RAS 5575 72-22-ISFSI STONE & WEBSTER, INC. State Exhibit 116 - Rec'd 5/8/02

5010.64

# CALCULATION SHEET

| 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<br>☐ II<br>☐ III ☐ (other) |

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

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

## DISTRIBUTION

| GROUP | NAME & LOCATION | COPY SENT (✓) | GROUP | NAME & LOCATION | COPY SENT (✓) |
|---|---|---|---|---|---|
| RECORDS MGT. FILES (OR FIRE FILE IF NONE) | JOB BOOK R4.2G | ORIG | | | |
| | FIRE FILE - Denver | ☑ | | | |
| Geotechnical | PJTrudeau – Stoughton/3 | ☑ | | | |

Template = SECY-028

State's Exhibit 116

SECY-02

DOCKETED
USNRC

2003 JAN 29 PM 3: 13

OFFICE OF THE SECRETARY
RULEMAKINGS AND
ADJUDICATIONS STAFF

NUCLEAR REGULATORY COMMISSION

Docket No. _____ Official Exh. No. 116

In the matter of PFS

Staff _____ IDENTIFIED ✓

Applicant _____ RECEIVED ✓

Intervenor ✓ REJECTED _____

Other _____ WITHDRAWN _____

DATE 5-8-02 Witness _____

Clerk pmp

This is greater than the criterion of 1.1; therefore, the cask storage pads have an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion.

## 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) = $\sum F_v = W_c + W_p + EQ_{vc} + EQ_{vp}$

$\phi = 0°$ (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.

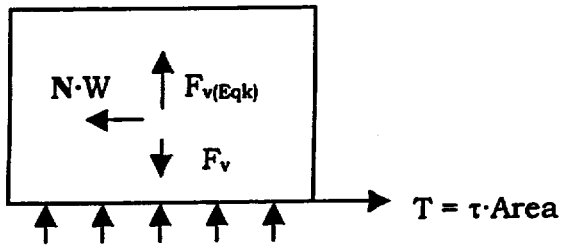
## EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

Adequate factors of safety against sliding due to maximum forces from the design basis ground motion have been obtained for the storage pads founded directly on the silty clay/clayey silt layer, conservatively ignoring the presence of the soil cement that will surround the pads. The shearing resistance is provided by the undrained shear strength of the silty clay/clayey silt layer, which is not affected by upward earthquake loads. As shown in SAR Figures 2.6-5, Pad Emplacement Area – Foundation Profiles, a layer, composed in part of sandy silt, underlies the clayey layer at a depth of about 10 ft below the cask storage pads. Sandy silts oftentimes are cohesionless; therefore, to be conservative, this portion of the sliding stability analysis assumes that the soils in this layer are cohesionless, ignoring the effects of cementation that were observed on many of the split-spoon and thin-walled tube samples obtained in the drilling programs.

The shearing resistance of cohesionless soils is directly related to the normal stress. Earthquake motions resulting in upward forces reduce the normal stress and, consequently, the shearing resistance, for purely cohesionless (frictional) soils. Factors of safety against sliding in such soils are low if the maximum components of the design basis ground motion are combined. The effects of such motions are evaluated by estimating the displacements the structure will undergo when the factor of safety against sliding is less than 1 to demonstrate that the displacements are sufficiently small that, should they occur, they will not adversely impact the performance of the pads.

The method proposed by Newmark (1965) is used to estimate the displacement of the pads, assuming they are founded directly on a layer of cohesionless soils. This simplification produces an upper-bound estimate of the displacement that the pads might see if a cohesionless layer was continuous beneath the pads. For motion to occur on a slip surface along the top of a cohesionless layer at a depth of 10 ft below the pads, the slip surface would have to pass through the overlying clayey layer, which, as shown above, is strong enough to resist sliding due to the earthquake forces. In this analysis, a friction angle of 30° is used to define the strength of the soils to conservatively model a loose cohesionless layer. The soils in the layer in question have a much higher friction angle, generally greater than 35°, as indicated in the plots of "Phi" interpreted from the cone penetration testing, which are presented in Appendix D of ConeTec (1999).

### *ESTIMATION OF HORIZONTAL DISPLACEMENT USING NEWMARK'S METHOD*



Newmark (1965) defines "N·W" as the steady force applied at the center of gravity of the sliding mass in the direction which the force can have its lowest value to just overcome the stabilizing forces and keep the mass moving. Note, Newmark defines "N" as the "Maximum Resistance Coefficient," and it is an acceleration coefficient in this case, not the normal force.

For a block sliding on a horizontal surface, $N \cdot W = T$,

where T is the shearing resistance of the block on the sliding surface.

Shearing resistance, $T = \tau \cdot \text{Area}$

where $\tau = \sigma_n \tan \phi$

$\sigma_n =$ Normal Stress

$\phi =$ Friction angle of cohesionless layer

$\sigma_n =$ Net Vertical Force/Area

$= (F_v - F_{v\,Eqk})/\text{Area}$

$T = (F_v - F_{v\,Eqk}) \tan \phi$

$N\,W = T$

$\Rightarrow \quad N = [(F_v - F_{v\,Eqk}) \tan \phi] / W$

The maximum relative displacement of the pad relative to the ground, $u_m$, is calculated as

$$u_m = [V^2 (1 - N/A)] / (2gN)$$

The above expression for the relative displacement is an upper bound for all of the data points for N/A less than 0.15 and greater than 0.5, as shown in Figure 5 , which is a copy of Figure 41 of Newmark (1965). Within the range of 0.5 to 0.15, the following expression gives an upper bound of the maximum relative displacement for all data.

$$u_m = V^2 / (2gN)$$

## MAXIMUM GROUND MOTIONS

The maximum ground accelerations used to estimate displacements of the cask storage pads were those due to the PSHA 2,000-yr return period earthquake; i.e., $a_H = 0.711g$ and $a_V = 0.695g$. The maximum horizontal ground velocities required as input in Newmark's method of analysis of displacements due to earthquakes were estimated for the cask storage pads assuming that the ratio of the maximum ground velocity to the maximum ground acceleration equaled 48 (i.e., 48 in./sec per g). Thus, the estimated maximum velocities applicable for the Newmark's analysis of displacements of the cask storage pads = 0.711 x 48 = 34.1 in./sec. Since the peak ground accelerations are the same in both horizontal directions, the velocities are the same as well.

## LOAD CASES

The resistance to sliding on cohesionless materials is lowest when the dynamic forces due to the design basis ground motion act in the upward direction, which reduces the normal forces and, hence, the shearing resistance, at the base of the foundations. Thus, the following analyses are performed for Load Cases IIIA, IIIB, and IIIC, in which the pads are unloaded due to uplift from the earthquake forces.

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.

## GROUND MOTIONS FOR ANALYSIS

| Load Case | North-South | | Vertical | East-West | |
|---|---|---|---|---|---|
| | Accel g | Velocity in./sec | Accel g | Accel g | Velocity in./sec |
| IIIA | 0.284g | 13.7 | 0.695g | 0.284g | 13.7 |
| IIIB | 0.284g | 13.7 | 0.278g | 0.711g | 34.1 |
| IIIC | 0.711g | 34.1 | 0.278g | 0.284g | 13.7 |

***Load Case IIIA: 40% N-S direction, -100% Vertical direction, 40% E-W direction.***

Static Vertical Force, $F_v$ = W = Weight of casks and pad = 2,852 K + 904.5 K = 3,757 K

Earthquake Vertical Force, $F_{v\,Eqk} = a_v \times W/g$ = 0.695g x 3,757 K/g = 2,611 K

$\phi$ = 30°

For Case IIIA, 100% of vertical earthquake force is applied upward and, thus, must be subtracted to obtain the normal force; thus, Newmark's maximum resistance coefficient is

$$N = [(\underbrace{3{,}757}_{F_v} - \underbrace{2{,}611}_{F_{v\,Eqk}}) \underbrace{\tan 30°}_{\phi}] \,/\, \underbrace{3{,}757}_{W} = 0.176$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\overset{40\%\ N\text{-}S}{0.284^2} + \overset{40\%\ E\text{-}W}{0.284^2})} = 0.402g$

Resultant velocity in horizontal direction, $V = \sqrt{(\overset{40\%\ N\text{-}S}{13.7^2} + \overset{40\%\ E\text{-}W}{13.7^2})} = 19.4$ in./sec

$$\Rightarrow \quad N / A = 0.176 / 0.402 = 0.438$$

The maximum displacement of the pad relative to the ground, $u_m$, calculated based on Newmark (1965) is

$$u_m = [V^2 (1 - N/A)] / (2gN)$$

where g is in units of inches/sec$^2$.

$$\Rightarrow \quad u_m = \left( \frac{(19.4 \text{ in./sec})^2 \cdot (1 - 0.438)}{2 \cdot 386.4 \text{ in./sec}^2 \cdot 0.176} \right) = 1.56"$$

The above expression for the relative displacement is an upper bound for all the data points for N /A less than 0.15 and greater than 0.5, as shown in Figure 5. For N/A values between 0.15 and 0.5 the data in Figure 5 is bounded by the expression

$$u_m = [V^2] / (2gN)$$

$$\Rightarrow \quad u_m = \left( \frac{(19.4 \text{ in./sec})^2}{2 \cdot 386.4 \text{ in./sec}^2 \cdot 0.176} \right) = 2.77"$$

In this case, N /A is = 0.438; therefore, use the average of the maximum displacements; i.e., 0.5 (1.56 + 2.77) = 2.2" . Thus the maximum displacement is ~2.2 inches.

***Load Case IIIB: 40% N-S direction, -40% Vertical direction, 100% E-W direction.***

Static Vertical Force, $F_v = W = 3{,}757$ K

Earthquake Vertical Force, $F_{v(Eqk)} = 2{,}611 \text{ K} \times 0.40 = 1{,}044$ K

$\phi = 30°$

$$N = [(\underbrace{3{,}757}_{F_v} - \underbrace{1{,}044}_{F_{v\,Eqk}}) \underbrace{\tan 30°}_{\phi}] / \underbrace{3{,}757}_{W} = 0.417$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\overset{40\%\ N\text{-}S}{0.284^2} + \overset{100\%\ E\text{-}W}{0.711^2})}\ g = 0.766g$

Resultant velocity in horizontal direction, $V = \sqrt{(\overset{40\%\ N\text{-}S}{13.7^2} + \overset{100\%\ E\text{-}W}{34.1^2})} =$ 36.7 in./sec

$$\Rightarrow \quad N / A = 0.417 / 0.766 = 0.544$$

The maximum displacement of the pad relative to the ground, $u_m$, calculated based on Newmark (1965) is

$$u_m = [V^2 (1 - N/A)] / (2g\,N)$$

$$\Rightarrow \quad u_m = \left(\frac{(36.7\,\text{in./sec})^2 \cdot (1 - 0.544)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.417}\right) = 1.91''$$

The above expression for the relative displacement is an upper bound for all the data points for N /A less than 0.15 and greater than 0.5, as shown in Figure 5. In this case, N /A is > 0.5; therefore, this equation is applicable for calculating the maximum relative displacement. Thus the maximum displacement is ~1.9 inches.

***Load Case IIIC: 100% N-S direction, -40% Vertical direction, 40% E-W direction.***

Since the horizontal accelerations and velocities are the same in the orthogonal directions, the result for Case IIIC is the same as those for Case IIIB.

**SUMMARY OF HORIZONTAL DISPLACEMENTS CALCULATED BASED ON NEWMARK'S METHOD FOR ASSUMPTION THAT CASK STORAGE PADS ARE FOUNDED DIRECTLY ON COHESIONLESS SOILS WITH $\phi$ = 30° AND NO SOIL CEMENT**

| LOAD COMBINATION | | | | DISPLACEMENT |
|---|---|---|---|---|
| **Case IIIA** | 40% N-S | -100% Vert | 40% E-W | 2.2 inches |
| **Case IIIB** | 40% N-S | -40% Vert | 100% E-W | 1.9 inches |
| **Case IIIC** | 100% N-S | -40% Vert | 40% E-W | 1.9 inches |

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with $\phi$ = 30°, the estimated relative displacement of the pads due to the design basis ground motion based on Newmark's method of estimating displacements of embankments and dams due to earthquakes 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 that is labeled "Clayey Silt/Silt & Some Sandy Silt" in the foundation profiles in the pad emplacement area (SAR Figures 2.6-5, Sheets 1 through 14), 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 (ConeTec, 1999) indicate that these soils have $\phi$ values that generally exceed 35 to 40°, as shown in Appendices D & F of ConeTec (1999). These high friction angles likely are the manifestation of 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.