

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.64

CLIENT & PROJECT PRIVATE FUEL STORAGE, LLC - PFSF				PAGE 1 OF 59 + 6 pp of ATTACHMENTS		
CALCULATION TITLE STABILITY ANALYSES OF CANISTER TRANSFER BUILDING				QA CATEGORY (✓) <input checked="" type="checkbox"/> I NUCLEAR SAFETY RELATED <input type="checkbox"/> II <input type="checkbox"/> III <input type="checkbox"/> (other)		
CALCULATION IDENTIFICATION NUMBER						
JOB ORDER NO.	DISCIPLINE	CURRENT CALC NO	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.		
05996.02	G(B)	13				
APPROVALS - SIGNATURE & DATE				REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED <input checked="" type="checkbox"/>
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)				YES NO
Original Signed By: LPSingh / 12-9-98	Original Signed By: DLAloysius / 12-10-98	Original Signed By: DLAloysiu / 12-10-98	0	N/A		✓
Original Signed By: DLAloysius / 9-3-99 SYBoakye / 9-3-99 <i>See page 2-1 for ID of</i>	Original Signed By: SYBoakye / 9-3-99 DLAloysius / 9-3-99 <i>Prepared / Reviewed By</i>	Original Signed By: TYChang / 9-3-99 TYChang / 9-3-99	1	G(C)-13 Rev. 0		✓
Original Signed By: PJTrudeau / 1-21-00	Original Signed By: TYChang / 1-21-00	Original Signed By: TYChang / 1-21-00	2	1		✓
Original Signed By: PJTrudeau / 6-19-00	Original Signed By: TYChang / 6-19-00	Original Signed By: TYChang / 6-19-00	3	2		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	4	3		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	5	4		✓
PJTrudeau / 7-26-01 <i>Paul J. Trudeau</i>	TYChang / 7-26-01 <i>Thomas W. Chang</i>	TYChang / 7-26-01 <i>Thomas W. Chang</i>	6	5		✓
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NUCLEAR REGULATORY COMMISSION

Docket No. 7272 Official Ex. No. 96
In the matter of PPS
Staff _____ IDENTIFIED
Applicant _____ RECEIVED
Intervenor REJECTED _____
Cont'g Off'r _____ DATE 6/20/02
Contractor _____ Witness Booklet
Other _____
Reporter G. Bosa

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USNRC



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RULEMAKINGS AND
ADJUDICATIONS STAFF

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The undrained strength used in the bearing capacity analyses presented herein is a weighted average strength that is applicable for the soils in the upper layer. This value is determined using the value of undrained shear strength of 2.2 ksf noted above for the soils tested at depths of ~10 ft and the relative strength increase measured for the soils below depths of ~12 ft in the cone penetration tests that were performed within the Canister Transfer Building footprint. As indicated on SAR Figure 2.6-18, these included CPT-37 and CPT-38. Similar increases in undrained strength for the deeper lying soils were also noted in all of the other CPTs performed in the pad emplacement area.

Attachment B presents copies of the plots of s_u vs depth for CPT-37 and CPT-38, which are included in Appendix D of ConeTec(1999). These plots are annotated to identify the average undrained strength of the cohesive soils measured with respect to depth. As shown by the plot of s_u for CPT-37, the weakest zone exists between depths of ~5 ft and ~12 ft. The results for CPT-38 are similar, but the bottom of the weakest zone is at a depth of ~11 ft. The underlying soils are all much stronger. The average value of s_u of the cohesive soils for the depth range from ~18 ft to ~28 ft is ~2.20 tsf, compared to s_u ~1.34 tsf for the zone between ~5 ft and ~12 ft. Therefore, the undrained strength of the deeper soils in the upper layer was ~64% ($\Delta s_u = 100\% \times [(2.20 \text{ tsf} - 1.34 \text{ tsf}) / 1.34 \text{ tsf}]$) higher than the strength measured for the soils within the depth range of ~5 ft to ~12 ft. The relative strength increase was even greater than this in CPT-38.

Using 2.2 ksf, as measured in the UU triaxial tests performed on specimens obtained from depths of ~10 ft, as the undrained strength applicable for the weakest soils (i.e., those in the depth range of ~5 ft to ~12 ft), the average strength for the soils in the entire upper layer is calculated as shown in Figure 4. The resulting average value, weighted as a function of the depth, is s_u ~3.18 ksf. This value would be much higher if the results from CPT-38 were used; therefore, this is considered to be a reasonable lower-bound value of the average strength applicable for the soils in the upper layer that underlie the Canister Transfer Building.

Further evidence that this is a conservative value of s_u for the soils in the upper layer is presented in Figure 6. This plot of s_u vs confining pressure illustrates that this value is slightly less than the average value of s_u measured in the CU triaxial tests that were performed on specimens obtained from depths of ~10 ft at confining stresses of 2.1 ksf. As indicated in this figure, the confining stress of 2.1 ksf used to test these specimens is comparable to the vertical stress that will exist ~7 ft $[(2.1 \text{ ksf} - 1.46 \text{ ksf}) + 0.09 \text{ kcf}]$ below the Canister Transfer Building mat following completion of construction. Since these tests were performed on specimens of the weakest soils underlying the Canister Transfer Building mat (the deeper lying soils are stronger based on the SPT and the cone penetration test data), it is conservative to use the weighted average value of s_u of 3.18 ksf for the soils in the entire upper layer of the profile in the bearing capacity analyses.

Direct shear tests were performed on undisturbed specimens of the silty clay/clayey silt obtained from Borings CTB-6 and CTB-S, which were drilled in the locations shown in SAR Figure 2.6-18. These specimens were obtained from Elevation ~4469, approximately the elevation of the bottom of the perimeter key proposed at the base of Canister Transfer

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Building mat. Note, this key is being constructed around the perimeter of the mat to ensure that the full shear strength of the clayey soils is available to resist sliding of the structure due to loads from the design basis ground motion. These direct shear tests were performed at normal stresses that ranged from 0.25 ksf to 3.0 ksf. This range of normal stresses bounds the ranges of stresses expected for static and dynamic loadings from the design basis ground motion.

The results of these tests are presented in Attachments 7 and 8 of the Appendix 2A of the SAR and they are plotted in Figures 7 and 8. Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, sliding stability analyses included below of the Canister Transfer Building constructed directly on the silty clay are performed using the average shear strength measured in these direct shear tests for a normal stress equal to the vertical stress under the building following completion of construction, but prior to imposition of the dynamic loading due to the earthquake. As shown in Figures 7 and 8, this average shear strength is 1.7 ksf and the friction angle is set equal to 0°.

Effective-stress strength parameters are estimated to be $\phi = 30^\circ$ and $c = 0$ ksf, even though these soils may be somewhat cemented. This value of ϕ is based on the PI values for these soils, which ranged between 5% and 23% (SWEC, 2000a), and the relationship between ϕ and PI presented in Figure 18.1 of Terzaghi & Peck (1967).

Therefore, static bearing capacity analyses are performed using the following soil strengths:

Case IA Static using undrained strength parameters: $\phi = 0^\circ$ & $c = 3.18$ ksf.

Case IB Static using effective-stress strength parameters: $\phi = 30^\circ$ & $c = 0$.

and dynamic bearing capacity analyses are performed using $\phi = 0^\circ$ & $c = 3.18$ ksf.

Soil Cement Properties:

The unit weight of the soil cement is assumed to be 100 pcf in the analyses included herein and the unconfined compressive strength is 250 psi. (Initial results of the soil-cement testing indicate that 110 pcf is a reasonable lower-bound value for the total unit weight of the soil cement adjacent to the Canister Transfer Building foundation.) This strength is consistent with the soil-cement mix proposed for use within the frost zone adjacent to the cask storage pads and is based on the assumption that the strength will be at least this value to obtain a soil cement mix design that will satisfy the durability requirements of the ASTM wet/dry and freeze/thaw tests.

PFS is developing the soil-cement mix design using standard industry practice, in accordance with the criteria specified by the Portland Cement Association. This effort includes performing laboratory testing of soils obtained from the site. This on-going laboratory testing is being performed in accordance with the requirements of Engineering Services Scope of Work (ESSOW) for Laboratory Testing of Soil-Cement Mixes, ESSOW 05996.02-G010, Rev. 0. This program includes measuring gradations and Atterberg limits of samples of the near-surface soils obtained from the site. It includes testing of mixtures

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the state of stress existing under the Canister Transfer Building mat. Note, that the average post-peak strength reduction for normal stress of 1.5 ksf for the three direct shear tests is only 15.6% for these very high shear displacements in the direct shear tests. The maximum value of the average the post-peak strength reductions for normal stress of 1.5 ksf occurred for Sample U-3B&C in CTB-6, and it equaled 20.8%. If the results of this test were used to define the residual strength of these soils, the analyses would be performed at $c = 1.5$ ksf, the average of the post-peak strengths measured at the maximum shear displacements in these tests for normal stresses of 1 ksf and 2 ksf. This would result in higher factors of safety than are calculated and presented in Table 2.6-14, based on $c = 1.36$ ksf.

**CALCULATION OF AVERAGE POST-PEAK STRENGTH REDUCTION FOR NORMAL STRESS
APPLICABLE TO FINAL TRESSES UNDER THE CANISTER TRANSFER BUILDING**

Boring	Sample	Normal Stress = 1 ksf			Normal Stress = 2 ksf			Average Post-Peak Strength Reduction for Normal Stress = 1.5 ksf
		Peak Strength	Strength at Maximum Shear Displacement	Post-Peak Strength Reduction	Peak Strength	Strength at Maximum Shear Displacement	Post-Peak Strength Reduction	
		ksf	ksf	%	ksf	ksf	%	
C-2	U-1C	1.67	1.2	28.1	2.13	2.1	1.4	14.8
CTB-6	U-3B&C	1.57	1.1	29.9	2.15	1.9	11.6	20.8
CTB-S	U-1AA	1.42	1.1	22.5	1.58	1.7	-0.0	11.3

Average = 15.6

The results of the sliding stability analysis of the Canister Transfer Building for this case are presented in Table 2.6-14. In this table, the components of the driving and resisting forces are combined using the SRSS rule. All of these factors of safety are greater than 1.1, the minimum required value. These results indicate that the factors of safety are acceptable for all load combinations examined. The lowest factor of safety is 1.26, which applies for Cases IIIC and IVC, where 100% of the dynamic earthquake forces act in the N-S direction and 40% act in the other two directions. These results demonstrate that there is additional margin available to resist sliding of the building due to the earthquake loads, even when very conservative estimates of the residual shear strength of the clayey soils are used.