RAS 5480 72-22-ISFSI - Applicant Exhibit 230 - Rec'd 6/18/02

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

This sequence extends to a depth of about 25 to 30 ft, where a continuous, nearly horizontal layer of dense, fine sand is encountered. This layer is the "Stansbury Transgressive", and it represents the oldest deposit of the Bonneville Cycle. The base of this unit occurs at a depth of about 45 to 50 ft and is believed to be an unconformity represented by the Promontory soil. This boundary is an apparent seismic velocity contrast that is recognizable on the recent seismic reflection profiles as a continuous, nearly horizontal layer, the Qp reflector (Geomatrix Consultants, Inc, 2001a).

2.6.1.11.4 Collapse Potential of High Void Ratio Soils

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. The following section demonstrates that these soils are not "collapsible soils". It also demonstrates that these soils will not be subject to wetting due to floodwaters associated with the Probable Maximum Flood (PMF), because, as indicated in Section 2.4.2.2, the tops of the cask storage pads are at least 4 ft above the nearest approach of the PMF to the PFSF pad emplacement area. The collapse potential due to vibrations from the design earthquake is demonstrated to be nonexistent based on the results of the cyclic triaxial tests, as described in Section 2.6.4.7.

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"

Template = Secy-028 230 Secy-02

NUCLEAR REGULATORY COMMISSION

Docket No. 72-22 Official Exh. No. PFS-230

In the matter of PFS, LLC

Staff ✓ IDENTIFIED ✓

Applicant ✓ RECEIVED ✓

Intervenor REJECTED

Cont'g Off'r

Contractor DATE 6/18/02

Other Witness Bartlett/Mitchell

Reporter G.Besa

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 performed ten consolidation tests. The results of these tests are summarized on Table 2.6-12, sorted in order of decreasing void ratio. As indicated, five of these were inundated with water in accordance with ASTM D5333–92 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, having 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.,

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

Template-Secy-028

Secy-02

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 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 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, Figure 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.

TABLE 2.6-12

RESULTS OF CONSOLIDATION TESTS IN ORDER OF DECREASING VOID RATIO

Boring	Sample	Average Depth (ft)	USC Code	INITIAL				FINAL				Inundated?	Comment
				Water Content (%)	Dry Density (pcf)	Void Ratio	Sat'n (%)	Water Content (%)	Dry Density (pcf)	Void Ratio	Sat'n (%)		
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	

Figure 2.6-31 present a plot of dry densities vs depth of the subsurface soils at the site and 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 silts, 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 (Figure 2.6-5, Sheets 1 through 14), the soil cement will nominally extend 2 ft below the bottoms of most of the pads (it will be a minimum of 1 ft thick and shall have a maximum thickness of 2 ft), making it approximately 5 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.

The results of the Probable Maximum Flood (PMF) analyses (SWEC, 1999d) demonstrate that the floodwater from the PMF will not inundate the pad emplacement area. As indicated in Section 2.4.2.2, 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 Section 2.6.1.1, the unconsolidated deposits at the site are sediments laid down in 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 Section 2.6.1.1, soils at the site are described in the County Soil Report (USDA, unpublished report). The purpose of such reports is to identify the locations of various soil types and to describe their suitability for construction of septic systems, dwellings, and roads. However, there is no mention of collapsible soils in the County Soil Report applicable for Skull Valley. 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.

2.6.1.11.5 Dynamic Strength of Cohesive Soils

It has been recognized in the past that the strength of cohesive soil increases as the rate of loading increases. For example, Casagrande and Shannon (1948) conducted soil dynamics investigations in 1948 with research efforts directed at finding the effects of rate of loading on soils common to the Panama Canal zone, i.e., clays, muck, shales, and dense dry sand. A "strain-rate" effect, defined as the ratio of maximum dynamic strength to the maximum static strength, was observed in all soils tested,