RAS 5551 72-22-ISFSI

STONE & WEBSTER, INC.

State Exhibit 95-Rec'd 6/20/02

CALCULATION SHEET

5010.64

CLIENT & PROJECT	PAGE 1 OF 115
PRIVATE FUEL STORAGE, LLC – PFSF	+ 22 pp of ATTACHMENTS
CALCULATION TITLE	QA CATEGORY (✓)
STABILITY ANALYSES OF CASK STORAGE PADS	☑ I NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ (other)

CALCULATION IDENTIFICATION NUMBER				
JOB ORDER NO.	DISCIPLINE	CURRENT CALC NO	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	G(B)	04		

PREPARER(S)/DATE(S)	REVIEWER(S)/DATES(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED ☑ YES	NO
Original Signed By: TESponseller / 2-18-97 PJTrudeau / 2-24-97	Original Signed By: PJTrudeau / 2-24-97 TESponseller / 2-24-97	Original Signed By: NTGeorges / 2-27-97	0		✓	
Original Signed By: TESponseller / 4-30-97 PJTrudeau / 4-30-97	Original Signed By: PJTrudeau / 4-30-97 TESponseller / 4-30-97	Original Signed By: AFBrown / 5-8-97	1	0		✓
Original Signed By: PJTrudeau / 6-20-97	Original Signed By: NTGeorges / 6-20-97	Original Signed By: AFBrown / 6-20-97	2	1		✓
Original Signed By: PJTrudeau / 6-27-97	Original Signed By: LPSingh / 7-1-97	Original Signed By: LPSingh / 7-1-97	3	2		✓
Original Signed By: DLAloysius / 9-3-99 SYBoakye / 9-3-99	Original Signed By: SYBoakye / 9-3-99 DLAloysius / 9-3-99	Original Signed By: TYChang / 9-3-99	4	3	✓	
Original Signed By: PJTrudeau / 1-26-00	Original Signed By: TYC for SYBoakye 1-26-00 Lliu / 1-26-00	Original Signed By: TYChang / 1-26-00	5	4		✓
Original Signed By: PJTrudeau / 6-16-00	Original Signed By: TYChang / 6-16-00	Original Signed By: TYChang / 6-16-00	6	5		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	7	6	✓	

DISTRIBUTION

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	FIRE FILE - Denver	☑			
Geotechnical	PJTrudeau – Stoughton/3	☑			

Template = SECY-028

State's Exhibit 95

SECY-02

NUCLEAR REGULATORY COMMISSION

Docket No. 72-22 Official Exh. No. 95

In the matter of PFS

Staff IDENTIFIED ✓

Applicant ✓ RECEIVED ✓

Intervenor REJECTED

Cont'g Off'r DATE 6/20/02

Contractor Witness Bartlett

Other

Reporter G Bégin

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5010.64

STONE & WEBSTER, INC.

CALCULATION SHEET

CLIENT & PROJECT	PAGE 2
PRIVATE FUEL STORAGE, LLC – PFSF	

CALCULATION TITLE	QA CATEGORY (✓)
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APPROVALS - SIGNATURE & DATE

PREPARER(S)/DATE(S)	REVIEWER(S)/DATES(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED ☑ YES	NO
Original Signed By: PJTrudeau / 5-31-01	Original Signed By: TYChang / 5-31-01	Original Signed By: TYChang / 5-31-01	8	7		✓
PJTrudeau / 7-26-01 Paul J. Trudeau	TYChang / 7-26-01 Thomas Y. Chang	TYChang / 7-26-01 Thomas Y. Chang	9	8		✓

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Geotechnical	PJTrudeau – Stoughton/3	☑			

Table 6 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C) summarizes the results of the triaxial tests that were performed within depths of ~10 ft. The undrained shear strengths measured in these tests are plotted vs confining pressure in Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C). This figure is annotated to indicate the vertical stresses existing prior to construction and following completion of construction.

The undrained shear strengths measured in the triaxial tests are used for the dynamic bearing capacity analyses because the soils are partially saturated and they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. As indicated in Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C), the undrained strength of the soils within ~10 ft of grade is assumed to be 2.2 ksf. This value is the lowest strength measured in the UU tests, which were performed at confining stresses of 1.3 ksf. This confining stress corresponds to the in situ vertical stress existing near the middle of the upper layer, prior to construction of these structures. It is much less than the final stresses that will exist under the cask storage pads and the Canister Transfer Building following completion of construction. Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C) illustrates that the undrained strength of these soils increase as the loadings of the structures are applied; therefore, 2.2 ksf is a very conservative value for use in the dynamic bearing capacity analyses of these structures.

Direct shear tests were performed on undisturbed specimens of the silty clay/clayey silt obtained at a depth of 5.7 ft to 6 ft in Boring C-2. These tests were performed at normal stresses that were essentially equal to the normal stresses expected:

1. under the fully loaded pads before the earthquake,
2. with all of the vertical forces due to the earthquake acting upward, and
3. with all of the vertical forces due to the earthquake acting downward.

The results of these tests are presented in Attachment 7 of the Appendix 2A of the SAR and they are plotted in Figure 7 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C). 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, in the sliding stability analyses of the cask storage pads, included below, the shear strength of the silty clay/clayey silt equals the shear strength measured in these direct shear tests for a normal stress equal to the vertical stress under the fully loaded cask storage pads prior to imposition of the dynamic loading due to the earthquake. As shown in Figure 7 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C), this shear strength is 2.1 ksf and the friction angle is set equal to 0°.

Effective-stress strength parameters are estimated to be $c = 0$ ksf, even though these soils may be somewhat cemented, and $\phi = 30°$. 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).

SLIDING STABILITY OF THE CASK STORAGE PADS

The factor of safety (FS) against sliding is defined as follows:

FS = resisting force ÷ driving force

For this analysis, ignoring passive resistance of the soil (soil cement) adjacent to the pad, the resisting, or tangential force (T), below the base of the pad is defined as follows:

$T = N \tan \phi + c\,B\,L$

where, N (normal force) = $\Sigma F_v = W_c + W_p + EQ_{vc} + EQ_{vp}$

$\phi = 0°$ (for Silty Clay/Clayey Silt)

c = 2.1 ksf, as indicated on p C-2.

B = 30 feet

L = 67 feet

DESIGN ISSUES RELATED TO SLIDING STABILITY OF THE CASK STORAGE PADS

Figure 3 presents a detail of the soil cement under and adjacent to the cask storage pads. Figure 8 presents an elevation view, looking east, that is annotated to facilitate discussion of potential sliding failure planes. The points referred to in the following discussion are shown on Figure 8.

1. Ignoring horizontal resistance to sliding due to passive pressures acting on the sides of the pad (i.e., Line AB or DC in Figure 8), the shear strength must be at least 1.60 ksf (11.10 psi) at the base of the cask storage pad (Line BC) to obtain the required minimum factor of safety against sliding of 1.1.
2. The static, undrained strength of the clayey soils exceeds 2.1 ksf (14.58 psi). This shear strength, acting only on the base of the pad, provides a factor of safety of 1.27 against sliding along the base (Line BC). This shear strength, therefore, is sufficient to resist sliding of the pads if the full strength can be engaged to resist sliding.
3. Ordinarily a foundation key would be used to ensure that the full strength of the soils beneath a foundation are engaged to resist sliding. However, the hypothetical cask tipover analysis imposes limitations on the thickness and stiffness of the concrete pad that preclude addition of a foundation key to ensure that the full strength of the underlying soils is engaged to resist sliding.
4. PFS will use a layer of soil cement beneath the pads (Area HITS) as an "engineered mechanism" to bond the pads to the underlying clayey soils.
5. The hypothetical cask tipover analysis imposes limitations on the stiffness of the materials underlying the pad. The thickness of the soil cement beneath the pads is limited to 2 ft and the static modulus of elasticity is limited to 75,000 psi.

6. The modulus of elasticity of the soil cement is directly related to its strength; therefore, its strength must be limited to values that will satisfy the modulus requirement. This criterion limits the unconfined compressive strength of the soil cement beneath the pads to 100 psi.

7. Therefore, the pads will be constructed on a layer of soil cement that is at least 1-ft thick, but no thicker than 2-ft, that extends over the entire pad emplacement area, as delineated by Area HITS.

8. The unconfined compressive strength of the soil cement beneath the pads is designed to provide sufficient shear strength to ensure that the bond between the concrete comprising the cask storage pad and the top of the soil cement (Line BC) and the bond between the soil cement and the underlying clayey soils (Line JK) will exceed the full, static, undrained strength of those soils. To ensure ample margin over the minimum shear strength required to obtain a factor of safety of 1.1, the unconfined compressive strength of the soil cement beneath the pads (Area HITS) will be at least 40 psi.

9. DeGroot (1976) indicates that this bond strength can be easily obtained between layers of soil cement, based on nearly 300 laboratory direct shear tests that he performed to determine the effect of numerous variables on the bond between layers of soil cement.

10. Soil cement also will be placed between the cask storage pads, above the base of the pads, in the areas labeled FGBM and NCQP. This soil cement is NOT required to resist sliding of the pads, because there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement (Line BC) and between that soil-cement layer and the underlying clayey soils (Line JK) that the factor of safety against sliding exceeds the minimum required value.

11. The pads are being surrounded with soil cement so that PFS can effectively use the eolian silt found at the site to provide an adequate subbase for support of the cask transporter, as well as to provide additional margin against any potential sliding.

12. The actual unconfined compressive strength and mix requirements for the soil cement around the cask storage pads will be based on the results of standard soil-cement laboratory tests.

13. The unconfined compressive strength of the soil cement adjacent to the pads needs to be at least 50 psi to provide an adequate subbase for support of the cask transporter, in lieu of placing and compacting structural fill, but it likely will be at least 250 psi to satisfy the durability requirements associated with environmental considerations (i.e., freeze/thaw and wet/dry cycles) within the frost zone (30 in. from the ground surface).

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The analysis presented on the following pages demonstrates that the static, undrained strength of the in situ clayey soils is sufficient to preclude sliding (FS = 1.27 vs minimum required value of 1.1), provided that the full strength of the clayey soils is engaged. The soil-cement layer beneath the pads provides an "engineered mechanism" to ensure that the full, static, undrained strength of the clayey soils is engaged in resisting sliding forces. It also demonstrates that the bond between this soil-cement layer and the base of the concrete pad will be stronger than the static, undrained strength of the in situ clayey soils and, thus, the interface between the in situ soils and the bottom of the soil-cement layer is the weakest link in the system. Since this "weakest link" has an adequate factor of safety against sliding, the overlying interface between the soil cement and the base of the pad will have a greater factor of safety against sliding. Therefore, the factor of safety against sliding of the overall cask storage pad design is at least 1.27.

SLIDING STABILITY AT INTERFACE BETWEEN IN SITU CLAYEY SOILS AND BOTTOM OF SOIL CEMENT BENEATH THE PADS

The factor of safety against sliding is calculated as follows:

$$FS_{\text{Soil Cement to Clayey Soil}} = \overset{T}{4{,}221\text{ K}} \div \underbrace{(\overset{F_{AE\ E\text{-}W\ 5'}}{181.5\text{K}} + \overset{EQhp}{643\text{ K}} + \overset{EQhc_{E\text{-}W}}{2{,}212\text{ K}} + \overset{EQhsc}{285.8\text{ K}})}_{(3{,}322.3\text{ K})} = \underline{\mathbf{1.27\ (=Min)}}$$

The factor of safety against sliding is higher than this if the lower-bound value of μ is used (= 0.2), because the driving forces due to the casks would be reduced.

Ignoring the passive resistance acting on the sides of the pad, the resistance to sliding is the same in both directions; therefore, for this analysis, the larger value of EQhc (i.e., acting in the E-W direction) was used. Even with these conservative assumptions, the factor of safety exceeds the minimum allowable value of 1.1; therefore the pads overlying 2 ft of soil cement are stable with respect to sliding for this load case, assuming the strength of the cement-treated soils underlying the pad is at least as high as the undrained strength of the underlying soils.

MINIMUM SHEAR STRENGTH REQUIRED AT THE BASE OF THE PADS TO PROVIDE A FACTOR OF SAFETY OF 1.1

The minimum shear strength required at the base of the pads to provide a factor of safety of 1.1 is calculated as follows:

$$FS = \overset{T}{T} \div \underbrace{(\overset{F_{AE\ E\text{-}W\ 3'}}{65.3\text{ K}} + \overset{EQhp}{643\text{ K}} + \overset{EQhc_{E\text{-}W}}{2{,}212\text{ K}})}_{(2{,}920.3\text{ K})} \geq 1.1$$

$$\rightarrow \quad T \geq 1.1 \times 2{,}920.3\text{ K} = 3{,}212.3\text{ K}$$

Dividing this by the area of the pad results in the minimum acceptable shear strength at the base of the pad:

$$\tau = \frac{3{,}212.3\text{ K}}{30\text{ ft} \times 67\text{ ft}} = 1.60\frac{\text{K}}{\text{ft}^2} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 \times \frac{1{,}000\text{ lbs}}{\text{K}} = \mathbf{11.10\ psi}$$

Soil cement

$T_{N\text{-}S}$ = 10 pads x 30 ft x 67 ft x 2.1 ksf + 9 zones between the pads x 30 ft x 5 ft x 1.4 ksf,

or $T_{N\text{-}S}$ = 42,210 K + 1,890 K = 44,100 K

Total driving force in N-S direction = 21,020 K + 8,355 + 81.3 K = 29,456 K, as calculated above.

The resulting FS against sliding in the N-S direction is calculated as:

$$FS_{\text{Pad to Clayey Soil N-S}} = \overset{T_{N\text{-}S}}{44{,}100\text{ K}} \div \overset{\text{Driving Force}_{N\text{-}S}}{29{,}456} = \underline{1.50}$$

Ignoring Passive Resistance at End of E-W Row of Pads

The resulting FS against sliding in the E-W direction will be even higher, because the soil cement zone between the pads is much wider (35 ft vs 5 ft) and longer (67 ft vs 30 ft) between the pads in the E-W direction than those in the N-S direction. The cask driving forces in the E-W direction are slightly higher than in the N-S direction, 10 pads x 2,212 K = 22,120 K vs 10 pads x 2,102 K = 21,020 K, resulting in an increased driving force of 22,120 K – 21,020 K = 1,100 K. The resistance to sliding in the E-W direction is increased much more than this, however. The increased resistance to sliding E-W = 35 ft x 67 ft x 1.4 ksf = 3,283 K / area between pads in the E-W row, compared to 5 ft x 30 ft x 1.4 ksf = 210 K / area between pads in the N-S column. Thus, the factor of safety against sliding of a row of pads in the E-W is much greater than that shown above for sliding of a column of pads in the N-S direction.

Including Passive Resistance at End of N-S Column of Pads

In this analysis, the resistance to sliding in the N-S direction includes the full passive resistance at the far end of the column of pads, which acts on the 2'-4" height of soil cement along the 30-ft width of the pad in the E-W direction.

Assuming the soil cement adjacent to the pad is constructed such that its unconfined compressive strength is 250 psi, its full passive resistance acting on the 2'-4" thickness of soil cement adjacent to the pad will provide a force resisting sliding in the N-S direction of:

$$T_{\text{SC Adjacent to Pad@N\&S}} = 250\,\frac{\text{lbs}}{\text{in.}^2}\ \text{x}\left(\frac{12\text{ in.}}{\text{ft}}\right)^2\ \text{x}\ \frac{\text{K}}{1{,}000\text{ lbs}}\ \text{x 2.33 ft x 30 ft} = 2{,}516\text{ K}$$

The total resistance based on the peak shear strength of the underlying clayey soil is

Soil cement

$T_{N\text{-}S}$ = 10 pads x 30 ft x 67 ft x 2.1 ksf + 9 zones between the pads x 30 ft x 5 ft x 1.4 ksf, or

$T_{N\text{-}S}$ = 42,210 K + 1,890 K = 44,100 K

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DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case II

Based on Inertial Forces Combined:
100 % N-S, 0 % Vert, 100 % E-W

Soil Properties:

$c =$ 2,200 Cohesion (psf)

$\phi =$ 0.0 Friction Angle (degrees)

$\gamma =$ 80 Unit weight of soil (pcf)

$\gamma_{surch} =$ 100 Unit weight of surcharge (pcf)

Footing Dimensions:

$B = 30.0$ Width - ft (E-W)

$L = 67.0$ Length - ft (N-S)

Foundation Properties:

$B' =$ 15.6 Effective Ftg Width - ft (E-W) $L' = 52.6$ Length - ft (N-S)

$D_f =$ 3.0 Depth of Footing (ft)

0.711 $g = a_H$

0.695 $g = a_V$

$FS =$ 1.1 Factor of Safety required for $q_{allowable}$

$F_{V\,Static} =$ 3,757 k & $EQ_V =$ 0 k → 3,757 k for F_V

$EQ_{H\,E\text{-}W} =$ 2,671 k & $EQ_{H\,N\text{-}S} =$ 2,671 k → 3,777 k for F_H

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi\tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.06	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.88	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2\,D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.08	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.79 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.39	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross $q_{ult} =$	5,338	psf =	5,038	+	300	+	0

$q_{all} =$ 4,850 psf $= q_{ult}/FS$

$q_{actual} =$ 4,565 psf $= (F_{v\,Static} + EQ_v)/(B' \times L')$

$FS_{actual} =$ 1.17 $= q_{ult}/q_{actual}$ > 1.1 Hence OK