(B)-17, Rev 1, "PFSF Flood Analysis with Proposed Access Road and Railroad," SWEC, 1999.

PFS will include the information presented herein in the next update to the SAR. If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

J L Cooper
John L. Donnell fm
Project Director
Private Fuel Storage L.L.C.

Enclosure:

Copy to (with enclosure):
Mark Delligatti
John Parkyn
Jay Silberg
Sherwin Turk
Asadul Chowdhury
Murray Wade
Scott Northard
Denise Chancellor
Richard E. Condit
John Paul Kennedy
Joro Walker

Results of Consolidation Tests in Order of Decreasing Void Ratio

Boring	Sample	Average Depth (ft)	USC Code	INITIAL Water Content (%)	INITIAL Dry Density (pcf)	INITIAL Void Ratio	INITIAL Sat'n (%)	FINAL Water Content (%)	FINAL Dry Density (pcf)	FINAL Void Ratio	FINAL Sat'n (%)	Inundated?	Comment
CTB-N	**U-2D**	8.6	MH	63.0	48.4	2.511	68.2	60.6	64.0	1.655	99.5	@ 1 TSF	Inundated 41 minutes after application of vertical stress of 2 ksf.
C-1	**U-3D**	11.4	ML	46.7	51.7	2.285	55.6	62.4	64.1	1.649	103.0	@ 0.5 TSF	
CTB-S	**U-3C**	10.0	MH	72.2	51.9	2.269	86.6	54.4	67.4	1.519	97.4	@ 1 TSF	Inundated 34 minutes after application of vertical stress of 2 ksf.
C-1	**U-3C**	11.2	ML	38.9	55.8	2.041	51.8	51.9	68.4	1.484	95.2	No	Porous stones moist.
C-2	**U-2E**	11.7	ML	39.7	57.5	1.952	55.3	65.0	59.8	1.840	96.0	@ 0.5 TSF	Test stopped @ 2 TSF
CTB-4	**U-2E**	9.8	CH	48.9	63.2	1.687	78.8	42.1	75.8	1.240	92.3	No	
CTB-5	**U-12C**	23.5	MH	52.4	63.3	1.683	84.6	43.6	75.0	1.265	93.8	No	
C-1	**U-3B**	10.8	ML	30.3	64.7	1.625	50.7	28.7	73.4	1.315	59.3	No	Porous stones dry.
C-2	**U-2C**	10.9	ML	27.6	64.9	1.615	46.4	44.2	76.2	1.230	97.7	@ 0.5 TSF	
CTB-5	**U-14E**	27.3	CL	26.2	90.9	0.868	82.1	24.9	97.9	0.735	92.2	No	

STONE WEBSTER ENGINEERING CORPORATION

Private Fuel Storage, L.L.C.
PFSF, Skull Valley, UT

Dry Densities of Subsurface Soils at the Site

JO 05996.02
March 2000