Private Fuel Storage, L.L.C.

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July 27, 2001

CALCULATION PACKAGE - JULY 20, 2001 SUBMITTAL
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: PFS letter, Parkyn to U.S. NRC, dated July 20, 2001.

The above referenced letter transmitted additional clarifying information for the Private Fuel Storage Facility (PFSF) to the NRC.

Attached are the affected calculations which have been updated to incorporate this same information as appropriate.

If you have any questions regarding this submittal, please contact me at 303-741-7009.

Sincerely,

John L. Donnell

John L. Donnell
Project Director
Private Fuel Storage L.L.C.

Enclosure

NMSS01 Public

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STONE & WEBSTER, INC.

CALCULATION SHEET

5010.64

CLIENT & PROJECT	
PRIVATE FUEL STORAGE, LLC – PFSF	PAGE 1 OF 115 + 22 pp of ATTACHMENTS
CALCULATION TITLE **STABILITY ANALYSES OF CASK STORAGE PADS**	QA CATEGORY (✓) ☑ 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**		

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: 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	✓	

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STONE & WEBSTER, INC.

CALCULATION SHEET

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

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		

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: 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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RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1

Revision 1 was prepared to incorporate the following:

- Revised cask weights and dimensions
- Revised earthquake accelerations
- Determine q_{all} as a function of the coefficient of friction between casks and pad.

REVISION 2

To add determination of dynamic bearing capacity of the pad for the loads and loading cases being analyzed by the pad designer. These include the 2-cask, 4-cask, and 8-cask cases. See Attachment A for background information, as well as bearing pressures for the 2-cask loading.

REVISION 3

The bearing pressures and the horizontal forces due to the design earthquake for the 2-cask case that are described in Attachment A are superseded by those included in Attachment B. Revision 3 also adds the calculation of the dynamic bearing capacity of the pad for the 4-cask and 8-cask cases and revises the cask weight to 356.5 K, which is based on Holtec HI-Storm Overpack with loaded MPC-32 (heaviest assembly weight shown on Table 3.2.1 of HI-Storm TSAR, Report HI-951312 Rev. 1 – p. C3, Calculation 05996.01-G(B)-05, Rev 0).

REVISION 4

Updated section on seismic sliding resistance of pads (pp 11-14F) using revised ground accelerations associated with the 2,000-yr return period design basis ground motion (horizontal = 0.528 g; vertical = 0.533 g) and revised soil parameters (c = 1,220 psf; ϕ = 24.9°, based on direct shear tests that are included in Attachments 7 and 8 of Appendix 2A of the SAR.). The horizontal driving forces used in this analysis (EQhc and EQhp) are based on the higher ground accelerations associated with the deterministic design basis ground motion (0.67g horizontal and 0.69g vertical). These forces were not revised for the lower ground accelerations associated with the 2,000-yr return period design basis ground motion (0.528g horizontal and 0.533g vertical) and, thus, this calculation will require confirmation at a later date.

Added a section on sliding resistance along a deeper slip plane (i.e., on cohesionless soils) beneath the pads.

Updated section on dynamic bearing capacity of pad for 8-cask case (pp 38-46). Inserted pp 46A and 46B. This case was examined because it previously yielded the lowest q_{all}

among the three loading cases (i.e., 2-cask, 4-cask, and 8-cask). The updated section shows a calculation of q_{all} based on revised soil parameters (c and ϕ). Note: this analysis will require confirmation and may be updated using revised vertical soil bearing pressures and horizontal shear forces, based on the lower ground accelerations associated with the 2,000-yr return period design basis ground motion (0.528g horizontal, and 0.533g vertical).

Modified/updated conclusions.

NOTE: SYBoakye prepared/DLAloysius reviewed pp 14 through 14F.

Remaining pages prepared by DLAloysius and reviewed by SYBoakye.

REVISION 5

Major re-write of the calculation.

1. Renumbered pages and figures to make the calculation easier to follow.
2. Incorporated dynamic loads due to revised design basis ground motion (PSHA 2,000-yr return period earthquake), as determined in CEC Calculation 05996.02-G(PO17)-2, Rev 0, and removed "Requires Confirmation".
3. Added overturning analysis.
4. Added analysis of sliding stability of cask storage pads founded on and within soil cement.
5. Revised dynamic bearing capacity analyses to utilize only total-stress strength parameters because these partially saturated soils will not have time to drain fully during the rapid cycling associated with the design basis ground motion. See Calculation 05996.02-G(B)-05-1 (SWEC, 2000a) for additional details.
6. Added reference to foundation profiles through pad emplacement area presented in SAR Figures 2.6-5, Sheets 1 through 14.
7. Changed "Load Combinations" to "Load Cases" and defined these cases to be consistent throughout the various stability analyses included herein. These are the same cases as are used in the stability analyses of the Canister Transfer Building, Calculation 05996.02-G(B)-13-2 (SWEC, 2000b).
8. Revised conclusions to reflect results of these changes.

REVISION 6

1. Added "References" section.
2. Revised shear strength used in the sliding stability analyses of the soil cement/silty clay interface to be the strength measured in the direct shear tests performed on samples obtained from depths of ~5.8 ft in the pad emplacement area. The shear strength equaled that measured for stresses corresponding to the vertical stresses at the bottom of the fully loaded cask storage pads.
3. Removed static and dynamic bearing capacity analyses based on total-stress strengths and added dynamic bearing capacity analyses based on c_u = 2.2 ksf..

Revised method of calculating the inclination factor in the bearing capacity analyses to that presented by Vesic in Chapter 3 of Winterkorn and Fang (1975). Vesic's method expands upon the theory developed by Hansen for plane strain analyses of footings with inclined loads. Vesic's method permits a more rigorous analysis of inclined loads acting in two directions on rectangular footings, which more closely represents the conditions applicable for the cask storage pads.

REVISION 7

1. Updated stability analyses to reflect revised design basis ground motions (a_H = 0.711g & a_V = 0.695g, per Table 1 of Geomatrix, 2001).
2. Resisting moment in overturning stability analysis calculated based on resultant of static and dynamic vertical forces.
3. Added analysis of sliding of an entire column of pads supported on at least 1' of soil cement, using an adhesion factor of 0.5 for the interface between the soil cement and the underlying silty clay layer.
4. Added discussion of strength limitations of the soil cement under the cask storage pads to comply with the maximum modulus of elasticity requirements of the materials supporting the pad in the hypothetical cask tipover analysis.
5. Changed pad length to 67 ft and pad embedment to 3 ft, in accordance with design change identified in Figure 4.2-7, "Cask Storage Pads," of SAR Revision 21.
6. Added definition of "m" used in the inclination factors for calculating allowable bearing capacity.
7. Updated references to supporting calculations.
8. Updated discussions and conclusions to incorporate revised results.

REVISION 8

1. Revised analyses of the stability of the storage pads to include a clear identification of the potential failure modes and failure surfaces and the material strengths required to satisfy the regulatory requirement, considering the critical failure modes and failure surfaces.
2. Added assessment of the edge effects of the last pad in the column of pads on the stability of the storage pads under the new seismic loads.
3. Horizontal cask earthquake forces in the dynamic bearing capacity calculations were changed to limit the resultant of the two horizontal components to the coefficient of friction between the cask and the top of the pad x the effective weight of the casks.
4. Reduced shear strength of clayey soils beneath the pads to 95% of peak shear strength measured in direct shear tests in analyses that included both shear resistance along base of sliding mass and passive resistance. This 5% reduction of peak strength to residual strengths is the maximum reduction measured in the three direct shear tests that were performed on these clayey soils for specimens confined at 2 ksf, which corresponds to the approximate final effective stress at the base of the pads.

REVISION 9

1. Revised unit weights of soil cement to reflect measured values obtained from ongoing laboratory testing program. Unit weight of soil cement adjacent to the pads exceeds 110 pcf and the cement-treated soil beneath the pads exceeds 100 pcf.
2. Added clarification of approximations used in calculation of K_{AE} and updated calculation of K_{AE} to remove excess conservatism inherent in the previous use of approximations "$\sin(\phi - \theta) \approx 0$" and "$\cos(\phi - \theta) \approx 1$".
3. Added inertial forces due to 2-ft thick layer of soil cement beneath pad to sliding stability analysis.
4. Added analysis of hypothetical case where resistance to sliding is comprised of frictional resistance along base of pads and soil cement + passive resistance. This analysis demonstrates that the factor of safety against sliding is less than 1.1. Also added analysis to estimate the maximum pad displacement for these very conservative assumptions. This analysis shows that the resulting maximum horizontal displacements, if they were to occur due to the earthquake, would be of no safety consequence to the pads or the casks.
5. Added Attachment E, plot of Total Stress Mohr's Circles from triaxial tests performed on samples from Boring B-1.

OBJECTIVE OF CALCULATION

Evaluate the static & seismic stability of the cask storage pad foundations at the proposed site. The failure modes investigated include overturning stability, sliding stability, and bearing capacity for static loads & for dynamic loads due to the design basis ground motion (PSHA 2,000-yr return period earthquake with peak horizontal ground acceleration of 0.711g).

Other potential failure modes are addressed elsewhere. Evaluation of static settlements are addressed in Calculation 05996.02-G(B)-3-3, which is supplemented by Calculation 05996.02-G(B)-21-0. Dynamic settlements are addressed in Calculation 05996.02-G(B)-11-3. The soils underlying the site are not susceptible to liquefaction, as documented in Calculation 05996.01-G(B)-6-1.

Evaluation of floatation of these pads is not required because they will never be submerged, since groundwater is approximately 125 ft below the ground surface at the site. In addition, as indicated in SAR Section 2.4.8, Flooding Protection Requirements,

> *"All Structures, Systems, and Components (SSCs) classified as being Important to Safety are protected from flooding by diversion berms to deflect potential flows generated by PMF from both the east mountain range (Basin A) and the west mountain range (Basin B) watersheds."*

The design of the concrete pad, to ensure that it will not suffer bending or shear failures due to static and dynamic loads, is addressed in Calculation 05996.02-G(PO17)-2-3 (CEC, 2001).

ASSUMPTIONS/DATA

The arrangement of the cask storage pads is shown on SAR Figure 1.2-1. The spacing of the pads is such that each N-S column of pads may be treated as one long strip footing with B/L ~ 0 & B=30 ft for the bearing capacity analyses.

The E-W spacing of the pads is great enough that adjacent pads will not significantly impact the bearing capacity of one another, as shown on Figure 1, "Foundation Plan & Profile."

The generalized soil profile, presented in Figure 1, indicates the soil profile consists of ~30 ft of silty clay/clayey silt with some sandy silt (Layer 1), overlying ~30 ft of very dense fine sand (Layer 2), overlying extremely dense silt (N ≥100 blows/ft, Layer 3). SAR Figures 2.6-5 (Sheets 1 through 14) present foundation profiles showing the relationship of the cask storage pads with respect to the underlying soils. These profiles, located as shown in SAR Figure 2.6-19, provide more detailed stratigraphic information, especially within the upper ~30-ft thick layer at the site.

Figure 1 also illustrates the coordinate system used in these analyses. Note, the X-direction is N-S, the Y-direction is vertical, and the Z-direction is E-W. This is the same

coordinate system that is used in the stability analyses of the Canister Transfer Building (Calculation 05996.02-G(B)-13-2, SWEC, 2000b).

The bearing capacity analyses assume that Layer 1, which consists of silty clay/clayey silt with some sandy silt, is of infinite thickness and has strength properties based on those measured at depths of ~10 ft for the clayey soils within the upper layer. These assumptions simplify the analyses and they are very conservative. With respect to bearing capacity, the strength of the sandy silt in the upper layer is greater than that of the clayey soils, based on the increases in Standard Penetration Test (SPT) blow counts (N-values) and the increased tip resistance (see SAR Figures 2.6-5) in the cone penetration testing (ConeTec, 1999) noted in these soils. The underlying soils are even stronger, based on their SPT N-values, which generally exceed 100 blows/ft.

Based on probabilistic seismic hazard analysis, the peak acceleration levels of 0.711g for horizontal ground motion and 0.695g for the vertical ground motion were determined as the design bases of the PFSF for a 2,000-yr return period earthquake (Geomatrix Consultants, Inc, 2001).

Geotechnical Properties

Based on laboratory test results presented in Tables 2, 3, and 4 of Calculation 05996.02-G(B)-05-2 (SWEC, 2000a),

γ_{moist} = 80 pcf is a conservative lower-bound value of the unit weight for the soils underlying the pad emplacement area.

The bearing capacity of the structures are dependant primarily on the strength of the soils in the upper ~25 to ~30-ft layer at the site. All of the borings drilled at the site indicate that the soils underlying this upper layer are very dense fine sands overlying silts with standard penetration test blow counts that exceed 100 blows/ft. The results of the cone penetration testing, presented in ConeTec(1999) and plotted in SAR Figure 2.6-5, Sheets 1 to 14, illustrate that the strength of the soils in the upper layer are much greater at depths below ~10 ft than in the range of ~5 ft to ~10 ft, where most of the triaxial tests were performed.

In practice, the average shear strength along the anticipated slip surface of the failure mode should be used in the bearing capacity analysis. This slip surface is normally confined to within a depth below the footing equal to the minimum width of the footing. In this case, the effective width of the footing is decreased because of the large eccentricity of the load on the pads due to the seismic loading. As indicated in Table 2.6-7, the minimum effective width occurs for Load Cases II and IIIB, where B' ~15 ft. Figure 7 illustrates that the anticipated slip surface of the bearing capacity failure would be limited to the soils within the upper half of the upper layer. Therefore, in the bearing capacity analyses presented herein, the undrained strength measured in the UU triaxial tests was not increased to reflect the increase in strength observed for the deeper-lying soils in the cone penetration testing.

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).

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

Case IA Static using undrained strength: ϕ = 0° & c = 2.2 ksf.

Case IB Static using effective-stress strength: ϕ = 30° & c = 0.

The pads will be constructed on and within soil cement, as illustrated in SAR Figure 4.2-7 and described in SAR Sections 2.6.1.7 and 2.6.4.11. The unit weight of the soil cement is assumed to be 100 pcf in the bearing capacity analyses included herein. The strength of the soil cement is conservatively ignored in these bearing capacity analyses.

METHOD OF ANALYSIS

DESCRIPTION OF LOAD CASES

Load cases analyzed consist of combinations of vertical static, vertical dynamic (compression and uplift, Y-direction), and horizontal dynamic (in X and Z-directions) loads.

The following load combinations are analyzed:

Case I Static

Case II Static + dynamic horizontal forces due to the earthquake

Case III Static + dynamic horizontal + vertical uplift forces due to the earthquake

Case IV Static + dynamic horizontal + vertical compression forces due to the earthquake

For Case II, 100% of the dynamic lateral forces in both X and Z directions are combined. For Cases III and IV, the effects of the three components of the design basis ground motion are combined in accordance with procedures described in ASCE (1986) to account for the fact that the maximum response of the three orthogonal components of the earthquake do not occur at the same time. For these cases, 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the dynamic loading acts in the other two directions. For these cases, the suffix "A" is used to designate 40% in the X direction (N-S, as shown in Figure 1), 100% in the Y direction (vertical), and 40% in the Z direction (E-W). Similarly, the suffix "B" is used to designate 40% in the X direction, 40% in the Y, and 100% in the Z, and the suffix "C" is used to designate 100% in the X direction and 40% in the other two directions. Thus,

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.

The negative sign for the vertical direction in Case III indicates uplift forces due to the earthquake. Case IV is the same as Case III, but the vertical forces due to the earthquake act downward in compression; therefore, the signs on the vertical components are positive.

OVERTURNING STABILITY OF THE CASK STORAGE PADS

The factor of safety against overturning is defined as:

$$FS_{OT} = \Sigma M_{Resisting} \div \Sigma M_{Driving}$$

The resisting moment is calculated as the resultant weight of the pad and casks x the distance from one edge of the pad to the center of the pad in the direction of the minimum width. The weight of the pad is calculated as 3 ft x 67 ft x 30 ft x 0.15 kips/ft^3 = 904.5 K, and the weight of 8 casks is 8 x 356.5 K/cask = 2,852 K. The moment arm for the resisting moment equals ½ of 30 ft, or 15 ft. Therefore,

$$\Sigma M_{Resisting} = [\underbrace{904.5\text{ K}}_{Wp} + \underbrace{2{,}852\text{K}}_{Wc}] \times \underbrace{15\text{ ft}}_{B/2}\ \underbrace{(1\text{-}0.695)}_{(1-a_v)} = 17{,}186\text{ ft-K}$$

The driving moment includes the moments due to the horizontal inertial force of the pad x ½ the height of the pad and the horizontal force from the casks acting at the top of the pad x the height of the pad. The casks are simply resting on the top of the pads; therefore, this force cannot exceed the friction force acting between the steel bottom of the cask and the top of the concrete storage pad. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad (μ = 0.8, as shown in SAR Section 8.2.1.2) x the normal force acting between the casks and the pad. This force is maximum when the vertical inertial force due to the earthquake acts downward. However, when the vertical force from the earthquake acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the dynamic vertical force acts in the upward direction, tending to unload the pad.

When the vertical inertial force due to the earthquake acts upward, the friction force = 0.8 x (2,852K - 0.695 x 2,852K) = 696 K. This is less than the maximum dynamic cask horizontal driving force of 2,212 K (Table D-1(c) in CEC, 2001). Therefore, the worst-case horizontal force that can occur when the vertical earthquake force acts upward is limited by the upper-bound value of the coefficient of friction between the bottom of the casks and the top of the storage pad, and it equals 696 K.

$$\Sigma M_{Driving} = 1.5\text{ ft} \times \underbrace{0.711}_{a_h} \times \underbrace{904.5\text{ K}}_{Wp} + 3\text{ ft} \times \underbrace{696\text{ K}}_{EQhc} = 3{,}053\text{ ft-K}.$$

$$\Rightarrow FS_{OT} = \frac{17{,}186\text{ ft}-\text{K}}{3{,}053\text{ ft}-\text{K}} = 5.63$$

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.

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).

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

Material under and around the pad will be soil cement. In this analysis, however, the presence of the soil cement adjacent to the sides of the pads is ignored to demonstrate that there is an acceptable factor of safety against sliding of the pads along the interface between in situ clayey soils and bottom of soil cement beneath the pads. The potential failure mode is sliding along the surface at the base of the pad. No credit is taken for the passive resistance acting on the sides of the pad above the base. This analysis is applicable for any of the pads at the site, including those at the ends of the rows or columns of pads, since it relies only on the strength of the material beneath the pads to resist sliding.

This analysis conservatively assumes that 100% of the dynamic forces due to the earthquake act in both the horizontal and vertical directions at the same time. The length of the pad in the N-S direction (67 ft) is greater than twice the width in the E-W direction (30 ft); therefore, the dynamic active earth pressures acting on the length of the pad will be greater than those acting on the width, and the critical direction for sliding will be E-W, since passive resistance is ignored.

The soil cement is assumed to have the following properties in calculation of the dynamic active earth pressure acting on the pad from the soil cement above the base of the pad:

γ = 100-110 pcf	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 pads and that 100 pcf is a reasonable lower-bound value for the total unit weight of the cement-treated soil to be placed beneath the pads.
ϕ = 40°	Tables 5 & 6 of Nussbaum & Colley (1971) indicate that ϕ exceeds 40° for all A-4 soils (CL & ML, similar to the eolian silts at the site) treated with cement; therefore, it is likely that ϕ will be higher than this value. This value also is used in this analysis only for determining upper-bound estimates of the active earth pressure acting on the pad due to the design basis ground motion. Because of the magnitude of the earthquake, this analysis is not sensitive to increases in this value.
H = 5 ft	As shown in SAR Figure 4.2-7, the pad is 3 ft thick, and it is constructed such that top of the pad is at the final ground surface (i.e., pads are embedded 3' below grade). Soil cement beneath the pad is 1-ft to 2-ft thick. The dynamic forces (active earth pressure + horizontal inertial forces) are greater for deeper depth of soil cement. Therefore, analyze for 2 ft of soil cement beneath the pad.

ACTIVE EARTH PRESSURE

$P_a = 0.5\,\gamma\,H^2\,K_a$

$K_a = (1 - \sin\phi)/(1 + \sin\phi) = 0.22$ for $\phi = 40°$ for the soil cement, ignoring cohesion (very conservative).

$P_{a\,E\text{-}W}$ = [0.5 x 0.11 kcf x (5 ft)^2 x 0.22] x 67 ft (length)/storage pad = 20.3 K E-W.

$P_{a\,N\text{-}S}$ = [0.5 x 0.11 kcf x (5 ft)^2 x 0.22] x 30 ft (width)/storage pad = 9.1 K N-S.

DYNAMIC EARTH PRESSURE

As indicated on p 11 of GTG 6.15-1 (SWEC, 1982), for active conditions, the combined static and dynamic lateral earth pressure coefficient is computed according to the analysis developed by Mononobe-Okabe and described in Seed and Whitman (1970) as:

$$K_{AE} = \frac{(1-\alpha_V)\cdot\cos^2(\phi-\theta-\alpha)}{\cos\theta\cdot\cos^2\alpha\cdot\cos(\delta+\alpha+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\theta-\beta)}{\cos(\delta+\alpha+\theta)\cdot\cos(\beta-\alpha)}}\right]^2}$$

whe re :

$$\theta = \tan^{-1}\left(\frac{\alpha_H}{1-\alpha_V}\right)$$

β = slope of ground behind wall,

α = slope of back of wall to vertical,

α_H = horizontal seismic coefficient, where a positive value corresponds to a horizontal inertial force directed toward the wall,

α_V = vertical seismic coefficient, where a positive value corresponds to a vertical inertial force directed upward,

δ = angle of wall friction,

ϕ = friction angle of the soil,

g = acceleration due to gravity.

The combined static and dynamic active earth pressure force, P_{AE}, is calculated as:

$P_{AE} = \frac{1}{2}\gamma H^2 K_{AE}$, where :

γ = unit weight of soil,

H = wall height, and

K_{AE} is calculated as shown above.

To simplify the analysis, assume $\delta = 0$. This is conservative, as illustrated in Figure 12 of Seed and Whitman (1970), which indicates that K_{AE} decreases with increasing values of δ.

$\beta = \alpha = 0$

$$\theta = \tan^{-1}\left(\frac{0.711}{1-0.695}\right) = 66.8^\circ$$

$\phi = 40^\circ$

To obtain a real solution to the equation for calculating K_{AE}, the sin $(\phi - \theta - \beta)$ must be positive; i.e., the sin $(\phi - \theta - \beta)$ can vary from 0 to 1. Because it is in the denominator of K_{AE}, K_{AE} will be greatest when it = 0. Therefore, assume sin $(\phi - \theta - \beta) \approx 0$.

Similarly, approximate cos $(\phi - \theta - \alpha) \approx 1$. This term is in the numerator of K_{AE}, and K_{AE} will be maximum when cos $(\phi - \theta - \alpha) = 1$; therefore, approximating it equals 1 is conservative.

With these approximations,

$$K_{AE} = \frac{1 - \alpha_V}{\cos\theta \cdot \cos\theta}$$

$$\therefore \quad K_{AE} = \frac{1 - 0.695}{\cos^2 66.8^\circ} = 1.97$$

Therefore, the combined static and dynamic active lateral earth pressure force at the base of the 3 ft pad is:

$$F_{AE\,E\text{-}W} = P_{AE} = \frac{1}{2} \times \underset{\gamma}{0.110\text{ kcf}} \times \underset{H^2}{(3\text{ ft})^2} \times \underset{K_{AE}}{1.97} \times \underset{L}{67\text{ ft}} / \text{storage pad} = 65.3\text{ K in the E - W direction.}$$

$$F_{AE\,N\text{-}S} = P_{AE} = \frac{1}{2} \times 0.110\text{ kcf} \times (3\text{ ft})^2 \times 1.97 \times 30\text{ ft} / \text{storage pad} = 29.3\text{ K in the N - S direction.}$$

The combined static and dynamic active lateral earth pressure force at the base of the 3 ft pad and underlying 2 ft of soil cement is:

$$F_{AE\,E\text{-}W} = P_{AE} = \frac{1}{2} \times \underset{\gamma}{0.110\text{ kcf}} \times \underset{H^2}{(5\text{ ft})^2} \times \underset{K_{AE}}{1.97} \times \underset{L}{67\text{ ft}} / \text{storage pad} = 181.5\text{ K in the E - W direction.}$$

$$F_{AE\,N\text{-}S} = P_{AE} = \frac{1}{2} \times 0.110\text{ kcf} \times (5\text{ ft})^2 \times 1.97 \times 30\text{ ft} / \text{storage pad} = 81.3\text{ K in the N - S direction.}$$

WEIGHTS

Casks: Wc = 8 x 356.5 K/cask = 2,852 K

Pad: Wp = 3 ft x 67 ft x 30 ft x 0.15 kips/ft^3 = 904.5 K

Soil Cement Beneath Pad: Wsc = 2 ft x 67 ft x 30 ft x 0.10 kips/ft^3 = 402 K

EARTHQUAKE ACCELERATIONS – PSHA 2,000-YR RETURN PERIOD

a_H = horizontal earthquake acceleration = 0.711g

a_V = vertical earthquake acceleration = 0.695g

CASK EARTHQUAKE LOADINGS

EQvc = -0.695 x 2,852 K = -1,982 K (minus sign signifies uplift force)

$EQhc_{E-W}$ = 2,212 K (acting short direction of pad, E-W) $Q_{xd\ max}$ in Table D-1(c) in Att B

$EQhc_{N-S}$ = 2,102 K (acting in long direction of pad, N-S) $Q_{yd\ max}$ in Table D-1(c) "

Note: These maximum horizontal dynamic cask driving forces are from Calc 05996.02-G(PO17)-2, (CEC, 2001), and they apply only when the dynamic forces due to the earthquake act downward and the coefficient of friction between the cask and the pad equals 0.8. $EQh_{c\ max}$ is limited to a maximum value of 696 K for Case III, based on the upper-bound value of μ = 0.8, as shown in the following table:

Cask Loads	WT K	EQ_{Vc} K	N K	0.2 x N K	0.8 x N K	$EQ_{hc\ max}$ K
Case III – Uplift	2,852	-1,982	870	174	696	**696**
Case IV - EQ_v Down	2,852	1,982	4,834	967	3,867	**2,212** E-W 2,102 N-S

Note:

Case III: 0% N-S, -100% Vertical, 100% E-W Earthquake Forces Act Upward

Case IV: 0% N-S, 100% Vertical, 100% E-W Earthquake Forces Act Downward

FOUNDATION PAD EARTHQUAKE LOADINGS	**SOIL CEMENT BENEATH PAD EARTHQUAKE LOADINGS**
EQvp = -0.695 x 904.5 K = -629 K	EQvsc = -0.695 x 402 K = -279.4 K
EQhp = 0.711 x 904.5 K = 643 K	EQhp = 0.711 x 402 K = 285.8 K

CASE III: 0% N-S, -100% VERTICAL, 100% E-W (EARTHQUAKE FORCES ACT UPWARD)

When EQvc and EQvp act in an upward direction (Case III), tending to unload the pad, sliding resistance is obtained as follows:

$$N = \overset{Wc}{2{,}852\text{ K}} + \overset{Wp}{904.5\text{ K}} + \overset{Wsc}{402\text{ K}} + \overset{EQvc}{(-1{,}982\text{ K})} + \overset{EQvp}{(-629\text{ K})}\,) + \overset{EQvsc}{(-279.4\text{ K})} = 1{,}268.6\text{ K}$$

$$T = \overset{N}{1{,}268.6\text{ K}} \times \tan \overset{\phi}{0^\circ} + \overset{c}{2.1\text{ ksf}} \times \overset{B}{30\text{ ft}} \times \overset{L}{67\text{ ft}} = 4{,}221\text{ K}$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + Eqhc + EQhsc$$

The factor of safety against sliding is calculated as follows:

$$FS = \overset{T}{4{,}221\text{ K}} \div \underbrace{(\overset{F_{AE\ E\text{-}W\ 5'}}{181.5\text{ K}} + \overset{EQhp}{643\text{ K}} + \overset{EQhc}{696\text{ K}} + \overset{EQhsc}{285.8\text{ K}})}_{(1{,}806.3\text{ K})} = \mathbf{2.34}$$

For this analysis, the value of the horizontal driving force due to the earthquake, EQhc, is limited to the upper-bound value of the coefficient of friction, μ = 0.8, x the cask normal load, because if EQhc exceeds this value, the cask will slide. The factor of safety exceeds the minimum allowable value of 1.1; therefore the pads plus 2-ft block of soil cement beneath them are stable with respect to sliding for this load case. 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.

CASE IV: 0% N-S, 100% VERTICAL, 100% E-W (EARTHQUAKE FORCES ACT DOWNWARD)

When the earthquake forces act in the downward direction:

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

where, N (normal force) = Σ Fv = Wc + Wp + EQvc + Eqvp + EQvsc

$$N = \overset{Wc}{2{,}852\text{ K}} + \overset{Wp}{904.5\text{ K}} + \overset{EQvc}{1{,}982\text{ K}} + \overset{EQvp}{629\text{ K}} + \overset{Eqvsc}{279.4\text{ K}} = 6{,}647\text{ K}$$

$$T = \overset{N}{6{,}647\text{ K}} \times \tan \overset{\phi}{0^\circ} + \overset{c}{2.1\text{ ksf}} \times \overset{B}{30\text{ ft}} \times \overset{L}{67\text{ ft}} = 4{,}221\text{ K}$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + Eqhc + EQhsc$$

The factor of safety against sliding is calculated as follows:

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

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:

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

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

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

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

ADHESION BETWEEN THE BASE OF PAD AND UNDERLYING CLAYEY SOILS

The preceding analysis demonstrates that the static undrained strength of the soils underlying the pads is sufficient to preclude sliding of the cask storage pads over 2 ft of soil cement for the 2,000-yr return period earthquake with a peak horizontal ground acceleration of 0.711g, conservatively ignoring the passive resistance acting on the sides of the pads. This analysis assumes that the full static undrained strength of the clay is engaged to resist sliding. To obtain the minimum factor of safety required against sliding of 1.1, 76% (= 1.60 ksf (required for FS=1.1) ÷ 2.1 ksf available) of the undrained shear strength must be engaged, or in other words, the adhesion factor between the base of the concrete storage pads plus 2 ft of soil cement and the surface of the underlying clayey soils must be 0.76. This adhesion factor, c_a, is higher than would normally be used, considering disturbance that may occur to the surface of the subgrade during construction. Therefore, an "engineered mechanism" is required to ensure that the full strength of the clayey soils is available to resist sliding of these pads on 2 ft of soil cement.

Ordinarily, a foundation key would be added to extend the shear plane below the disturbed zone and to ensure that the full strength of the clayey soils are available to resist sliding forces. However, adding a key to the base of the storage pads would increase the stiffness of the foundation to such a degree that it would exceed the target hardness limitation of the hypothetical cask tipover analysis. Therefore, PFS decided to construct the cask storage pads on (and within) a layer of soil cement constructed throughout the entire pad emplacement area.

As shown in Figure 3, the soil cement will extend to the bottom of the eolian silt or a minimum of 1 ft below the base of the storage pads and up the vertical face at least 2 ft. In the sliding stability analysis, it is required that the following interfaces be strong enough to resist the sliding forces due to the design earthquake. Working from the bottom up, these include:

1. The interface between the in situ clayey soils and the bottom of the soil cement, and
2. The top of the soil cement and the bottom of the concrete storage pad.

The purpose of soil cement below the pads is to provide the "engineered mechanism" required to effectively transmit the sliding forces down into the underlying clayey soils. The techniques used to construct soil cement are such that the bond between the soil cement and the underlying clayey soils will exceed the undrained strength of the underlying clayey soils.

DeGroot (1976) indicates that this bond strength can be easily obtained between layers of soil cement. He performed nearly 300 laboratory direct shear tests to determine the effect of numerous variables on the bond between layers of soil cement. These variables included the length of time between placement of successive layers of soil cement, the frequency of watering while curing soil cement, the surface moisture condition prior to construction of the next lift, the surface texture prior to construction of the next lift, and

various surface treatments and additives. His results demonstrated that, with the exception of treating the surface of the lifts with asphalt emulsion, asphalt cutback, and chlorinated rubber compounds, the bond strength nearly always exceeded 11.10 psi, the minimum required value of shear strength of the bond between the base of the pads and the underlying material. The minimum bond strength he reports, other than for the asphalt and chlorinated rubber surface treatments identified above, is 7.7 psi. This value applied for only one test (Sample No. 15R-149, Series No. 3, Spec. No. 12) that was performed on a sample that had no special surface treatment along the lift line. This test, however, was anomalous, since all of the other specimens in this series had bond strengths in excess of 38.5 psi. He reports that nearly all of the specimens that used a cement surface treatment broke along planes other than along the lift lines, indicating that the bond between the layers of soil cement was stronger than the remainder of the specimens. Excluding the specimens that did not use the cement surface treatment, the minimum bond strength was 47.7 psi, which greatly exceeds the bond strength (11.10 psi) required to obtain an adequate factor of safety against sliding of the pads without including the passive resistance acting on the sides of the pads.

DeGroot reached the following conclusions:

1. Increasing the time delay between lifts decreases bond.
2. High frequency of watering the lift line decreases the bond.
3. Moist curing conditions between lift placements increases the bond.
4. Removing the smooth compaction plane increases the bond.
5. Set retardants decreased the bond at 4-hr time delay.
6. Asphalt and chlorinated rubber curing compounds decreased the bond.
7. Small amounts of cement placed on the lift line bonded the layers together, such that failure occurred along planes other than the lift line, indicating that the bond exceeded the shear strength of the soil cement.

DeGroot (1976) noted that increasing the time delay between placement of subsequent lifts decreases the bond strength. The nature of construction of soil cement is such that there will be occasions when the time delay will be greater than the time required for the soil cement to set. This will clearly be the case for construction of the concrete storage pads on top of the soil-cement surface, because it will take some period of time to form the pad, build the steel reinforcement, and pour the concrete. He noted that several techniques can be used to enhance the bond between lifts to overcome this decrease in bond due to time delay. In these cases, more than sufficient bond can be obtained between layers of soil cement and between the set soil-cement surface and the underside of the cask storage pads by simply using a cement surface treatment.

DeGroot's direct shear test results demonstrate that the specimens having a cement surface treatment all had bond strengths that ranged from 47.7 psi to 198.5 psi, with the

average bond strength of 132.5 psi. Even the minimum value of this range greatly exceeds the bond strength (11.10 psi) required to obtain a factor of safety against sliding of 1.1, conservatively ignoring the passive resistance available on the sides of the pads. Therefore, when required due to unavoidable time delays, the techniques DeGroot describes for enhancing bond strength will be used between the top of the soil cement and succeeding lifts or between the top of the soil cement and the concrete cask storage pads, to assure that the bond at the interfaces are greater than the minimum required value. These techniques will include roughening and cleaning the surface of the underlying soil cement, proper moisture conditioning, and using a cement surface treatment.

The shear strength available at each of the interfaces applicable to resisting sliding of the cask storage pads will exceed the undrained strength of the underlying clayey soils. PFS has committed (SAR p. 2.6-113) to performing laboratory tests during the design of the soil cement to demonstrate that the required shear strengths can be achieved at the various interfaces, and PFS has committed (SAR p. 2.6-114) to performing field tests during construction to demonstrate that the required shear strengths at these interfaces have been achieved.

The soil cement beneath the pads is used as an "engineered mechanism" to ensure that the full static undrained shear strength of the underlying clayey soils is engaged to resist sliding and, as shown above, the minimum factor of safety against sliding of the pads is very conservatively calculated as 1.27 when the static undrained strength of the clayey soils is fully engaged. This value exceeds the minimum value required for the factor of safety against sliding (=1.1); therefore, the pads constructed on top of a layer of soil cement have an adequate factor of safety against sliding.

LIMITATION OF STRENGTH OF SOIL CEMENT BENEATH THE PADS

As indicated in Figure 3, the soil cement will extend at least 1 ft below all of the cask storage pads, and, as shown in SAR Figures 2.6-5, Pad Emplacement Area Foundation Profile's, it will typically extend ~2 ft below most of the pads. Thus, the area available to resist sliding will greatly exceed that of the pads alone. The hypothetical cask tipover analysis imposes limitations on the modulus of elasticity of the soils underlying the pad. 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, but it must still provide an adequate factor of safety with respect to sliding of the pads embedded within the soil cement.

Table 5-6 of Bowles (1996) indicates $E = 1{,}500\ s_u$, where s_u = the undrained shear strength. Note, s_u is half of q_u, the unconfined compressive strength.

Based on this relationship, $E = 750\ q_u$,

Where E = Young's modulus

q_u = Unconfined compressive strength

An unconfined compressive strength of 100 psi for the soil cement under the pad will limit the modulus value to 75,000 psi. Thus, designing the soil cement to have an unconfined compressive strength that ranges from 40 psi to 100 psi will provide an adequate factor of safety against sliding and will limit the modulus of the soil cement under the pads to an acceptable level for the hypothetical cask tipover considerations.

SLIDING ALONG CONTACT BETWEEN THE CONCRETE PAD AND THE UNDERLYING SOIL CEMENT

The soil cement will be designed to have an unconfined compressive strength of at least 40 psi to ensure that it will be stronger than required to provide a factor of safety against sliding that exceeds the required minimum value of 1.1. The shear strength equals half of the unconfined compressive strength, 20 psi, which equals 2.88 ksf. Therefore, the resistance to sliding between the concrete storage pad and the top of the soil cement layer beneath the pad will be greater than:

$$T = \underset{N}{6{,}368\ K} \times \tan \underset{\phi}{0°} + \underset{c}{2.88\ ksf} \times \underset{B}{30\ ft} \times \underset{L}{67\ ft} = \underset{T}{5{,}789\ K}$$

As indicated above, the driving force, V, is defined as: $V = F_{AE} + EQhp + EQhc$

The factor of safety against sliding between the pad and the surface of the underlying soil cement is calculated as the resisting force ÷ the driving force, as follows:

$$\mathbf{FS_{Pad\ to\ Soil\ Cement}} = \underset{T}{5{,}789\ K} \div (\underset{F_{AE\ E\text{-}W}}{65.3\ K} + \underset{EQhp}{643\ K} + \underset{EQhc_{E\text{-}W}}{2{,}212\ K}) = \mathbf{1.98}$$

$$(2{,}920.3\ K)$$

Thus, designing the soil cement to have an unconfined compressive strength of at least 40 psi results in an acceptable factor of safety against sliding between the concrete at the base of the pad and the surface of the underlying soil cement that exceeds the factor of safety between the bottom of the soil cement and the underlying clayey soils. In other words, the soil cement will have higher strength than the underlying silty clay/clayey silt layer; therefore, the resistance to sliding on that interface will be limited by the strength of the silty clay/clayey silt.

Soil cement with strengths higher than this are readily achievable, as illustrated by the lowest curve in Figure 4.2 of ACI 230.1R-90, which applies for fine-grained soils similar to the eolian silt in the pad emplacement area. Note, f_c = 40C where C = percent cement in the soil cement. Therefore, to obtain f_c >40 psi, the percentage of cement required would be ~40/40 = 1%. This is even less cement than would typically be used in constructing soil cement for use as road base. The resulting material will more likely be properly classified as a cement-treated soil, rather than a true soil cement. Because this material is located below the frost zone (which is only 30" below grade at the site), it does not need to comply with the durability requirements of soil cement; i.e., ASTM freeze/thaw and wet/dry tests. The design of the mix for this material will require that the unconfined compressive strength of this layer of material will exceed 40 psi to ensure that the shear strength available to resist sliding of the concrete pads exceeds the shear strength of the in situ clayey soils.

SOIL CEMENT ABOVE THE BASE OF THE PADS

Soil cement also will be placed between the cask storage pads, above the base of the pads. Earlier versions of this calculation demonstrated that this soil cement could be designed such that its compressive strength alone would be sufficient to resist all of the sliding forces due to the design earthquake. However, as shown above, 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 and between that soil cement and the underlying clayey soils that the factor of safety against sliding exceeds the minimum required value. 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. The eolian silt, otherwise, would be inadequate for this purpose and would require replacement with imported structural fill. The soil cement surrounding the pad may also help to spread the seismic load into the clayey soil outside the pad area to engage additional resistance against sliding of the pad. This effect would result in an increase in the factor of safety against sliding.

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).

The beneficial effect of this soil cement on the factor of safety against sliding can be estimated by considering that the passive resistance provided by this soil cement is available to resist sliding before a sliding failure can occur. In this case, the shear strength of the clayey soils under the pad may be reduced to the residual strength, because of the horizontal displacement required to reach the full passive state. Note, the soil cement is much stiffer than normal soils; therefore, these horizontal displacements will not be as high as they typically are for soils to reach the full passive state.

The results of the direct shear tests, presented as plots of shear stress vs horizontal displacement in Attachment 7 of Appendix 2A of the SAR (copies included in Attachment D), illustrate that the residual strength of these soils is nearly equal to the peak strength. Looking at the test results for the specimens that were tested at confining stresses comparable to the loading at the base of the cask storage pads, σ_v ~2 ksf, at horizontal displacements of ~0.025" past the peak strength, there is ~1.5% reduction in the shear strength indicated for Sample U-1C from Boring C-2. Also note that Boring C-2 was drilled within the pad emplacement area. The results for Sample U-1AA from Boring CTB-S showed no decrease in shear strength following the peak at ~0.025" horizontal displacement, and Samples U-3B&C from Boring CTB-6 showed a decrease of ~5%.

Based of these results, conservatively assume that the strength of the clayey soils beneath the soil cement layer underlying the pads is reduced by 5% to account for horizontal straining required to reach the full passive resistance of the soil cement adjacent to the pad. This results in resisting forces acting on the base of the soil cement layer beneath each pad of 0.95 x 2.1 ksf x 30 ft x 67 ft = 4,010 K.

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

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

$$T_{N\text{-}S} = \underset{\text{Clay}}{4{,}010\text{ K}} + \underset{\text{Soil Cement}}{2{,}516\text{ K}} = 6{,}526\text{ K}$$

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

$$\mathbf{FS}_{\textbf{Pad to Clayey Soil N-S w/Passive}} = \underset{T_{N\text{-}S}}{6{,}526\text{ K}} \div \underbrace{(\underset{F_{AE\,N\text{-}S}}{29.3\text{ K}} + \underset{EQhp}{643\text{ K}} + \underset{Eqhc_{N\text{-}S}}{2{,}102\text{ K}})}_{(2{,}774.3\text{ K})} = \mathbf{2.35}$$

Ignoring the passive resistance provided by the soil cement adjacent to the pads, it is appropriate to use the peak shear strength of the underlying clayey soils, and the resulting FS against sliding in the N-S direction is calculated as:

$$\mathbf{FS}_{\textbf{Pad to Clayey Soil N-S w/o Passive}} = \underset{T_{N\text{-}S}}{4{,}221\text{ K}} \div \underbrace{(\underset{F_{AE\,N\text{-}S}}{29.3\text{ K}} + \underset{EQhp}{643\text{ K}} + \underset{Eqhc_{N\text{-}S}}{2{,}102\text{ K}})}_{(2{,}774.3\text{ K})} = \mathbf{1.52}$$

The resulting FS against sliding in the E-W direction will be even higher, since there is much greater length available to resist sliding in that direction. It is calculated as:

$$T_{\text{SC Adjacent to Pad}@\text{E\&W}} = 250\,\frac{\text{lbs}}{\text{in.}^2}\ \text{x}\left(\frac{12\text{ in.}}{\text{ft}}\right)^2\ \text{x}\ \frac{\text{K}}{1{,}000\text{ lbs}}\ \text{x } 2.33\text{ ft x } 67\text{ ft} = 5{,}620\text{ K}$$

$$T_{E\text{-}W} = \underset{\text{Clay}}{4{,}010\text{ K}} + \underset{\text{Soil Cement}}{5{,}620\text{ K}} = 9{,}630\text{ K}$$

$$\mathbf{FS}_{\textbf{Pad to Clayey Soil E-W}} = \underset{T_{E\text{-}W}}{9{,}630\text{ K}} \div \underbrace{(\underset{F_{AE\,E\text{-}W}}{65.3\text{ K}} + \underset{EQhp}{643\text{ K}} + \underset{EQhc_{E\text{-}W}}{2{,}212\text{ K}})}_{(2{,}920.3\text{ K})} = \mathbf{3.30}$$

These values are greater than the minimum value (1.1) required for factor of safety against sliding, and they ignore the beneficial effects of the 1 to 2-ft thick layer of soil cement underneath the concrete pad. Therefore, adding the soil cement adjacent to the pads does enhance the sliding stability of each pad.

SLIDING RESISTANCE OF ENTIRE N-S COLUMN OF PADS

The resistance to sliding of the entire column (running N-S) of pads exceeds that of each individual pad because there is more area available to engage more shearing resistance from the underlying soils than just the area directly beneath the individual pads. The extra area is provided by the 5-ft long x 30-ft wide plug of soil cement that exists between each of the pads in the north-south direction. This analysis assumes that the soil cement east and west of the long column of pads provides no resistance to sliding, conservatively assuming that the soil cement somehow shears along a vertical plane at the eastern and western sides of the column of 10 pads running north-south.

Consider a column of 10 pads with 2'-4" of soil cement in between the pads and at least 1' of soil cement under the pads:

Cask Earthquake Loads$_{N\text{-}S}$ = 10 x 2,102 K = 21,020 K

Inertial forces due to Pads + Soil Cement:

Weight of Pads	=	10 x 904.5 K	=	9,045 K
Weight of Soil Cement	=	9 x 3.33 ft x 30 ft x 5 ft x 0.11 kips/ft^3	=	495 K
		+10 x 30 ft x 67 ft x 1 ft x 0.11 kips/ft^3	=	2,211 K
		Total Weight	=	11,751 K

Inertial forces due to Pads + Soil Cement = 0.711 x 11,751 K = 8,355 K

Dynamic active earth pressure acting in the N-S direction on pads + 2 ft (more conservative than using 1 ft, since it results in higher driving forces) of soil cement beneath the pads = 81.3 K

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

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

This analysis conservatively ignores the passive resistance of the soil cement adjacent to the northern or southern end of the N-S column of pads. The resistance to sliding in the N-S direction is provided only by the shear strength of the soils underlying the soil cement layer beneath the pads (i.e., along Line IT in Figure 8). This case uses the soil cement beneath the pads as the engineered mechanism to bond the pads to the underlying clayey soils so that their peak shear strength can be engaged to resist sliding. As shown in Figure 7 on p. C2 of Attachment 2, the shear strength of the clayey soils under the pads is 2.1 ksf. The effective stresses under the soil cement between the pads is less than that directly under the pads; therefore, the shear strength available to resist sliding is lower. As shown in this figure, the shear strength available to resist sliding of the soil cement between the pads is 1.4 ksf. Using these strengths, the total resisting force is calculated as follows:

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:

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

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

As discussed above, conservatively assume that the strength of the clayey soils beneath the soil cement layer underlying the pads is reduced to its residual strength (i.e., by 5%) to account for horizontal straining required to reach a strain that will result in the full passive resistance of the soil cement adjacent to the pad.

$T_{N\text{-}S\ \text{Residual Strength}} = 0.95 \times 44{,}100\ K = 41{,}895\ K$

$$T_{N\text{-}S} = \underset{\text{Clay}}{41{,}895\ K} + \underset{\text{Soil Cement}}{2{,}516\ K} = 44{,}411\ K$$

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

$$FS_{\text{Pad to Clayey Soil N-S}} = \underset{T_{N\text{-}S}}{44{,}411\ K} \div \underset{\text{Driving Force}_{N\text{-}S}}{29{,}456\ K} = \underline{\mathbf{1.51}}$$

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

The resulting FS against sliding in the E-W direction will be even higher, since there is much greater length available to resist sliding in that 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 more than this, including only the difference between the length vs the width of the pad. The soil cement adjacent to the pad provides (67 ft ÷ 30 ft) x 2,516 K, or 5,619 K of resistance based on the full passive pressure acting on the length of the pad, which is an increase of 5,619 K – 2,516 K = 3,103 K compared to the resistance provided by the soil cement to sliding in the N-S direction. This is greater than the increase in driving forces in the E-W direction; therefore, the factor of safety against sliding will be higher in the E-W direction. The soil cement zone between the pads also is much wider and longer between the pads in the E-W direction; therefore, there will be even more resistance to sliding E-W than N-S.

DETERMINE RESIDUAL STRENGTH REQUIRED ALONG BASE OF ENTIRE COLUMN OF PADS IN N-S DIRECTION, ASSUMING FULL PASSIVE RESISTANCE IS PROVIDED BY 250 PSI SOIL CEMENT ADJACENT TO LAST PAD IN COLUMN

To obtain FS = 1.1, the total resisting force, T, must =

1.1 x [Cask Earthquake Loads + (Wt of Pads + Wt of Soil Cement) x 0.711 + $F_{AE\ N\text{-}S}$]

= 1.1 x [21,020 K + (11,751 K x 0.711) + 81.3 K]

Therefore, $T_{FS=1.1} = 32{,}402\ K$

In this case, the resisting forces to sliding in the N-S direction include all of the passive resistance at the far end of the column of pads, which acts on the 2'-4" height of soil cement along the 30' width of the pad in the E-W direction + the 1' minimum thickness of soil cement under the pads.

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05996.02	G(B)	04 - 9		

Assuming the soil cement adjacent to the pad is constructed such that its unconfined compressive strength is 250 psi, the passive resistance acting on the 2'-4" thickness of soil cement adjacent to the pad + a minimum of 1' below the pad will provide a force resisting sliding in the N-S direction of:

$$T_{SC\text{ Adjacent to Pad}_{@\,N\&S}} = 250\,\frac{\text{lbs}}{\text{in.}^2} \times \left(\frac{12\text{ in.}}{\text{ft}}\right)^2 \times \frac{\text{K}}{1{,}000\text{ lbs}} \times 3.33\text{ ft} \times 30\text{ ft} = 3{,}596\text{ K}$$

Base area, A, of a column of 10 pads is given by

$A = 10 \times 30\text{ ft} \times 67\text{ ft} + 9 \times 30\text{ ft} \times 5\text{ ft}$

$A = 20{,}100\text{ ft}^2 + 1{,}350\text{ ft}^2 = 21{,}450\text{ ft}^2$

Therefore the minimum shear strength required to provide the resisting force T is given by

$T_{N\text{-}S} = \tau \times \text{area (A)}$

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\text{ ft}^2 + \tau_{Soil\ Cement} \times 1{,}350\text{ ft}^2 = 32{,}402\text{ K} - 3{,}596\text{ K} = 28{,}806\text{ K}$

$\tau_{Pad} = 2.1$ ksf & $\tau_{Soil\ Cement} = 1.4$ ksf; thus, $\tau_{Soil\ Cement} = (1.4 \div 2.1) \times \tau_{Pad} = 0.67 \times \tau_{Pad}$

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\text{ ft}^2 + 0.67 \times \tau_{Pad} \times 1{,}350\text{ ft}^2 = \tau_{Pad} \times 21{,}000\text{ ft}^2$

$\tau_{Pad} \times 21{,}000\text{ ft}^2 = 28{,}806\text{ K}$

$\tau_{Pad} = 28{,}806\text{ K} \div 21{,}000\text{ ft}^2 = 1.37\text{ ksf}$

The peak shear strength of the clayey soils is 2.1 ksf. Therefore, the maximum reduction in peak strength permitted to obtain a factor of safety of 1.1 is calculated as:

$\Delta\tau = 1.37 \div 2.1 = 0.65$

In other words, the residual strength of the underlying clayey soils must drop below 65% of the peak shear strength before the factor of safety against sliding in the N-S direction of an entire column of pads will drop below 1.1.

Repeating this analysis, but ignoring the passive resistance of the soil cement adjacent to the pads at the northern or southern end of the column of pads,

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\text{ ft}^2 + \tau_{Soil\ Cement} \times 1{,}350\text{ ft}^2 = 32{,}402\text{ K}$

$\tau_{Pad} = 2.1$ ksf & $\tau_{Soil\ Cement} = 1.4$ ksf; thus, $\tau_{Soil\ Cement} = (1.4 \div 2.1) \times \tau_{Pad} = 0.67 \times \tau_{Pad}$

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\text{ ft}^2 + 0.67 \times \tau_{Pad} \times 1{,}350\text{ ft}^2 = \tau_{Pad} \times 21{,}000\text{ ft}^2$

$\tau_{Pad} \times 21{,}000\text{ ft}^2 = 32{,}402\text{ K}$

$\tau_{Pad} = 32{,}402\text{ K} \div 21{,}000\text{ ft}^2 = 1.54\text{ ksf}$

The peak shear strength of the underlying clayey soils is 2.1 ksf. Therefore, the maximum reduction in peak strength permitted to obtain a factor of safety of 1.1 is calculated as:

$\Delta\tau = 1.54 \div 2.1 = 0.73.$

In other words, even if the beneficial effects of the soil cement adjacent to the last pad in the N-S column of pads is ignored, the residual strength only needs to exceed 73% of the peak strength of the clayey soils to obtain a factor of safety against sliding in the N-S direction of an entire column of pads that is greater than 1.1.

As discussed above, the direct shear test results indicate that the greatest reduction between the peak shear strength and the residual shear strength is less than 5% for the specimens tested at effective stresses of 2 ksf, which are comparable to the final stresses under the fully loaded pads. The average reduction from peak stress is only ~20% for the specimens tested at effective vertical stresses of 1 ksf. Therefore, there is ample margin against sliding of an entire column of pads in the N-S direction.

SLIDING RESISTANCE OF LAST PAD IN COLUMN OF PADS ("EDGE EFFECTS")

Since the resistance to sliding of the cask storage pads is provided by the strength of the bond at the interface between the concrete pad and the underlying soil cement and by the bond between the soil cement under the pad and the in situ clayey soils, the sliding stability of the pads at the end of each column or row of pads are no different than that of the other pads. Therefore, the pads along the perimeter of the pad emplacement area also have an adequate factor of safety against sliding.

WIDTH OF SOIL CEMENT ADJACENT TO LAST PAD TO PROVIDE FULL PASSIVE RESISTANCE

As discussed above, the resisting force provided by the full passive resistance of the soil cement with an unconfined compressive strength of 250 psi acting on the last pad in the column of pads + a 1-ft thick layer of soil cement under the pad is:

$$T_{\text{SC Adjacent to Pad}_{@\,N\&S}} = 250\,\frac{\text{lbs}}{\text{in.}^2} \times \left(\frac{12\text{ in.}}{\text{ft}}\right)^2 \times \frac{\text{K}}{1{,}000\text{ lbs}} \times 3.33\text{ ft} \times 30\text{ ft} = 3{,}596\text{ K}$$

The base area required to provide this shear resistance = 30 ft x $L_{N\text{-}S}$ x 1.4 ksf, where 1.4 ksf is the shear strength of the underlying clayey soil for the effective vertical stress (~0.4 ksf) at the base of the soil cement layer beyond the end of the column of pads – See p C2.

$L_{N\text{-}S}$ = 3,596 K ÷ (30 ft x 1.4 ksf) = 85.62 ft.

Less than half of this amount is actually required due to 3D effects, similar to analysis of laterally loaded piles. Further, as shown above, the factor of safety against sliding of these pads exceeds the minimum allowable value without taking credit for the passive resistance provided by the soil cement adjacent to the pads. Therefore, this soil cement is not required for resisting sliding. However, the soil cement will be constructed adjacent to the pads, and it will extend further than this from the pads at the perimeter of the pad emplacement area. This soil cement will enhance the factor of safety against sliding, providing defense in depth against sliding of these pads due to the design ground motion.

SLIDING STABILITY OF THE PADS ASSUMING RESISTANCE IS BASED ON ONLY FRICTIONAL RESISTANCE ALONG BASE PLUS PASSIVE RESISTANCE

The design basis for the sliding stability of the cask storage pads relies on:

1. the assumption that sufficient "bonding" can be achieved at the interfaces between (a) the concrete comprising the pad and the soil cement beneath the pads, (b) soil cement lifts, and (c) soil cement and the underlying clayey soils such that the shear strength at these interfaces will be at least as high as the undrained strength measured in direct shear tests performed on samples of the underlying soils, and
2. the commitment to perform testing in the laboratory during the soil cement design phase to demonstrate that this "bonding" can be achieved, as well as during construction to demonstrate that this "bonding" has been achieved.

Laboratory testing to demonstrate the validity of this assumption are expected to be performed in the second half of 2001. Prior to completion of these tests, it is recognized that the resistance along the base of the pads + soil cement beneath the pads will be at least equal to the frictional resistance of the underlying soils, ignoring any contribution from the cohesive portion of the strength of these soils. Therefore, the purpose of this analysis is to demonstrate that even if the cohesion of the underlying soils is ignored along the interface between the soil cement and those soils, the resulting displacements of the pads would be minimal, and since there are no safety-related connections to these pads or casks, such displacements would have no safety consequence.

This hypothetical case assumes resistance to sliding is comprised of only frictional resistance along base of pads and soil cement + passive resistance, using obviously conservative values of the friction angle for the underlying soils. Although the resulting factor of safety is less than 1.1, the resulting maximum horizontal displacements, if they were to occur due to the earthquake, would be of no safety consequence to the pads or the casks.

Considering a single pad, assume that the shear strength available on the base of the pad to resist sliding is limited to that provided by friction alone. For this case, conservatively assume that friction is based on Table 1 of DM-7 (p. 7.2-63, NAVFAC, 1986), "Ultimate Friction Factors and Adhesion for Dissimilar Materials." This table indicates that an obviously conservative value of the friction angle for these clayey soils is 17 degrees. This is the lowest friction angle reported for the interface between mass concrete on any of the materials, and it applies for mass concrete on either "Fine sandy silt, nonplastic silt" or "Medium stiff and stiff clay and silty clay." Without including the cohesion, the resulting shear strength available to resist sliding of the pad is calculated as N tan ϕ. N = 1,146 K, as shown on p. 21:

SLIDING STABILITY OF THE PADS ASSUMING RESISTANCE IS BASED ON ONLY FRICTIONAL RESISTANCE ALONG BASE PLUS PASSIVE RESISTANCE

$$N = \underset{Wc}{2{,}852\text{ K}} + \underset{Wp}{904.5\text{ K}} + \underset{EQvc}{(-1{,}982\text{ K})} + \underset{EQvp}{(-629\text{ K})} = 1{,}146\text{ K}$$

$$T = \underset{N}{1{,}146\text{ K}} \times \tan \underset{\phi}{17^\circ} + \underset{c}{0\text{ ksf}} \times \underset{B}{30\text{ ft}} \times \underset{L}{67\text{ ft}} = 350.4\text{ K}$$

The driving force, V, is defined as: $V = F_{AE} + EQhp + EQhc$

The factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{350.4\text{ K}} \div \underset{(1{,}368.3\text{ K})}{(\underset{F_{AE\,N\text{-}S}}{29.3\text{ K}} + \underset{EQhp}{643\text{ K}} + \underset{EQhc}{696\text{ K}})} = 0.26$$

This analysis assumes that the maximum forces due to the earthquake act in both the north-south and vertical directions at the same time, which is not the case, and, thus, is overly conservative. Combining the effects of the earthquake components in accordance with ASCE 4-86, 100% of the vertical forces are assumed to act at the same time that 40% of the maximum forces act in the other two orthogonal directions. This results in the following, for a single pad:

Case IIIA: 40% N-S, -100% Vertical, 40% E-W (Earthquake Forces Act Upward)

$$N = \underset{Wc}{2{,}852\text{ K}} + \underset{Wp}{904.5\text{ K}} + \underset{EQvc}{(-1{,}982\text{ K})} + \underset{EQvp}{(-629\text{ K})} = 1{,}146\text{ K}$$

$$T = \underset{N}{1{,}146\text{ K}} \times \tan \underset{\phi}{17^\circ} + \underset{c}{0\text{ ksf}} \times \underset{B}{30\text{ ft}} \times \underset{L}{67\text{ ft}} = 350.4\text{ K}$$

The driving force, V, is defined as $V = F_{AE} + EQhp + Eqhc$, and using 40% in the north-south direction for this case (Case IIIA), the factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{350.4\text{ K}} \div \underset{(964.9\text{ K})}{[\underset{40\%\text{ of }[F_{AE\,N\text{-}S}}{0.4 \times (29.3\text{ K}} + \underset{EQhp}{643\text{ K})} + \underset{Eqhc]}{696\text{ K}}]} = 0.36$$

In this case, note that $Eqhc_{N\text{-}S}$ = the minimum of 0.4 x $Eq_{hc\ max\ N\text{-}S}$ and 0.8 x N_{Casks}.

$Eq_{hc\ max\ N\text{-}S}$ = 2,101 K, as shown in the table on p. 20; thus, 40% of it = 841K.

0.8 x N_{Casks} = 696 K, as shown in the table on p. 20; therefore, $Eqhc_{N\text{-}S}$ equals 696 K. This is the maximum horizontal force that can be transmitted from the casks to the top of the pad due to friction.

SLIDING STABILITY OF THE PADS ASSUMING RESISTANCE IS BASED ON ONLY FRICTIONAL RESISTANCE ALONG BASE PLUS PASSIVE RESISTANCE

To ensure the pad does not slide, the factor of safety should be greater than 1.1. Therefore, the resistance to sliding must be increased by 1.1 x 965 K – 350 K, or 615 K.

The soil cement adjacent to the pad is 2'-4" deep and 30' wide. The resisting force provided by the soil cement adjacent to the pad is calculated as the unconfined compressive strength, q_u, of the soil cement, multiplied by the area of the end of the pad, which equals 2.33' x 30'. Therefore,

$$q_u = \frac{615\text{ K}}{2.33\text{ ft} \times 30\text{ ft}} = 8.8\frac{\text{K}}{\text{ft}^2} \times \frac{\text{ft}^2}{(12\text{ in.})^2} \times \frac{1{,}000\text{ lbs}}{\text{K}} = 61.1\text{ psi}$$

As indicated above, in the section titled " Soil Cement Above the Base of the Pads":

> *"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)."*

Therefore, the resistance required to prevent an individual pad from sliding can readily be provided by passive resistance from the soil cement adjacent to the pad, ***if the soil cement can be demonstrated to stay in place*** to provide that resistance. Sliding of the soil cement is resisted by the shear strength along the base of the soil cement layer and the passive resistance of the in situ soils at the edge of the soil cement away from the pad, where the soil cement bears against the existing soils. The shear resistance available at the bottom of the soil cement is insignificant if we include only the frictional portion of the strength of the underlying clayey soils, ignoring the cohesive portion of the strength.

The following hypothetical analysis demonstrates that, even without imposing the horizontal loads from the pads, the frictional resistance along the base of the soil cement layer is not sufficient to preclude sliding of the soil cement block itself due to the earthquake loads.

The soil cement layer will be approximately 5-ft thick over most of the pad emplacement area; therefore, consider the sliding stability of a block of soil cement adjacent to the pads that is 5-ft thick. For Case IIIA, where 100% of the vertical earthquake forces act upward, tending to unload the soil cement, the normal stress at the base of the soil cement is very small. Preliminary results of the moisture-density tests that have been performed to-date on the soil-cement specimens indicate that 110 pcf is a reasonable unit weight to use for the soil cement adjacent to the pads. Without the earthquake loading, the normal stress at the base of the 5-ft deep soil cement layer is 5' x 0.110 kcf = 0.55 ksf. Subtracting the uplift forces, the normal stress is reduced to (1 – 0.695) x 0.55 ksf = 0.168 ksf. The shear resistance available due to friction at the base of the soil cement overlying the clayey soils is calculated as N tan ϕ, or 0.168 ksf x tan 17° = 0.051 ksf.

SLIDING STABILITY OF THE PADS ASSUMING RESISTANCE IS BASED ON ONLY FRICTIONAL RESISTANCE ALONG BASE PLUS PASSIVE RESISTANCE

Assume there are no external forces acting on this block of soil cement, other than the horizontal and vertical dynamic forces due to the earthquake. In reality, there will be large horizontal forces imposed on the soil cement block from the pad, but these are ignored in this example to demonstrate the point that the soil cement cannot preclude sliding of the soil cement block itself during the earthquake ***based only on the frictional resistance*** along its base.

In this hypothetical case, the driving forces are due to the horizontal inertia of the soil-cement block. The maximum horizontal driving force is calculated as the mass of the block x the peak horizontal acceleration, 0.711g, which equals 0.711g x 5' x 0.110 kcf/g x the width and length of the block of soil cement. The resulting horizontal shear stress at the base of the block = 0.39 ksf. In this case (Case IIIA) only 40% of this value is considered to act horizontally at the same time as the full uplift force, resulting in a maximum horizontal shear stress due to the driving force of 0.4 x 0.39 ksf = 0.156 ksf.

The factor of safety against sliding is calculated as the resisting forces ÷ the driving forces, or, since the area of the base of the block is the same for resisting and driving forces,

$$FS_{\text{Soil-cement Block}}\,\text{Case IIIA} = \frac{\text{Shear Strength Due to Friction}}{\text{Shear Stress Due to Horiz Inertia}} = \frac{0.051\text{ ksf}}{0.156\text{ ksf}} = 0.33$$

Similar results apply for Loading Case IIIC, where 100% of the earthquake forces are assumed to act in the north-south direction when 40% act in the other two orthogonal directions; e.g.,

$$FS_{\text{Soil-cement Block}}\,\text{Case IIIC} = \frac{(1-0.4\times0.695)\times5\text{ ft}\times0.11\text{ kcf}\times\tan17^\circ}{100\%\times0.711\times5\text{ ft}\times0.11\text{ kcf}} = \frac{0.121\text{ ksf}}{0.391\text{ ksf}} = 0.31$$

Thus, the soil cement cannot provide adequate resistance ***based solely on the friction acting along its base*** to preclude sliding of the pad. As a matter of fact, the soil cement cannot even resist sliding of itself during the earthquake ***if only the frictional portion of the strength is assumed to be available*** along its base. Even using an unreasonably high value of the friction angle in this calculation, say 40°, the factor of safety against sliding of the soil-cement block is still not adequate to preclude sliding of the block due to only the inertia forces of the block itself; e.g.,

$$FS_{\text{Soil-cement Block}}\,\frac{\text{Case IIIA}}{\text{w}/\phi=40^\circ} = \frac{(1-0.695)\times5\text{ ft}\times0.11\text{ kcf}\times\tan40^\circ}{40\%\times0.711\times5\text{ ft}\times0.11\text{ kcf}} = \frac{0.141\text{ ksf}}{0.156\text{ ksf}} = 0.90$$

Therefore, the effects of the frictional resistance acting on the base of the soil-cement block are ignored in the following hypothetical analysis of the factor of safety against sliding of a single pad.

The passive resistance at the edge of the soil cement, where it bears against the existing soil, is included, however. The soil cement layer is 5-ft deep at the edge away from the end of the pad. The passive resistance of the soils at this edge is calculated as follows. In this case, assume the strength of the soil is based on the triaxial test results presented in

Attachment 8 of Appendix 2A of the SAR. A copy of the summary plot of these test results is included in Attachment E of this calculation, and it indicates c = 1.4 ksf and ϕ = 21.3°.

Equation 23.7 of Lambe and Whitman (1969) indicates that the passive resisting force, P_p, is calculated as:

$$P_p = \frac{1}{2}\gamma_b \times H^2 \times N_\phi + 2c \times H \times \sqrt{N_\phi},$$

where $N_\phi = \frac{1 + \sin\phi}{1 - \sin\phi} = \frac{1 + \sin 21.3^\circ}{1 - \sin 21.3^\circ} = 2.14$ Eq 23.2 Lambe & Whitman (1969)

and H = 5 ft

$$\therefore\ P_p = \frac{1}{2} 0.080\ \text{kcf} \times (5\ \text{ft})^2 \times 2.14 + 2 \times 1.4\ \text{ksf} \times 5\ \text{ft} \times \sqrt{2.14} = 20.91\ \text{K/LF}$$

For the 30 ft width of the pad, full passive resistance of the in situ soils = 30 ft × 20.91 K/LF = 627.3 K.

Thus, for a single pad, the factor of safety against sliding based on friction acting on the base of the pad and the full passive resistance of the existing soils is calculated as follows:

$$FS = \underbrace{(\overset{T}{350.4\ K} + \overset{P_p}{627.3\ K})}_{(977.7\ K)} \div \underbrace{[\overset{40\%\ of\ [F_{AE\ N\text{-}S}}{0.4 \times (29.3\ K} + \overset{EQhp}{643\ K)} + \overset{Eqhc]}{696\ K}]}_{(964.9\ K)} = 1.01$$

This is less than 1.1, the minimum acceptable factor of safety to preclude sliding of the pads. Therefore, a single pad is not stable for the loads associated with Case IIIA, ***assuming that resistance to sliding is provided only by friction acting on the base*** of the pads and the full passive resistance of the site soils.

Check Sliding of an Entire Row of Pads in the North-South Direction for the Hypothetical Case Where Resistance Along the Base Is Due Solely to Frictional Resistance

Note, the length of the pads, 67 ft in the north-south direction, is more than twice the width, 30 ft in the east-west direction; therefore, the resistance to sliding is greater in the east-west direction when passive resistance is considered. Thus, these analyses are performed for sliding in the north-south direction.

Considering one north-south row of pads, assume that the shear strength available on the base of the pads to resist sliding is limited to that provided by friction alone. As discussed above, the resulting shear strength available to resist sliding of each pad is calculated as N tan ϕ. N = 1,146 K, calculated as follows:

$$N = \overset{Wc}{2{,}852\ K} + \overset{Wp}{904.5\ K} + \overset{EQvc}{(-1{,}982\ K)} + \overset{EQvp}{(-629\ K)} = 1{,}146\ K$$

$$T = \overset{N}{1{,}146\ K} \times \tan \overset{\phi}{17^\circ} + \overset{c}{0\ ksf} \times \overset{B}{30\ ft} \times \overset{L}{67\ ft} = 350.4\ K$$

Therefore, the total resistance due to friction acting on the base of 20 pads in the row is 20 x 350.4 K = 7,008 K. Note, ϕ is assumed to be 17°, an obviously conservative value based on Table 1 on p. 7.2-63 of DM-7 (NAVFAC, 1986), as discussed above.

The passive resistance of the soils at the edge of the 5-ft deep layer of soil cement away from the end of the pad is available to resist sliding of the entire row of pads. It is calculated, as shown above, and it equals 20.91 K/LF of width of the 5-ft deep soil cement layer surrounding the pad emplacement area. For a strip 30-ft wide at either the northern or southern end of the row of pads, this provides an additional resistance to sliding of 627.3 K. It is reasonable to expect that, due to 3D effects, the soil cement will distribute the horizontal loads from the row of pads over more than just the 30-ft width of the pad. This passive resistance would be limited, however, to the width of the pad, 30 ft, + the width of the aisle between the rows of pads north-south, 35 ft. Thus, the maximum credible contribution of the passive resistance of the existing soils at the edge of the soil-cement layer north or south of the entire row of pads is 20.91 K/LF x (30' + 35'), which equals 1,359 K.

As shown above, the shear strength available due to friction along the base of the soil cement between the pads and at the end of the row of pads (0.051 ksf) is not sufficient to resist the inertial forces of the soil cement (0.156 ksf) and, thus, is ignored in this analysis. It is recognized that the forces due to the difference between this frictional shear strength along the base of the soil cement and the horizontal shear stresses due to the inertial forces should be accounted for in the analysis of sliding, but it is ignored in this example to demonstrate the point that the soil cement cannot preclude sliding of the entire row of pads if the resistance along the base of the soil cement ***is limited to only the frictional component***.

Therefore, the total resisting force available for the entire row of 20 pads due to only friction along the base of the row + passive resistance of the existing soils at the edge of the soil cement = 7,008 K + 627.3 K = 7,635.3 K. If 3D effects are included to distribute the horizontal loads beyond the 30-ft width of the pad, the maximum credible resisting force is 7,008 K + 1,359 K = 8,367 K.

The driving force, V, is defined as V = F_{AE} + EQhp + EQhc. For the entire row of 20 pads, the maximum horizontal driving force is calculated as:

$$V = \overset{F_{AE\ N\text{-}S}}{29.3\text{ K}} + 20\text{ pads} \times [\overset{EQhp}{643\text{ K}} + \overset{EQhc}{696\text{ K}}] = 26{,}809\text{ K.}$$

For Case IIIA, 40% of the horizontal driving force is assumed to act in the north-south direction at the same time as 100% of the uplift force due to the earthquake. Thus, the driving force for Case $IIIA_{N\text{-}S}$ is:

$$V_{IIIA\ N\text{-}S} = 0.4 \times (\overset{F_{AE\ N\text{-}S}}{29.3\text{ K}} + 20\text{ pads} \times \overset{EQhp}{643\text{ K}}) + 20\text{ pads} \times \overset{EQhc}{696\text{ K}} = 19{,}076\text{ K.}$$

And the factor of safety against sliding of the entire row for Case IIIA is calculated as follows:

$$FS = \overset{T}{7{,}635.3\text{ K}} \div \overset{40\%\text{ of }F_{AE\ N\text{-}S} + EQhp + EQhc}{19{,}076\text{ K}} = 0.40$$

or, for the maximum credible passive resistance, relying on distribution of the horizontal loads through the soil cement in to the soils due to 3D effects, the factor of safety against sliding is calculated as follows:

$$FS = \overset{T}{8{,}367\text{ K}} \div \overset{40\%\text{ of }F_{AE\ N\text{-}S} + EQhp + EQhc}{19{,}076\text{ K}} = 0.44$$

These values are less than 1.1; therefore, assuming the resistance to sliding is provided only by frictional resistance along the base of the row of pads and soil cement + passive resistance available at the edge of the soil cement, the pads might slide due to the design earthquake. As indicated in Section 4.4.2 of the Storage Facility Design Criteria (Stone & Webster, 2000),

> *"Where the factor of safety against sliding is less than 1 due to the design basis ground motion, the displacements the structure may experience are calculated using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes. The magnitude of these displacements are evaluated to assess the impact on the performance of the structure."*

The following analyses estimate the horizontal displacement of the pads, assuming they are supported directly on frictional soils with ϕ = 17°. These analyses are based on the method proposed by Newmark (1965) to estimate the displacement of the pads, which is described in the section titled " Evaluation of Sliding on Deep Slip Surface Beneath Pads."

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

20 Pads in N-S Row

Static Vertical Force, F_v = W = Weight of casks, pads, and soil cement in the row

Pads + Casks = $20 \times [904.5\text{ K} + 2{,}852\text{ K}] = \mathbf{75{,}130\text{ K}}$

Soil cement adjacent to pads is 30 ft wide and 3 ft deep =

$$30\text{ ft width} \times 3\text{ ft deep} \times \left[9\,\frac{\text{gaps}}{\text{area}} \times 5\text{ ft}\,\frac{\text{length}}{\text{gap}} \times 2\text{ areas} + 90\text{ ft between areas}\right] \times 0.110\text{ kcf} = \mathbf{1{,}782\text{ K}}$$

Soil cement 2 ft deep beneath the pads, which are 30 ft wide =

$$30\text{ ft} \times 2\text{ ft} \times \left[20\text{ pads} \times 67\,\frac{\text{ft}}{\text{pad}} + 9\,\frac{\text{gaps}}{\text{area}} \times 5\text{ ft}\,\frac{\text{length}}{\text{gap}} \times 2\text{ areas} + 90\text{ ft between areas}\right]$$

$$\times 0.100\text{ kcf} = \mathbf{9{,}120\text{ K}}$$

$$\Rightarrow \text{Fv} = 75{,}130\text{ K} + 1{,}728\text{ K} + 9{,}120\text{ K} = \mathbf{86{,}032\text{ K}}$$

Earthquake Vertical Force, $F_{v\,Eqk}$ = a_v x W/g = 0.695g x 86,032 K/g = 59,792 K

$\phi = 17°$

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

$$\mathbf{N} = [(\underbrace{86{,}032}_{F_v} - \underbrace{59{,}792}_{F_{v\,Eqk}}) \tan \underbrace{17°}_{\phi} + \underbrace{627.3\text{ K}}_{Pp}] / \underbrace{86{,}032}_{W} = 0.101$$

Acceleration in N-S direction, A = 0.284g

Velocity in N-S direction, V = 13.7 in./sec

$$\Rightarrow \quad \mathbf{N} / A = 0.101 / 0.284 = 0.354$$

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

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

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

$$\Rightarrow \quad u_m = \left(\frac{(13.7\text{ in./sec})^2 \cdot (1 - 0.354)}{2 \cdot 386.4\text{ in./sec}^2 \cdot 0.101}\right) = 1.55''$$

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

$$u_m = [V^2] / (2g\mathbf{N})$$

$$\Rightarrow \quad u_m = \left(\frac{(13.7 \text{ in./sec})^2}{2 \cdot 386.4 \text{ in./sec}^2 \cdot 0.101}\right) = 2.40''$$

In this case, N /A is = 0.354. As shown in Figure 5, at this value of N/A, the data points for actual earthquake records are between the two curves, and the maximum displacement is closer to the average of these two curves. Therefore, use the average of the maximum displacements calculated above, or the maximum displacement is 1.98 inches.

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

Since the pads are longer in the north-south direction than in the east-west direction, the passive resistance available to resist sliding in the east-west direction will be greater than that resisting sliding in the north-south direction. Thus, sliding in the north-south direction is more critical than sliding east-west. See Load Case IIIC for estimate of displacement in the north-south direction.

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

Static Vertical Force, F_v = W = 86,032 K

Earthquake Vertical Force, $F_{v(Eqk)}$ = 59,792 K x 0.40 = 23,917 K

$$\phi = 17^\circ$$

$$\mathbf{N} = [(\underset{F_v}{86{,}032} - \underset{F_{v\,Eqk}}{23{,}917}) \tan \underset{\phi}{17^\circ} + \underset{Pp}{627.3\text{ K}}] / \underset{W}{86{,}032} = 0.228$$

Acceleration in N-S direction, A = 0.711g

Velocity in N-S direction, V = 34.1 in./sec

$$\Rightarrow \quad \mathbf{N} / A = 0.228 / 0.711 = 0.321$$

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

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

$$\Rightarrow \quad u_m = \left(\frac{(34.1 \text{ in./sec})^2 \cdot (1 - 0.321)}{2 \cdot 386.4 \text{ in./sec}^2 \cdot 0.228}\right) = 4.48''$$

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

$$u_m = [V^2] / (2g\mathbf{N})$$

$$\Rightarrow \quad u_m = \left(\frac{(34.1\,\text{in./sec})^2}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.228}\right) = 6.60''$$

In this case, N /A is = 0.321. As shown in Figure 5, at this value of N/A, the data points for actual earthquake records are between the two curves; the data points for actual earthquake records are between the two curves, and the maximum displacement is closer to the upper curve. Therefore, the maximum displacement is ~6 inches.

SUMMARY OF HORIZONTAL DISPLACEMENTS CALCULATED BASED ON NEWMARK'S METHOD FOR ASSUMPTION THAT CASK STORAGE PADS ARE FOUNDED DIRECTLY ON COHESIONLESS SOILS WITH ϕ = 17° AND PASSIVE PRESSURE DUE TO SITE SOILS ACTS ON 5-FT THICK LAYER OF SOIL CEMENT AT END OF ROW OF 20 PADS

LOAD COMBINATION				DISPLACEMENT
Case IIIA	40% N-S	-100% Vert	40% E-W	~2 inches
Case IIIB	40% N-S	-40% Vert	100% E-W	< Case IIIC
Case IIIC	100% N-S	-40% Vert	40% E-W	~6 inches

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with ϕ = 17°, 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 ~2 inches to ~6 inches. There are several conservative assumptions that were made in determining these values for this hypothetical case, and, therefore, the estimated displacements represent upper-bound values. Even if the maximum horizontal displacement were to occur from an earthquake, there would be no safety consequence to the pads or the casks, since the pads and casks do not rely on any external "Important to Safety" connections.

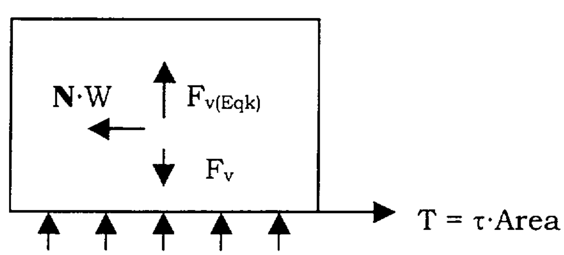
EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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, $\mathbf{N}\cdot W = T$,

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

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

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

$\mathbf{N}\,W = T$

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

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

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

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.

$$u_m = V^2 / (2g\mathbf{N})$$

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.

EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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 x 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

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

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

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

$$\Rightarrow \quad \mathbf{N} / A = 0.176 / 0.402 = 0.438$$

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

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

where g is in units of inches/sec².

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

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

$$u_m = [V^2] / (2g\mathbf{N})$$

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

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

$$\phi = 30°$$

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

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

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

$$\Rightarrow \quad \mathbf{N} / A = 0.417 / 0.766 = 0.544$$

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

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

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

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 ϕ = 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 ϕ 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.

ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS

The bearing capacity for shallow foundations is determined using the general bearing capacity equation and associated factors, as referenced in Winterkorn and Fang (1975). The general bearing capacity equation is a modification of Terzaghi's bearing capacity equation, which was developed for strip footings and indicates that $q_{ult} = c \cdot N_c + q \cdot N_q + ½ \gamma \cdot B \cdot N_\gamma$. The ultimate bearing capacity of soil consists of three components: 1) cohesion, 2) surcharge, and 3) friction, which are represented by the bearing capacity factors N_c, N_q, and N_γ. Terzaghi's bearing capacity equation has been enhanced by various investigators to incorporate shape, depth, and load inclination factors for different foundation geometries and loads as follows:

$$q_{ult} = c\, N_c\, s_c\, d_c\, i_c + q\, N_q\, s_q\, d_q\, i_q + ½\, \gamma\, B\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma$$

where

q_{ult} = ultimate bearing capacity

c = cohesion or undrained strength

q = effective surcharge at bottom of foundation, = γD_f

γ = unit weight of soil

B = foundation width

s_c, s_q, s_γ = shape factors, which are a function of foundation width to length

d_c, d_q, d_γ = depth factors, which account for embedment effects

i_c, i_q, i_γ = load inclination factors

N_C, N_q, N_γ = bearing capacity factors, which are a function of ϕ.

γ in the third term is the unit weight of soil below the foundation, whereas the unit weight of the soil above the bottom of the footing is used in determining q in the second term.

BEARING CAPACITY FACTORS

Bearing capacity factors are computed based on relationships proposed by Vesic (1973), which are presented in Chapter 3 of Winterkorn and Fang (1975). The shape, depth and load inclination factors are calculated as follows:

$$N_q = e^{\pi \tan\phi} \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$N_c = (N_q - 1)\ \cot\phi, \text{ but } = 5.14 \text{ for } \phi = 0.$$

$$N_\gamma = 2\ (N_q + 1)\ \tan\phi$$

SHAPE FACTORS (FOR *L>B*)

$$s_c = 1 + \frac{B}{L} \cdot \frac{N_q}{N_c}$$

$$s_q = 1 + \frac{B}{L} \tan\phi$$

$$s_\gamma = 1 - 0.4\frac{B}{L}$$

DEPTH FACTORS (FOR $\frac{D_f}{B} \le 1$)

$$d_c = d_q - \frac{(1 - d_q)}{N_q \cdot \tan\phi} \text{ for } \phi > 0 \text{ and } d_c = 1 + 0.4\left(\frac{D_f}{B}\right) \text{ for } \phi = 0.$$

$$d_q = 1 + 2\tan\phi \cdot (1 - \sin\phi)^2 \cdot \left(\frac{D_f}{B}\right)$$

$$d_\gamma = 1$$

INCLINATION FACTORS

$$i_q = \left(1 - \frac{F_H}{F_V + B'L'c\cot\phi}\right)^m$$

$$i_c = i_q - \frac{(1 - i_q)}{N_c \cdot \tan\phi} \text{ for } \phi > 0 \text{ and } i_c = 1 - \left(\frac{m F_H}{B'L'c N_c}\right) \text{ for } \phi = 0$$

$$i_\gamma = \left(1 - \frac{F_H}{F_V + B'L'c\cot\phi}\right)^{m+1}$$

Where: F_H and F_V are the total horizontal and vertical forces acting on the footing and

m_B = (2 + B/L) / (1 + B/L)

m_L = (2 + L/B) / (1 + L/B)

STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

The following pages present the details of the bearing capacity analyses for the static load cases. These cases are identified as follows:

Case IA Static using undrained strength parameters (ϕ = 0° & c = 2.2 ksf).

Case IB Static using effective-stress strength parameters (ϕ = 30° & c = 0).

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STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

Allowable Bearing Capacity of Cask Storage Pads

Static Analysis: **Case IA - Static**

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)	

Foundation Properties:

$B' =$	**30.0**	Footing Width - ft (E-W)	$L' =$ **67.0** Length - ft (N-S)
$D_f =$	**3.0**	Depth of Footing (ft)	
			0 $g = a_H$
$FS =$	**3.0**	Factor of Safety required for $q_{allowable}$	**0** $g = a_V$

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

$EQ_{H\,E\text{-}W} =$ **0** k & $EQ_{H\,N\text{-}S} =$ **0** k → **0** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.82	"
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 $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.04	Eq 3.27

No inclined loads; therefore, $i_c = i_q = i_\gamma = 1.0$.

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**13,085**	**psf =**	**12,785**	**+**	**300**	**+**	**0**
q_{all} =	**4,360**	**psf = q_{ult} / FS**					
q_{actual} =	**1,869**	**psf = $(F_{V\,Static} + EQ_V) / (B' \times L')$**					
FS_{actual} =	**7.00**	**$= q_{ult} / q_{actual}$**			**> 3**	**Hence OK**	

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STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

Allowable Bearing Capacity of Cask Storage Pads

Static Analysis: **Case IB - Static**

Soil Properties:	$c =$	**0**	Cohesion (psf)		
Effective Stress Strengths	$\phi =$	**30.0**	Friction Angle (degrees)		
	$\gamma =$	**80**	Unit weight of soil (pcf)		
	$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)		
Foundation Properties:	$B' =$	**30.0**	Footing Width - ft (E-W)	$L' =$ **67.0**	Length - ft (N-S)
	$D_f =$	**3.0**	Depth of Footing (ft)		
					0 $g = a_H$
	$FS =$	**3.0**	Factor of Safety required for $q_{allowable}$		**0** $g = a_V$
	$F_{V\ Static} =$	**3,757** k	& $EQ_V =$ 0 k →	**3,757**	k for F_V
	$EQ_{H\ E\text{-}W} =$	**0** k	& $EQ_{H\ N\text{-}S} =$ 0 k →	**0**	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 $\phi = 0$	=	30.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	18.40	Eq 3.6
$N_\gamma = 2\,(N_q + 1)\ \tan(\phi)$	=	22.40	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.27	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.26	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.82	"
For $D_f/B \leq 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\ D_f/B$	=	1.03	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	1.03	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	N/A	Eq 3.27

No inclined loads; therefore, $i_c = i_q = i_\gamma = 1.0$.

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**29,216**	**psf =**	**0**	**+**	**7,148**	**+**	**22,068**
q_{all} =	**9,730**	**psf = q_{ult} / FS**					
q_{actual} =	**1,869**	**psf = $(F_{V\ Static} + EQ_V) / (B' \times L')$**					
FS_{actual} =	**15.63**	**$= q_{ult} / q_{actual}$**			**> 3**		**Hence OK**

Table 2.6-6 presents a summary of the results of the bearing capacity analyses for the static load cases. As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 4 ksf. However, loading the storage pads to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and $c = 2.2$ ksf, as measured in the UU tests that are reported in Attachment 2 of Appendix 2A of the SAR, to model the end of construction. Using the estimated effective-stress strength of $\phi = 30°$ and $c = 0$ results in higher allowable bearing pressures. As shown in Table 2.6-6, the gross allowable bearing capacities of the cask storage pads for static loads for this soil strength is greater than 9 ksf.

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS

Dynamic bearing capacity analyses are performed using two different sets of dynamic forces. In the first set of analyses, the dynamic loads are determined as the inertial forces applicable for the peak ground accelerations from the design basis ground motion. The second set of analyses use the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 2001), for the pad supporting 2 casks, 4 casks, and 8 casks.

BASED ON INERTIAL FORCES

This section presents the analysis of the allowable bearing capacity of the pad for supporting the dynamic loads defined as the inertial forces applicable for the peak ground accelerations from the design basis ground motion. The total vertical force includes the static weight of the pad and eight fully loaded casks ± the vertical inertial forces due to the earthquake. The vertical inertial force is calculated as a_V x [weight of the pad + cask dead loads], multiplied by the appropriate factor (±40% or ±100%) for the load case. In these analyses, the minus sign for the percent loading in the vertical direction signifies uplift forces, which tend to unload the pad. Similarly, the horizontal inertial forces are calculated as a_H x [weight of the pad + cask dead loads], multiplied by the appropriate factor (40% or 100%) for the load case. The horizontal inertial force from the casks was confirmed to be less than the maximum force that can be transmitted from the cask to the pad through friction for each of these load cases. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad considered in the HI-STORM cask stability analysis (μ = 0.8, as shown in SAR Section 8.2.1.2, Accident Analysis) x the normal force acting between the casks and the pad.

The lower-bound friction case (discussed in SAR Section 4.2.3.5.1B), wherein μ between the steel bottom of the cask and the top of the concrete storage pad = 0.2, results in lower horizontal forces being applied at the top of the pad. This decreases the inclination of the load applied to the pad, which results in increased bearing capacity. Therefore, the dynamic bearing capacity analyses are not performed for μ = 0.2.

Table 2.6-7 presents the results of the bearing capacity analyses for the following cases, which include static loads plus inertial forces due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the undrained strength that was measured in unconsolidated-undrained triaxial tests (ϕ = 0° and c = 2.2 ksf).

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
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.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction

Case II: ***100% N-S, 0% Vertical, 100% E-W***

Determine forces and moments due to earthquake.

$$F_v = \overset{Wc}{2{,}852\ K} + \overset{Wp}{904.5\ K} = 3{,}757\ K \text{ and } EQ_v = 0 \text{ for this case.}$$

$$EQ_{H\ Pad} = \overset{a_H}{0.711} \times \overset{HT_{pad}}{3'} \times \overset{B}{30'} \times \overset{L}{67'} \times \overset{\gamma_{conc}}{0.15\ kcf} = 643\ K$$

$$EQhc = \text{Minimum of } [\underset{2{,}028\ K}{\overset{a_H}{0.711} \times \overset{Wc}{2{,}852\ K}} \ \&\ \underset{2{,}282K}{\overset{\mu}{0.8} \times \overset{Nc}{2{,}852\ K}}] \quad \Rightarrow \quad EQhc = 2{,}028\ K$$

Note, Nc = Wc in this case, since $a_V = 0$.

$$EQ_{H\ N\text{-}S} = \overset{EQhp}{643\ K} + \overset{EQhc}{2{,}028\ K} = 2{,}671\ K$$

The horizontal components are the same for this case; therefore, $EQ_{H\ E\text{-}W} = EQ_{H\ N\text{-}S}$

Combine these horizontal components to calculate F_H:

$$\Rightarrow \quad F_H = \sqrt{EQ^2{}_{H\ E\text{-}W} + EQ^2{}_{H\ N\text{-}S}} = \sqrt{2{,}671^2 + 2{,}671^2} = 3{,}777\ K$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb.

$$\Delta b = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 2{,}028\ K}{2{,}852\ K + 0} = 6.99\ ft$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{a_H}{0.711} \times \overset{Wp}{904.5\ K} + 3' \times \overset{EQhc}{2{,}028\ K} + \overset{\Delta b}{6.99'} \times (\overset{Wc}{2{,}852K} + \overset{EQvc}{0})$$
$$= 965\ ft\text{-}K + 6{,}084\ ft\text{-}K + 19{,}935\ ft\text{-}K = 26{,}984\ ft\text{-}K$$

The horizontal forces are the same N-S and E-W for this case; therefore,

$$\Sigma M_{@E\text{-}W} = \Sigma M_{@N\text{-}S} = 26{,}984\ ft\text{-}K$$

See Table 2.6-7 for definition and calculation of B' and L' for these forces and moments.

Determine $q_{allowable}$ for FS = 1.1.

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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)	Footing Dimensions:		
	$\phi =$	**0.0** Friction Angle (degrees)	B = **30.0**	Width - ft (E-W)	
	$\gamma =$	**80** Unit weight of soil (pcf)	L = **67.0**	Length - ft (N-S)	
	$\gamma_{surch} =$	**100** Unit weight of surcharge (pcf)			
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 $g = a_H$	
	$FS =$	**1.1** Factor of Safety required for $q_{allowable}$		**0.695** $g = a_V$	
	$F_{V\ Static} =$	**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 $D_f/B \leq 1$:	$d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	= 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**	

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Case IIIA: 40% N-S, -100% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$$EQ_V = -100\% \times \overset{a_V}{0.695} \times (\overset{Wp}{904.5\ K} + \overset{Wc}{2{,}852\ K}) = -2{,}611\ K$$

$$EQhp = \overset{a_H}{0.711} \times \overset{Wc}{904.5\ K} = 643\ K$$

Normal force at base of the cask = Cask DL = 2,852 K

— Cask EQvc = -1. x 0.695 x 2,852 K = - 1,982 K = a_V x Wc

⇒ Nc = 870 K

⇒ $F_{EQ\ \mu=0.8} = 0.8 \times 870\ K = 696\ K$

$$EQhc = \text{Minimum of } [\underbrace{\overset{a_H}{0.711} \times \overset{Wc}{2{,}852\ K}}_{2{,}028\ K}\ \&\ \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{870\ K}}_{696\ K}]$$

Note: Use only 40% of the horizontal earthquake forces in this case. 40% of 2,028 K = 811 K, which is > 696 K (= $F_{EQ\ \mu=0.8}$); therefore, EQhc is limited to the friction force at the base of the casks, which = 696 K in the direction of the resultant of both the N-S and E-W components of EQhc. For this case, the N-S and E-W components of EQhc are the same, and they are calculated as follows:

$$EQ^2_{hc\ E\text{-}W} + EQ^2_{hc\ N\text{-}S} = EQ^2_{hc} = 696^2 \Rightarrow EQ_{hc\ E\text{-}W} = EQ_{hc\ N\text{-}S} = \sqrt{\frac{696^2}{2}} = 492.1\ K$$

$$\Rightarrow EQ_{H\ N\text{-}S} = \overset{40\%\ of\ EQhp}{0.4 \times 643\ K} + \overset{EQhc_{N\text{-}S}}{492.1\ K} = 749.3\ K$$

Since horizontal components are the same for this case, $EQ_{H\ E\text{-}W} = EQ_{H\ N\text{-}S}$

$$\Rightarrow F_H = \sqrt{EQ^2_{H\ E\text{-}W} + EQ^2_{H\ N\text{-}S}} = \sqrt{749.3^2 + 749.3^2} = 1{,}060\ K$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb. Note: EQvc = -1. x 0.695 x 2,852 K = -1,982 K

$$\Delta b_{E\text{-}W} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 492.1\ K}{2{,}852\ K - 1{,}982\ K} = 5.56\ ft$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{40\%\ a_H}{0.4 \times 0.711} \times \overset{Wp}{904.5\ K} + 3' \times \overset{Eqhc_{E\text{-}W}}{492.1\ K} + \overset{\Delta b}{5.56'} \times (\overset{Wc}{2{,}852K} - \overset{EQvc}{1{,}982\ K})$$

$$= 386\ ft\text{-}K + 1{,}476\ ft\text{-}K + 4{,}837\ ft\text{-}K = 6{,}699\ ft\text{-}K$$

The horizontal forces are the same N-S and E-W for this case; therefore,

$$\Sigma M_{@E\text{-}W} = \Sigma M_{@N\text{-}S} = 6{,}699\ ft\text{-}K$$

Determine $q_{allowable}$ for FS = 1.1.

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 IIIA

Based on Inertial Forces Combined: 40 % N-S, -100 % Vert, 40 % 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:

$B' =$ **18.3** Effective Ftg Width - ft (E-W) $L' =$ **55.3** 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 & $EQ_V =$ **-2,611** k → **1,146** k for F_V
$EQ_{H\ E\text{-}W} =$ **749** k & $EQ_{H\ N\text{-}S} =$ **749** k → **1,060** 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	"
$s_\gamma = 1 - 0.4\ (B/L)$	=	0.87	"
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 $\phi = 0$: $d_c = 1 + 0.4\ (D_f/B)$	=	1.07	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 $\phi = 0$: $i_c = 1 - (m\ F_H / B'\ L'\ c\ N_c)$	=	0.86	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**11,344**	**psf =**	**11,044**	+	**300**	+	**0**
q_{all} =	**10,310**	**psf = q_{ult} / FS**					
q_{actual} =	**1,132**	**psf = $(F_{v\ Static} + EQ_v) / (B' \times L')$**					
FS_{actual} =	**10.02**	**= q_{ult} / q_{actual}**					

> 1.1 Hence OK

CALCULATION IDENTIFICATION NUMBER				PAGE 62
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

Dynamic Bearing Capacity of the Cask Storage Pads Based on Inertial Forces

Case IIIB: 40% N-S, -40% Vertical, 100% E-W

Determine forces and moments due to earthquake.

$$EQ_V = -40\% \times \overset{a_V}{0.695} \times (\overset{Wp}{904.5\ K} + \overset{Wc}{2{,}852\ K}) = -1{,}044\ K$$

Normal force at base of the cask = Cask DL = 2,852 K

— 40% of Cask EQvc = -0.4 x 0.695 x 2,852 K = - 793 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 2,059 K

$\Rightarrow$ $F_{EQ\ \mu=0.8}$ = 0.8 x 2,059 K = 1,647 K

$$EQhc = \text{Min of } [\overset{a_H}{0.711} \times \overset{Wc}{2{,}852\ K} \ \& \ \overset{\mu}{0.8} \times \overset{Nc}{2{,}059\ K}] \Rightarrow EQhc = 1{,}647\ K;$$

2,028 K 1,647K

i.e., EQhc is limited to the friction force at the base of the casks, which = 1,647 K in the direction of the resultant of both the N-S and E-W components of EQhc. For this case, the N-S component of EQhc = 0.4 x 2,028 K = 811 K, and the E-W component is calculated as follows:

$$EQ^2_{hc\ E\text{-}W} + EQ^2_{hc\ N\text{-}S} = EQ^2_{hc} = 1{,}647^2 \Rightarrow EQ_{hc\ E\text{-}W} = \sqrt{1{,}647^2 - 811^2} = 1{,}433.5\ K$$

Using 40% of N-S:

$$\Rightarrow EQ_{H\ N\text{-}S} = \overset{\text{40\% of EQhp}}{0.4 \times 643\ K} + \overset{Eqhc_{N\text{-}S}}{811\ K} = 1{,}068\ K$$

Using 100% of E-W:

$$\Rightarrow EQ_{H\ E\text{-}W} = \overset{\text{100\% of EQhp}}{1.0 \times 643\ K} + \overset{Eqhc_{E\text{-}W}}{1{,}433.5\ K} = 2{,}076.5\ K$$

$$\Rightarrow F_H = \sqrt{EQ^2_{H\ E\text{-}W} + EQ^2_{H\ N\text{-}S}} = \sqrt{2{,}076.5^2 + 1{,}068^2} = 2{,}335\ K$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb. Note: EQvc = -0.4 x 0.695 x 2,852 K = -793 K

$$\Delta b_{E\text{-}W} = \frac{9.83' \times EQhc_{E\text{-}W}}{Wc + EQvc} = \frac{9.83' \times 1{,}433.5K}{2{,}852K - 793K} = 6.84\ ft$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{\text{100\% } a_H}{0.711} \times \overset{Wp}{904.5\ K} + 3' \times \overset{Eqhc_{E\text{-}W}}{1{,}433.5\ K} + \overset{\Delta b}{6.84'} \times (\overset{Wc}{2{,}852K} - \overset{EQvc}{793\ K})$$

$$= 965\ ft\text{-}K + 4{,}300\ ft\text{-}K + 14{,}084\ ft\text{-}K = 19{,}349\ ft\text{-}K$$

$$\Delta b_{N\text{-}S} = \frac{9.83' \times EQhc_{N\text{-}S}}{Wc + EQvc} = \frac{9.83' \times 811K}{2{,}852K - 793K} = 3.87\ ft$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \overset{\text{40\% } a_H}{0.4 \times 0.711} \times \overset{Wp}{904.5\ K} + 3' \times \overset{Eqhc_{N\text{-}S}}{811\ K} + \overset{\Delta b}{3.87'} \times (\overset{Wc}{2{,}852K} - \overset{EQvc}{793\ K})$$

$$= 386\ ft\text{-}K + 2{,}434\ ft\text{-}K + 7{,}969\ ft\text{-}K = 10{,}787\ ft\text{-}K$$

Determine $q_{allowable}$ for FS = 1.1.

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case IIIB

Based on Inertial Forces Combined:
40 % N-S, -40 % 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:

$B' =$ **15.7** Effective Ftg Width - ft (E-W) $L' =$ **59.0** Length - ft (N-S)

$D_f =$ **3.0** Depth of Footing (ft)

0.711 $g = a_H$

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$ **0.695** $g = a_V$

$F_{V\ Static} =$ **3,757** k & $EQ_V =$ **-1,044** k → **2,712** k for F_V

$EQ_{H\ E\text{-}W} =$ **2,077** k & $EQ_{H\ N\text{-}S} =$ **1,068** k → **2,336** 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)

Equation		Value	Reference
$N_c = (N_q - 1)\cot(\phi)$, 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.05	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.89	"
For $D_f/B \leq 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	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 $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	1.10 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.61	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 $\phi = 0$: $i_c = 1 - (m F_H / B'L'c N_c)$	=	0.64	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**8,513**	**psf =**	**8,213**	**+**	**300**	**+**	**0**
q_{all} =	**7,730**	**psf = q_{ult} / FS**					
q_{actual} =	**2,922**	**psf = $(F_{v\ Static} + EQ_v)/(B' \times L')$**					
FS_{actual} =	**2.91**	**= q_{ult}/q_{actual}**			**> 1.1**	**Hence OK**	

Case IIIC: 100% N-S, -40% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$$EQ_V = -40\% \times \overset{a_v}{0.695} \times (\overset{Wp}{904.5\text{ K}} + \overset{Wc}{2{,}852\text{ K}}) = -1{,}044\text{ K}$$

Normal force at base of the cask = Cask DL = 2,852 K

— 40% of Cask EQvc = -0.4 x 0.695 x 2,852 K = - 793 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 2,059 K

$$\Rightarrow \quad F_{EQ\ \mu=0.8} = 0.8 \times 2{,}059\text{ K} = 1{,}647\text{ K}$$

$$EQhc = \text{Min of } [\underbrace{\overset{a_H}{0.711} \times \overset{Wc}{2{,}852\text{ K}}}_{2{,}028\text{ K}} \;\&\; \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{2{,}059\text{ K}}}_{1{,}647\text{K}}] \Rightarrow EQhc = 1{,}647\text{ K};$$

i.e., EQhc is limited to the friction force at the base of the casks, which = 1,647 K in the direction of the resultant of both the N-S and E-W components of EQhc. For this case, the E-W component of EQhc = 0.4 x 2,028 K = 811 K, and the N-S component is calculated as follows:

$$EQ^2_{hc\ N\text{-}S} + EQ^2_{hc\ E\text{-}W} = EQ^2_{hc} = 1{,}647^2 \quad \Rightarrow \quad EQ_{hc\ N\text{-}S} = \sqrt{1{,}647^2 - 811^2} = 1{,}433.5\text{ K}$$

Using 100% of N-S:

$$\Rightarrow \quad EQ_{H\ N\text{-}S} = \overset{100\%\text{ of }EQhp}{1.0 \times 643\text{ K}} + \overset{Eqhc_{N\text{-}S}}{1{,}433.5\text{ K}} = 2{,}076\text{ K}$$

Using 40% of E-W:

$$\Rightarrow \quad EQ_{H\ E\text{-}W} = \overset{40\%\text{ of }EQhp}{0.4 \times 643\text{ K}} + \overset{Eqhc_{E\text{-}W}}{811\text{ K}} = 1{,}068\text{ K}$$

$$\Rightarrow \quad F_H = \sqrt{EQ^2_{H\ E\text{-}W} + EQ^2_{H\ N\text{-}S}} = \sqrt{1{,}068^2 + 2{,}076^2} = 2{,}335\text{ K}$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb. Note: EQvc = -0.4 x 0.695 x 2,852 K = -793 K

$$\Delta b_{E\text{-}W} = \frac{9.83' \times EQhc_{E\text{-}W}}{Wc + EQvc} = \frac{9.83' \times 811\text{K}}{2{,}852\text{K} - 793\text{K}} = 3.87\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{40\%\ a_H}{0.4 \times 0.711} \times \overset{Wp}{904.5\text{ K}} + 3' \times \overset{Eqhc_{E\text{-}W}}{811\text{ K}} + \overset{\Delta b}{3.87'} \times (\overset{Wc}{2{,}852\text{K}} - \overset{EQvc}{793\text{ K}})$$

$$= 386\text{ ft-K} + 2{,}434\text{ ft-K} + 7{,}969\text{ ft-K} = 10{,}787\text{ ft-K}$$

$$\Delta b_{N\text{-}S} = \frac{9.83' \times EQhc_{N\text{-}S}}{Wc + EQvc} = \frac{9.83' \times 1{,}433.5\text{K}}{2{,}852\text{K} - 793\text{K}} = 6.84\text{ ft}$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \overset{100\%\ a_H}{0.711} \times \overset{Wp}{904.5\text{ K}} + 3' \times \overset{Eqhc_{N\text{-}S}}{1{,}433.5\text{ K}} + \overset{\Delta b}{6.84'} \times (\overset{Wc}{2{,}852\text{K}} - \overset{EQvc}{793\text{ K}})$$

$$= 965\text{ ft-K} + 4{,}300\text{ ft-K} + 14{,}084\text{ ft-K} = 19{,}349\text{ ft-K}$$

Determine $q_{allowable}$ for FS = 1.1.

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 IIIC

Based on Inertial Forces Combined:
100 % N-S, -40 % Vert, 40 % 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:

$B' =$ **22.0** Effective Ftg Width - ft (E-W) $L' =$ **52.7** 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 & $EQ_V =$ **-1,044** k → **2,712** k for F_V
$EQ_{H\ E\text{-}W} =$ **1,068** k & $EQ_{H\ N\text{-}S} =$ **2,077** k → **2,336** 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
	$s_c = 1 + (B/L)(N_q/N_c)$	= 1.08	Table 3.2
	$s_q = 1 + (B/L) \tan\phi$	= 1.00	"
	$s_\gamma = 1 - 0.4\ (B/L)$	= 0.83	"
For $D_f/B \leq 1$:	$d_q = 1 + 2 \tan\phi\ (1 - \sin\phi)^2\ D_f/B$	= 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 $\phi = 0$:	$d_c = 1 + 0.4\ (D_f/B)$	= 1.05	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 $EQ_{H\ N\text{-}S} > 0$:	$\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	= 0.48 rad	
	$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	= 1.39	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 $\phi = 0$:	$i_c = 1 - (m\ F_H / B'\ L'\ c\ N_c)$	= 0.75	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross $q_{ult} =$	10,010	psf =	9,710	+	300	+	0
$q_{all} =$	9,100	psf = q_{ult} / FS					
$q_{actual} =$	2,334	psf = $(F_{v\ Static} + EQ_v) / (B' \times L')$					
$FS_{actual} =$	4.29	= q_{ult} / q_{actual}			> 1.1 Hence OK		

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

Case IVA: 40% N-S, 100% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$$EQ_V = 100\% \times \underset{a_V}{0.695} \times (\underset{Wp}{904.5\text{ K}} + \underset{Wc}{2{,}852\text{ K}}) = 2{,}611\text{ K}$$

$$EQhp = \underset{a_H}{0.711} \times \underset{Wc}{904.5\text{ K}} = 643\text{ K}$$

Normal force at base of the cask = Cask DL = 2,852 K

+ Cask EQvc = 1. x 0.695 x 2,852 K = + 1,982 K = a_V x Wc

$\Rightarrow$ Nc = 4,834 K

$\Rightarrow$ $F_{EQ\ \mu=0.8}$ = 0.8 x 4,834 K = 3,867 K

$$EQhc = \text{Min of } [\underset{a_H}{0.711} \times \underset{Wc}{2{,}852\text{ K}} \ \&\ \underset{\mu}{0.8} \times \underset{Nc}{4{,}834\text{ K}}]$$

2,028 K 3,867K

Note: Use only 40% of the horizontal earthquake forces in this case. 40% of 2,028 K = 811 K, which is < 3,867 K (= $F_{EQ\ \mu=0.8}$); therefore, EQhc = 811 K in both the N-S and E-W directions for this case.

$$\Rightarrow \quad EQ_{H\ N\text{-}S} = \underset{40\%\text{ of }EQhp}{0.4 \times 643\text{ K}} + \underset{Eqhc_{N\text{-}S}}{811\text{ K}} = 1{,}068\text{ K}$$

Since horizontal components are the same for this case, $EQ_{H\ E\text{-}W} = EQ_{H\ N\text{-}S}$

$$\Rightarrow \quad F_H = \sqrt{EQ^2_{H\ E\text{-}W} + EQ^2_{H\ N\text{-}S}} = \sqrt{1{,}068^2 + 1{,}068^2} = 1{,}510\text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb. Note: EQvc = 1.0 x 0.695 x 2,852 K = 1,982 K

$$\Delta b_{E\text{-}W} = \frac{9.83' \times EQhc_{E\text{-}W}}{Wc + EQvc} = \frac{9.83' \times 811\text{K}}{2{,}852\text{K} + 1{,}982\text{K}} = 1.65\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \underset{40\%\ a_H}{0.4 \times 0.711} \times \underset{Wp}{904.5\text{K}} + 3' \times \underset{Eqhc_{E\text{-}W}}{811\text{ K}} + \underset{\Delta b}{1.65'} \times (\underset{Wc}{2{,}852\text{K}} + \underset{EQvc}{1{,}982\text{ K}})$$

$$= 386\text{ ft-K} + 2{,}433\text{ ft-K} + 7{,}976\text{ ft-K} = 10{,}795\text{ ft-K}$$

The horizontal forces are the same N-S and E-W for this case; therefore,

$$\Sigma M_{@E\text{-}W} = \Sigma M_{@N\text{-}S} = 10{,}795\text{ ft-K}$$

Determine $q_{allowable}$ for FS = 1.1.

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 IVA

Based on Inertial Forces Combined:
40 % N-S, 100 % Vert, 40 % 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:

$B' =$ **26.6** Effective Ftg Width - ft (E-W) $L' =$ **63.6** Length - ft (N-S)

$D_f =$ **3.0** Depth of Footing (ft)

0.711 $g = a_H$

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$ **0.695** $g = a_V$

$F_{V\,Static} =$ **3,757** k & $EQ_V =$ **2,611** k → **6,368** k for F_V

$EQ_{H\,E\text{-}W} =$ **1,068** k & $EQ_{H\,N\text{-}S} =$ **1,068** k → **1,511** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.08	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.83	"
For $D_f/B \leq 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	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 $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.88	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**11,567**	**psf =**	**11,267**	+	**300**	+	**0**
q_{all} =	**10,510**	**psf = q_{ult} / FS**					
q_{actual} =	**3,762**	**psf = $(F_{v\,Static} + EQ_v)/(B' \times L')$**					
FS_{actual} =	**3.07**	**= q_{ult}/q_{actual}**					

> 1.1 Hence OK

[geot]\05996\calc\brng_cap\Pad\Wint_Fang-8.xls

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

Case IVB: 40% N-S, 40% Vertical, 100% E-W

Determine forces and moments due to earthquake.

$$EQ_V = 0.4 \times \underset{a_v}{0.695} \times (\underset{Wp}{904{,}5\ K} + \underset{Wc}{2{,}852\ K}) = 1{,}044\ K$$

Normal force at base of the cask = Cask DL = 2,852 K

\+ 40% of Cask EQvc = +0.4 x 0.695 x 2,852 K = + 793 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 3,645 K

$\Rightarrow$ $F_{EQ\ \mu=0.8} = 0.8 \times 3{,}645\ K = 2{,}916\ K$

$$EQhc = \text{Min of } [\underset{2{,}028\ K}{\underset{a_H}{0.711} \times \underset{Wc}{2{,}852\ K}} \ \&\ \underset{2{,}916K}{\underset{\mu}{0.8} \times \underset{Nc}{3{,}645\ K}}] \Rightarrow EQhc = 2{,}028\ K, \text{ since it is} < F_{EQ\ \mu=0.8}$$

The horizontal inertial force of the casks acting on the pad is less than the friction force at the base of the casks. Applying 40% in the N-S direction, $Eqhc_{N-S}$ = 0.4 x 2,028 K = 811 K and 100% in the E-W direction, $Eqhc_{E-W}$ = 2,028 K for this case.

Using 40% of N-S:

$$\Rightarrow EQ_{H\ N-S} = \underset{40\%\text{ of }EQhp}{0.4 \times 643\ K} + \underset{Eqhc_{N-S}}{811\ K} = 1{,}068\ K$$

Using 100% of E-W:

$$\Rightarrow EQ_{H\ E-W} = \underset{100\%\text{ of }EQhp}{1.0 \times 643\ K} + \underset{Eqhc_{E-W}}{2{,}028\ K} = 2{,}671\ K$$

$$\Rightarrow F_H = \sqrt{EQ^2{}_{H\ E-W} + EQ^2{}_{H\ N-S}} = \sqrt{2{,}671^2 + 1{,}068^2} = 2{,}877\ K$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb. Note: EQvc = 0.4 x 0.695 x 2,852 K = 793 K

$$\Delta b_{E-W} = \frac{9.83' \times EQhc_{E-W}}{Wc + EQvc} = \frac{9.83' \times 2{,}028\,K}{2{,}852\,K + 793\,K} = 5.47\ ft$$

$$\Sigma M_{@N-S} = 1.5' \times \underset{100\%\ a_H}{0.711} \times \underset{Wp}{904.5\ K} + 3' \times \underset{Eqhc_{E-W}}{2{,}028\ K} + \underset{\Delta b}{5.47'} \times (\underset{Wc}{2{,}852K} + \underset{EQvc}{793\ K})$$

$$= 965\ ft\text{-}K + 6{,}084\ ft\text{-}K + 19{,}938\ ft\text{-}K = 26{,}987\ ft\text{-}K$$

$$\Delta b_{N-S} = \frac{9.83' \times EQhc_{N-S}}{Wc + EQvc} = \frac{9.83' \times 811\,K}{2{,}852\,K + 793\,K} = 2.19\ ft$$

$$\Sigma M_{@E-W} = 1.5' \times \underset{40\%\ a_H}{0.4 \times 0.711} \times \underset{Wp}{904.5\ K} + 3' \times \underset{Eqhc_{N-S}}{811\ K} + \underset{\Delta b}{2.19'} \times (\underset{Wc}{2{,}852K} + \underset{EQvc}{793\ K})$$

$$= 386\ ft\text{-}K + 2{,}433\ ft\text{-}K + 7{,}982\ ft\text{-}K = 10{,}801\ ft\text{-}K$$

Determine $q_{allowable}$ for FS = 1.1.

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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 IVB

Based on Inertial Forces Combined: 40 % N-S, 40 % Vert, 100 % E-W

Soil Properties:

$c =$	**2,200**	Cohesion (psf)	Footing Dimensions:		
$\phi =$	**0.0**	Friction Angle (degrees)	B =	**30.0**	Width - ft (E-W)
$\gamma =$	**80**	Unit weight of soil (pcf)	L =	**67.0**	Length - ft (N-S)
$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)			

Foundation Properties:

$B' =$	**18.8**	Effective Ftg Width - ft (E-W)	L' =	**62.5**	Length - ft (N-S)
$D_f =$	**3.0**	Depth of Footing (ft)			
				0.711	$g = a_H$
FS =	**1.1**	Factor of Safety required for $q_{allowable}$		**0.695**	$g = a_V$

$F_{V\ Static} =$ **3,757** k & $EQ_V =$ **1,044** k → **4,801** k for F_V

$EQ_{H\ E\text{-}W} =$ **2,671** k & $EQ_{H\ N\text{-}S} =$ **1,068** k → **2,877** k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, 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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.06	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 $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	1.19 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.64	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 $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.64	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**8,508**	**psf =**	**8,208**	+	**300**	+	**0**
q_{all} =	**7,730**	**psf = q_{ult} / FS**					
q_{actual} =	**4,095**	**psf = $(F_{V\ Static} + EQ_v)/(B' \times L')$**					
FS_{actual} =	**2.08**	**= q_{ult}/q_{actual}**					

> 1.1 Hence OK

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

Case IVC: 100% N-S, 40% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$$EQ_V = 0.4 \times \overset{a_V}{0.695} \times (\overset{W_p}{904.5\text{ K}} + \overset{W_c}{2{,}852\text{ K}}) = 1{,}044\text{ K}$$

Normal force at base of the cask = Cask DL = 2,852 K

+ 40% of Cask EQvc = 0.4 x 0.695 x 2,852 K = + 793 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 3,645 K

$\Rightarrow$ $F_{EQ\ \mu=0.8} = 0.8 \times 3{,}645\text{ K} = 2{,}916\text{ K}$

$$EQhc = \text{Min of } [\underbrace{\overset{a_H}{0.711} \times \overset{W_c}{2{,}852\text{ K}}}_{2{,}028\text{ K}} \ \&\ \underbrace{\overset{\mu}{0.8} \times \overset{N_c}{3{,}645\text{ K}}}_{2{,}916\text{ K}}] \Rightarrow EQhc = 2{,}028\text{ K, since it is} < F_{EQ\ \mu=0.8}$$

The horizontal inertial force of the casks acting on the pad is less than the friction force at the base of the casks. Applying 100% in the N-S direction, $Eqhc_{N-S}$ = 2,028 K and 40% in the E-W direction, $Eqhc_{E-W}$ = 0.4 x 2,028 K = 811 K for this case.

Using 100% of N-S:

$$\Rightarrow \quad EQ_{H\ N-S} = \overset{100\%\text{ of }EQhp}{1.0 \times 643\text{ K}} + \overset{Eqhc_{N-S}}{2{,}028\text{ K}} = 2{,}671\text{ K}$$

Using 40% of E-W:

$$\Rightarrow \quad EQ_{H\ E-W} = \overset{40\%\text{ of }EQhp}{0.4 \times 643\text{ K}} + \overset{Eqhc_{E-W}}{811\text{ K}} = 1{,}068\text{ K}$$

$$\Rightarrow \quad F_H = \sqrt{EQ^2_{H\ E-W} + EQ^2_{H\ N-S}} = \sqrt{1{,}068^2 + 2{,}671^2} = 2{,}877\text{ K}$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb. Note: EQvc = 0.4 x 0.695 x 2,852 K = 793 K

$$\Delta b_{E-W} = \frac{9.83' \times EQhc_{E-W}}{Wc + EQvc} = \frac{9.83' \times 811\text{K}}{2{,}852\text{K} + 793\text{K}} = 2.19\text{ ft}$$

$$\Sigma M_{@N-S} = 1.5' \times \overset{40\%\ a_H}{0.4 \times 0.711} \times \overset{W_p}{904.5\text{ K}} + 3' \times \overset{EQhc}{811\text{ K}} + \overset{\Delta b}{2.19'} \times (\overset{W_c}{2{,}852\text{K}} + \overset{EQvc}{793\text{ K}})$$

$$= 386\text{ ft-K} + 2{,}433\text{ ft-K} + 7{,}982\text{ ft-K} = 10{,}801\text{ ft-K}$$

$$\Delta b_{N-S} = \frac{9.83' \times EQhc_{N-S}}{Wc + EQvc} = \frac{9.83' \times 2{,}028\text{K}}{2{,}852\text{K} + 793\text{K}} = 5.47\text{ ft}$$

$$\Sigma M_{@E-W} = 1.5' \times \overset{100\%\ a_H}{0.711} \times \overset{W_p}{904.5\text{ K}} + 3' \times \overset{EQhc}{2{,}028\text{ K}} + \overset{\Delta b}{5.47'} \times (\overset{W_c}{2{,}852\text{K}} + \overset{EQvc}{793\text{ K}})$$

$$= 965\text{ ft-K} + 6{,}084\text{ ft-K} + 19{,}938\text{ ft-K} = 26{,}987\text{ ft-K}$$

Determine $q_{allowable}$ for FS = 1.1.

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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 IVC

Based on Inertial Forces Combined:
100 % N-S, 40 % Vert, 40 % 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:

$B' =$ **25.5** Effective Ftg Width - ft (E-W) $L' =$ **55.8** 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 & $EQ_V =$ **1,044** k → **4,801** k for F_V

$EQ_{H\ E\text{-}W} =$ **1,068** k & $EQ_{H\ N\text{-}S} =$ **2,671** k → **2,877** k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.82	"
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 $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.05	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 $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	=	0.38 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.36	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 $\phi = 0$: $i_c = 1 - (m\, F_H / B' L' c\, N_c)$	=	0.76	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	10,052	psf =	9,752	+	300	+	0
q_{all} =	9,130	psf = q_{ult} / FS					
q_{actual} =	3,376	psf = $(F_{v\ Static} + EQ_v) / (B' \times L')$					
FS_{actual} =	2.98	= q_{ult} / q_{actual}			> 1.1 Hence OK		

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As indicated in Table 2.6-7, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the inertial loads due to the design basis ground motion exceeds 4.8 ksf for all loading cases identified above. The minimum allowable value was obtained for Load Case II, wherein 100% of the earthquake loads act in the N-S and E-W directions and 0% acts in the vertical direction. The actual factor of safety for this very conservative load case was 1.2, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1). In Load Cases III and IV, the effects of the three components of the earthquake in accordance with procedures described in ASCE (1986) to account for the fact that the maximum response of the three orthogonal components of the earthquake do not occur at the same time. For these cases, 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the dynamic loading acts in the other two directions. For these load cases, the gross allowable bearing capacity of the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the inertial loads due to the design basis ground motion exceeds 6.7 and the factor of safety exceeds 2.1.

BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

The following pages determine the allowable bearing capacity for the cask storage pads with respect to the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 2001) for the pad supporting 2 casks, 4 casks, and 8 casks. These dynamic forces represent the maximum force occurring at any time during the earthquake at each node in the model used to represent the cask storage pads. It is expected that these maximum forces will not occur at the same time for every node. These forces, therefore, represent an upper bound of the dynamic forces that could act at the base of the pad.

The coordinate system used in the analyses presented on the following pages is the same as that used for the analyses discussed above, and it is shown in Figure 1. Note, this coordinate system is different than the one used in Calculation 05996.02-G(PO17)-2 (CEC, 2001), which is shown on Page B11. Therefore, in the following pages, the X direction is still N-S, the Y direction remains vertical, and the Z direction remains E-W.

These maximum dynamic cask driving forces were confirmed to be less than the maximum force that can be transmitted from the cask to the pad through friction acting at the base of the cask for each of these load cases. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad (μ = 0.8, as shown in SAR Section 8.2.1.2) x the normal force acting between the casks and the pad. These maximum dynamic cask driving forces can be transmitted to the pad through friction only when the inertial vertical forces act downward; therefore, these analyses are performed only for Load Case IV. These analyses are performed for Load Case IVA, where 40% of the horizontal forces due to the earthquake are applied in both the N-S and the E-W directions, while 100% of the vertical force is applied to obtain the maximum vertical load on the cask storage pad. The width (30 ft) is less in the E-W direction than the length N-S (67 ft); therefore, the E-W direction is the critical direction with respect to a bearing capacity failure.

NOTED ~~JUN 19 1997~~ P. J. Trudeau
1-26-00

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

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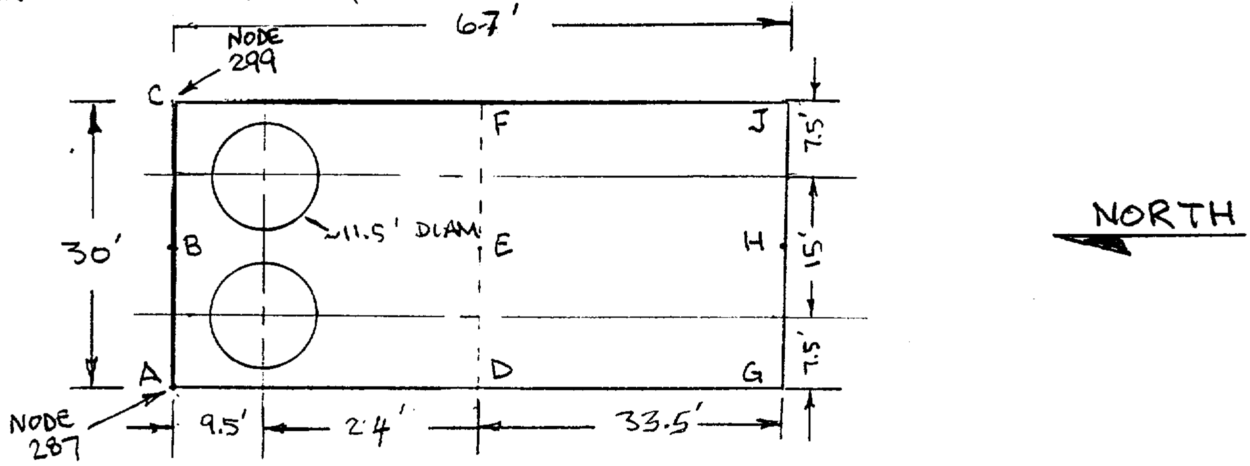
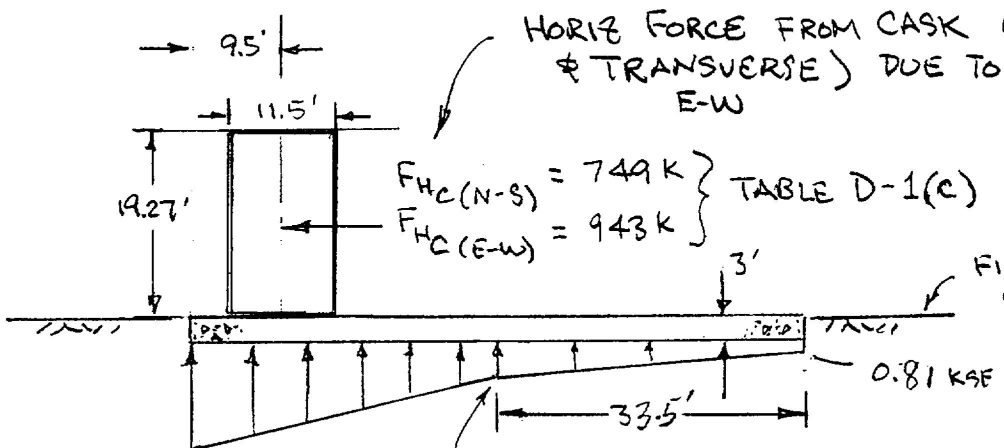
DYN BEARING CAPACITY OF PAD: 2-CASK CASE



PLAN



HORIZ FORCE FROM CASK (LONGITUDINAL N-S & TRANSVERSE E-W) DUE TO DESIGN EARTHQK*

$F_{H_C(N-S)} = 749$ K
$F_{H_C(E-W)} = 943$ K
} TABLE D-1(c) CEC (2001)

1.83 KSF (NEAR SIDE)
1.55 " (FAR SIDE)

$q_v = 6.28$ KSF NEAR SIDE
5.33 " FAR SIDE
} SEE NEXT PAGE & ATT B

ELEV

* STRESSES AT PAD/SOIL INTERFACE OBTAINED FROM CEC (2001) CALC 05996.02-G(PO17)-2, REV.3 – COPIES OF PERTINENT PAGES ARE INCLUDED IN ATT B OF THIS CALC

~~00866~~

DYN BEARING CAPACITY OF PAD: 2-CASK CASE

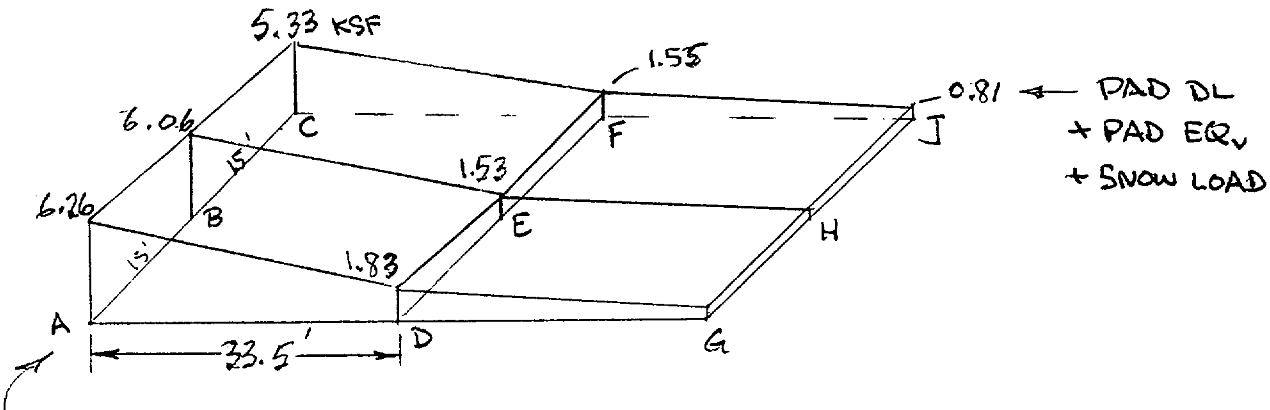
SOIL BEARING PRESSURES ARE BASED ON INFO FROM CEC(2001) INCLUDED IN ATT B AND ARE SUMMARIZED IN TABLE 1.

VERTICAL PRESSURES INCLUDE: PAD DL = 0.45 KSF
PAD EQ = 0.24 KSF
SNOW LOAD = 0.045 KSF

CASK LL ≃ 1.35 KSF ALONG LINE AC & IS ASSUMED TO DECREASE LINEARLY TO 0 ALONG LINE DF.

CASK EQ PRESSURES ARE SHOWN ON TABLE 1.

SUMMING THESE VERTICAL PRESSURES RESULTS IN THE FOLLOWING MAXIMUM TOTAL PRESSURE DISTRIBUTION. NOTE, LOADING FROM CASKS & PAD ARE ESSENTIALLY APPLIED TO ONLY ~1/2 OF THE PAD.



FOR LOADED HALF OF PAD:

$$F_v \simeq \left[\frac{15'}{2} \times \overbrace{(6.26 + 2\times 6.06 + 5.33)^{KSF}}^{177.82} + \frac{15'}{2}\overbrace{(1.83 + 2\times 1.53 + 1.55)^{KSF}}^{48.30}\right]\frac{33.5}{2}$$

$F_v \simeq 3796$ K FOR LOADED 1/2 OF PAD

$A_{AC} \sim 177.82 \frac{K}{FT} = 30' \times q_{AVG_{AC}} \Rightarrow q_{AVG_{AC}} = 5.92$ KSF

$A_{DF} \sim 48.30 \frac{K}{FT} = 30\, q_{AVG_{DF}} \Rightarrow q_{AVG_{DF}} = 1.61$ KSF

~~0086~~

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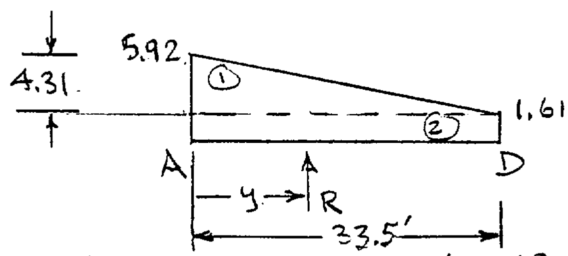
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-9	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 2-CASK CASE

DETERMINE ~ ECCENTRICITY OF F_V IN L DIRECTION (N-S) USING $q_{AVG\,AC}$ & $q_{AVG\,DF}$



NORTH

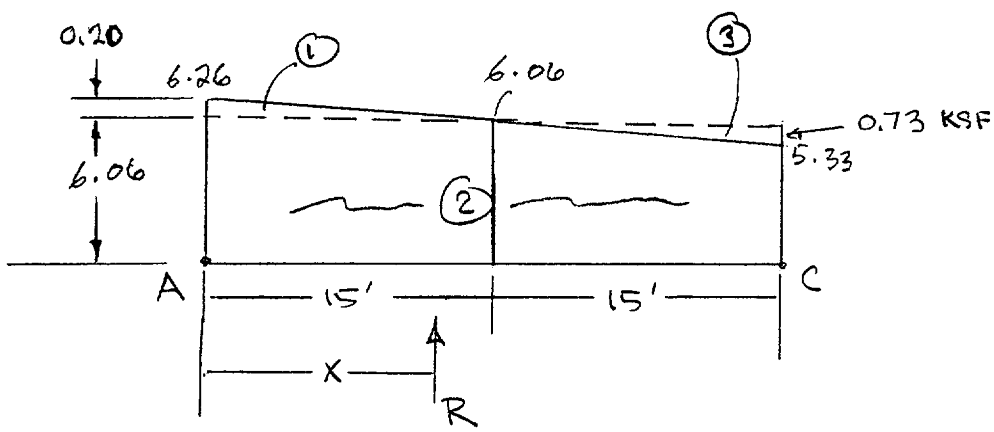
ΣM_A TO FIND LOC'N OF F_V

$$R y = \tfrac{1}{2}\, 4.31 \times 33.5 \times \tfrac{1}{3} 33.5 + 1.61 \times 33.5 \times \frac{33.5}{2}$$

$$R = \tfrac{1}{2}\, 4.31 \times 33.5 + 1.61 \times 33.5 = 72.19 + 53.94 = 126.1 \text{ K}$$

$$y = \frac{806.15 + 903.41}{126.1} \text{ K-FT} = 13.55 \text{ FT}$$

DETERMINE ECCENTRICITY OF F_V IN B DIRECTION (E-W) USING MAX SOIL PRESSURES ALONG LINE AC



WEST

~~00868~~

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CALCULATION SHEET

▲5010.65

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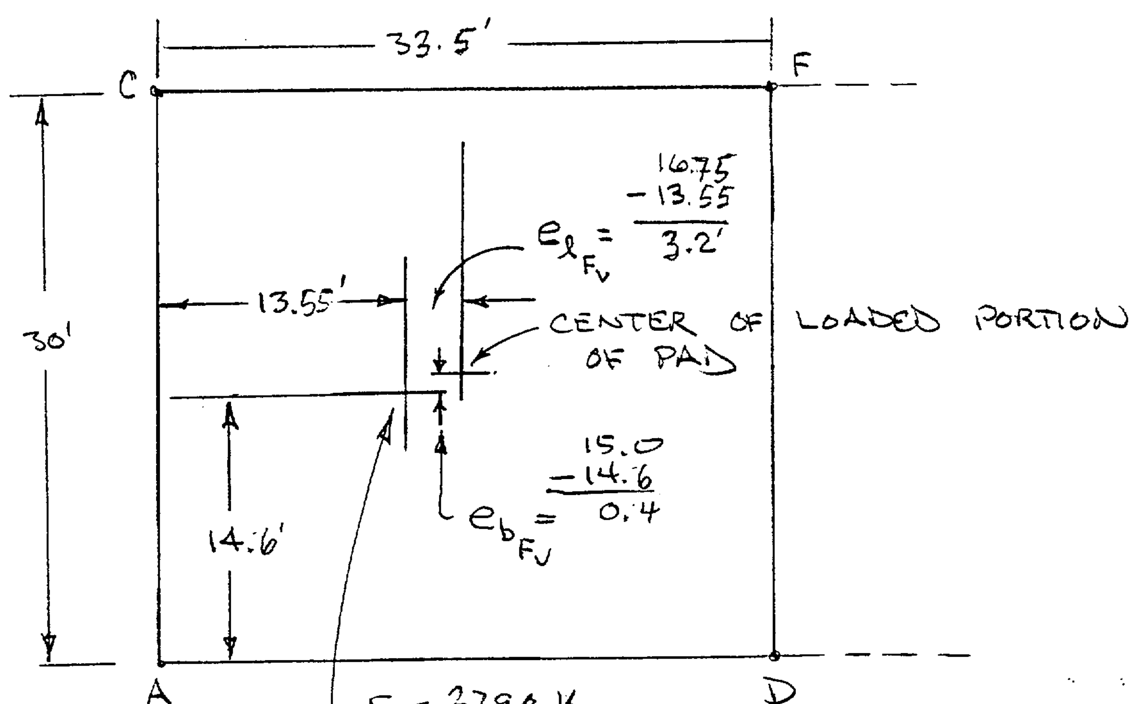
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-9	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD : 2-CASK CASE

ΣM_A

	AREA (K/FT)	MOMENT ARM (FT)	MOMENT K-FT/FT
1	$\frac{1}{2}$ 0.20 KSF × 15' = 1.5	$\frac{1}{3} \cdot 15' = 5'$	7.5
2	6.06 KSF × 30' = 181.8	$\frac{1}{2} \cdot 30' = 15'$	2,727
3	$-\frac{1}{2}$ 0.73 KSF × 15' = −5.48	$15' + \frac{2}{3} 15 = 25'$	− 136.88
	$\Sigma F_v = R = 177.8$ K/FT		2597.6

$$\therefore \quad x = \frac{\Sigma M_A}{\Sigma F_v} = \frac{2597.6 \text{ K-FT/FT}}{177.8 \text{ K/FT}} = 14.61'$$



$F_v = 3790$ K

POINT OF APPLICATION OF F_v DUE TO PAD (DL+EQ) & CASKS (LL+EQ) ~~00809~~ FOR 2-CASK CASE

10	CALCULATION IDENTIFICATION NUMBER			PAGE 78
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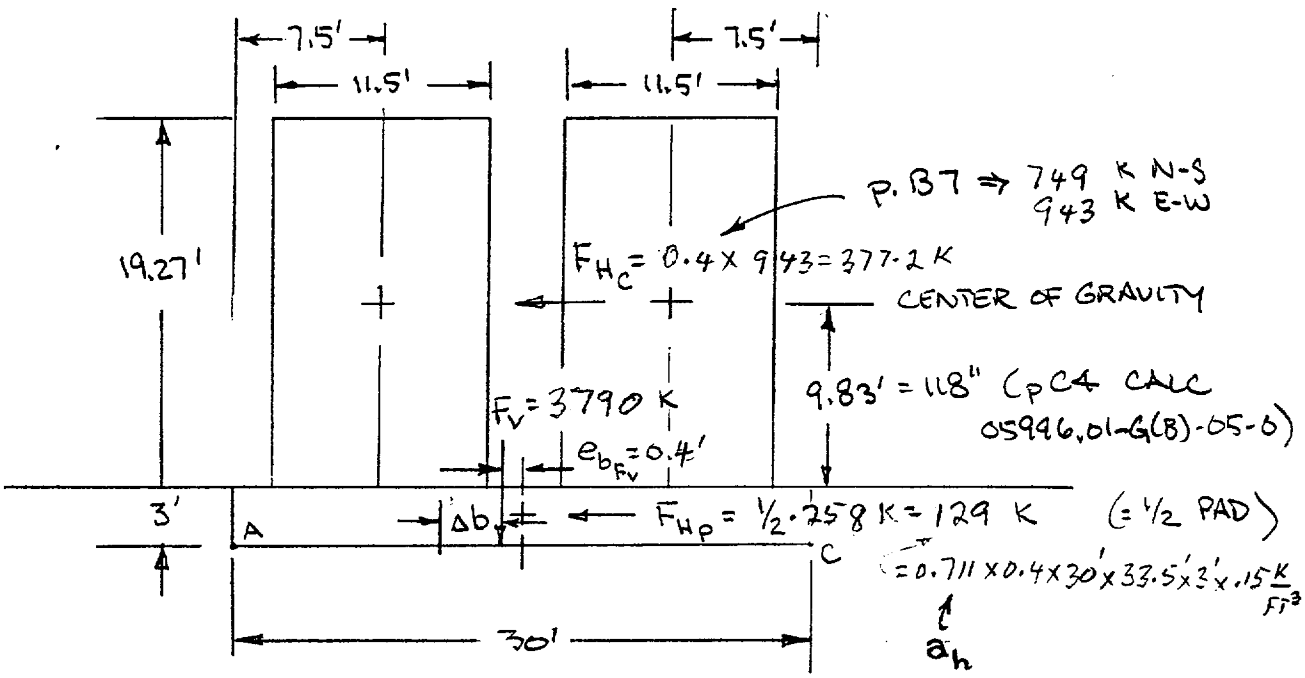
DYN BEARING CAPACITY OF PAD: 2-CASK CASE



$\Sigma M_{F_V @ \text{BOTTOM OF PAD}}$ TO FIND LOC'N OF RESULTANT TO RESIST F_H's

$$R\Delta b = 1.5' \times \underbrace{\frac{129^K}{2}}_{\text{1/2 PAD *}} + (3' + 9.83')\ \underbrace{377.2^K}_{\text{2-CASKS TRANSVERSE LOADING (E-W)}}$$

$R = F_V = 3790$ K

$$\therefore \Delta b = \frac{193.5 + 4839.5}{3790\ \text{K}}\ \text{K-FT} = 1.30\ \text{FT}$$

ADD $e_{b_{F_V}} = 0.4 \Rightarrow e_b = 1.30' + 0.4' = 1.7'$

$$B' = B - 2e_b = 30' - 2 \times 1.7 = 26.6'$$

* NOTE: HORIZ INERTIA OF OTHER ½ OF PAD $\left(\frac{322^K}{30' \times 33.5'} = 0.33\ \text{KSF}\right)$
IS RESISTED BY C = 1.4 KSF & N tan φ w/ φ = 21.3°
ALONG BASE OF THAT ½ OF PAD

~~00870~~

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CALCULATION IDENTIFICATION NUMBER				
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DYN BEARING CAPACITY OF PAD: 2-CASK CASE

CALCULATE L' SIMILARLY FOR LONGITUDINAL DIRECTION

$F_{H_{C_L}} = 300\text{ K}\ (= 40\%\ Q_{yd_{max}}$ FROM p B7 FOR 2 CASKS)

ΣM_{F_V}

$$R\,\Delta l = 1.5' \times \underbrace{\frac{129}{2}\text{ K}}_{1/2\text{ PAD}} + \underbrace{(3'+9.83')(300\text{ K})}_{\text{2-CASKS LONGITUDINAL LOADG}}$$

$R = F_V = 3790\text{ K}$

$$\therefore \Delta l = \frac{193.5\text{ K-FT} + 3849\text{ K-FT}}{3790\text{ K}} = 1.07\text{ FT}$$

ADD $e_{l_{F_V}} = 3.2' \Rightarrow e_l = 1.07' + 3.2' = 4.27'$

$L' = L - 2e_l = 33.5 - 2 \times 4.27' = 24.96'$ ← < 26.6'
∴ THIS = B'
& L' = 26.6'

$$q_{ACTUAL} = \frac{F_V}{B' \times L'} = \frac{3790^K}{24.96' \times 26.6'} = 5.71\text{ KSF}$$

CALC q_{ALLOW} FOR THE FOLLOWING: B' = 24.96' L' = 26.6'

$F_{H_{E-W}} = 377.2\text{K} + 129\text{ K} = 506.2\text{ K}$
(377.2 K: 2-CASK EQ_h (p B7); 129 K: 1/2 PAD EQ_h)

$F_{H_{N-S}} = 300\text{K N-S} + 129\text{ K PAD} = 429\text{ K N-S}$

FS = 1.1

$F_V = 3790$ K FOR 2-CASK (STATIC + DYN)

ASSUME $\gamma_{SURCH} = 100$ PCF FOR SOIL CEMENT &

$D_f = 3'$ (TOP OF PAD FLUSH WITH GRADE)

~~00871~~

FOR DYN LOADS, $\phi = 0°$ C = 2.2 KSF

6

CALCULATION IDENTIFICATION NUMBER				PAGE 80
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 2 CASKS

PSHA 2,000-Yr Earthquake: Case IVA — **40 % N-S, 100 % Vert, 40 % 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:

$B' =$ **25.0** Effective Ftg Width - ft (E-W) $L' =$ **26.6** Length - ft (N-S)
$D_f =$ **3.0** Depth of Footing (ft)

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
$F_V =$ **3,790** k (Includes EQ_V)
$EQ_{H\ E\text{-}W} =$ **506** k & $EQ_{H\ N\text{-}S} =$ 429 k → **664** 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)

Equation		Value	Reference
$N_c = (N_q - 1)\cot(\phi)$, 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.18	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.62	"
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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	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 $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.87 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.53	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 $\phi = 0$: $i_c = 1 - (m F_H / B'L'c N_c)$	=	0.86	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	12,419	psf =	12,119	+	300	+	0
q_{all} =	11,280	psf = q_{ult} / FS					
q_{actual} =	5,708	psf = $(F_v + EQ_v)/(B' \times L')$					
FS_{actual} =	2.18	= q_{ult}/q_{actual}			> 1.1	Hence OK	

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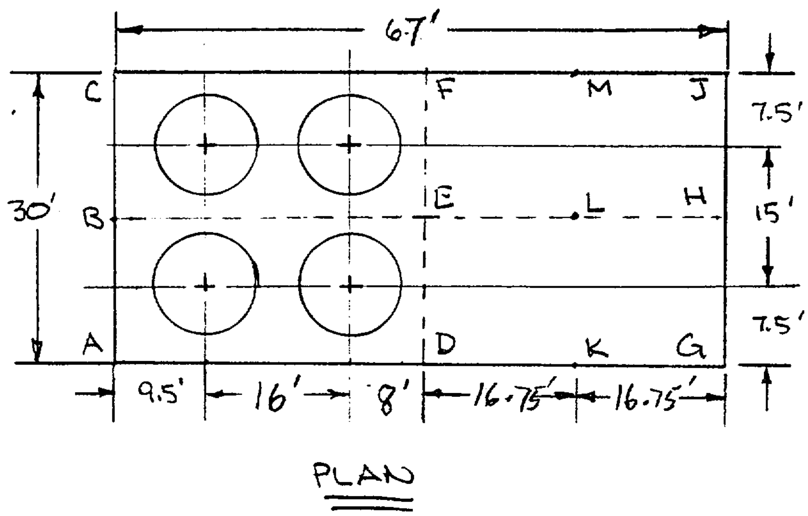
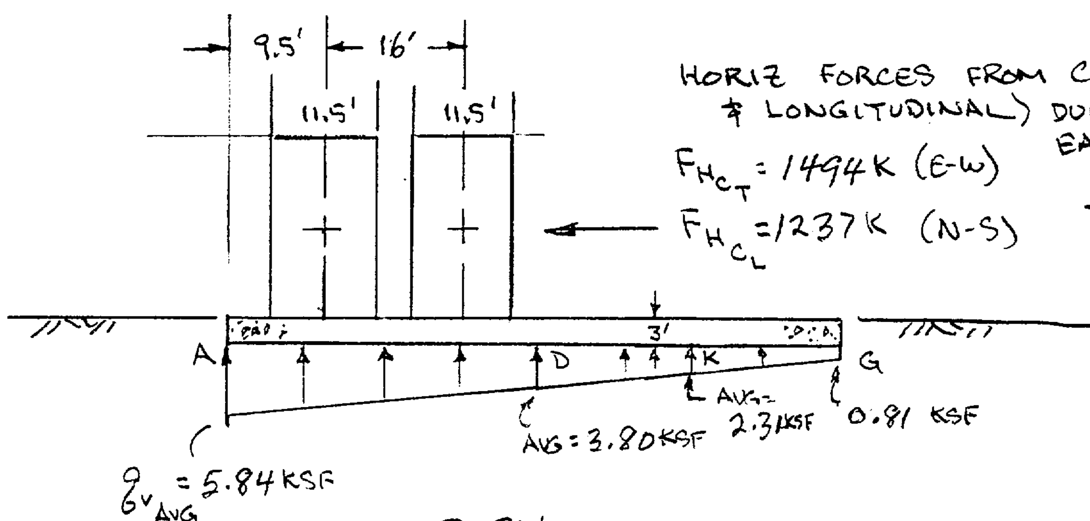
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-9	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 4-CASK CASE



PLAN



HORIZ FORCES FROM CASK (TRANSVERSE & LONGITUDINAL) DUE TO DESIGN EARTHQUAKE (FROM p 87)

$F_{H_{C_T}} = 1494^K$ (E-W)

$F_{H_{C_L}} = 1237^K$ (N-S)

TABLE D-1(c) CEC (2001)

ELEV

STRESSES AT PAD/SOIL INTERFACE FROM CEC (2001) SEE ATTACHMENT B AND NEXT 2 PAGES.

~~00874~~

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

5010.65

A CALCULATION IDENTIFICATION NUMBER				
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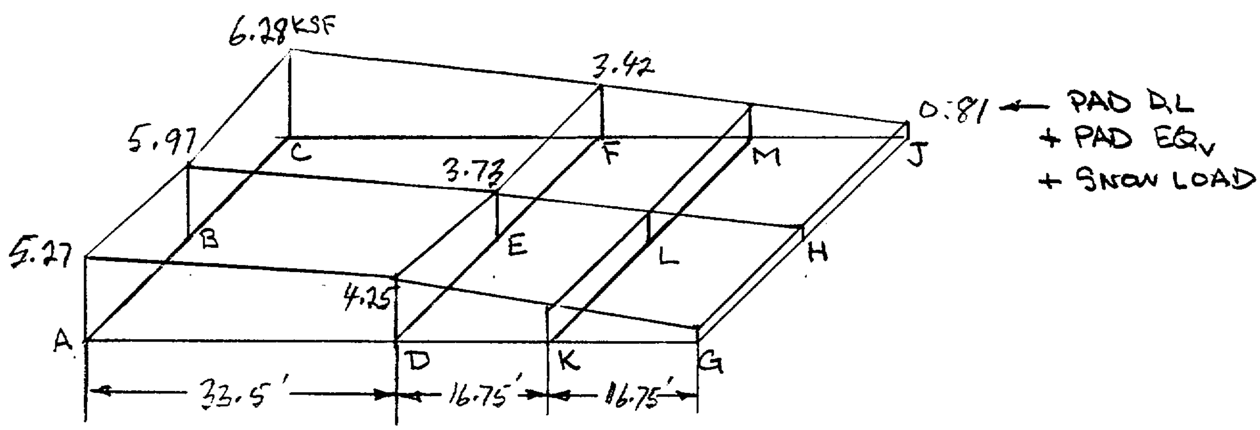
DYN BEARING CAPACITY OF PAD: 4-CASK CASE

SOIL BEARING PRESSURES ARE BASED ON INFO FROM CEC (2001) INCLUDED IN ATT B AND ARE SUMMARIZED IN TABLE 1.

VERTICAL PRESSURES INCLUDE: PAD DL = 0.45 KSF
PAD EQ = 0.31 KSF
SNOW LOAD = 0.045 KSF

LL OF CASKS = 1.71 KSF ALONG LINE AC & IS ASSUMED TO DECREASE LINEARLY TO 0 ALONG LINE GJ.

CASK EQ PRESSURES ARE SHOWN ON TABLE 1
RESULTING PRESSURE DISTRIBUTION:



ASSUME 3/4 OF PAD IS EFFECTIVE IN RESISTING LOADS OF 4-CASK CASE

$\therefore$ $B = 30'$ $\qquad L = \frac{3}{4}\, 67 = 50.25'$

LINEARLY DISTRIBUTE STATIC + DYN LOADING FROM LINE DF TO 50.25' AWAY FROM LINE AC & DETERMINE FV

VERT STRESSES		KSF	POINT
0.5 (4.25 + 0.81)	=	2.53	K
0.5 (3.73 + 0.81)	=	2.27	L
0.5 (3.42 + 0.81)	=	2.12	M

~~00875~~

B CALCULATION IDENTIFICATION NUMBER				PAGE 83
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

CALCULATE F_V

ALONG LINE	AREA = K/FT		
AC	$\frac{15'}{2}(5.27 + 2\times5.97 + 6.28)$ KSF	= 176.18	K/FT
DF	$\frac{15'}{2}(4.25 + 2\times3.73 + 3.42)$	= 113.48	
KM	$\frac{15'}{2}(2.53 + 2\times2.27 + 2.12)$	= 68.93	

$$F_V \sim \frac{33.5}{2}(176.18 + 113.48)^{K/_1} + \frac{16.75'}{2}(113.48 + 68.93)^{K/_1}$$

$$F_V = 4851.8\ K + 1527.7\ K = \underline{6379.5}\ K$$

ESTIMATE LOCATION WHERE F_V ACTS ON 30' x 50.25 PORTION OF PAD.

NOTE AVG VERT STRESS ALONG LINES.

LINE

$$AC = \frac{176.18\ K/FT}{30\ FT} = 5.87\ KSF$$

$$DF = \frac{113.48\ K/FT}{30'} = 3.78\ KSF$$

$$KM = \frac{68.93\ K/FT}{30'} = 2.30\ KSF$$

~~00876~~

CALCULATION IDENTIFICATION NUMBER				
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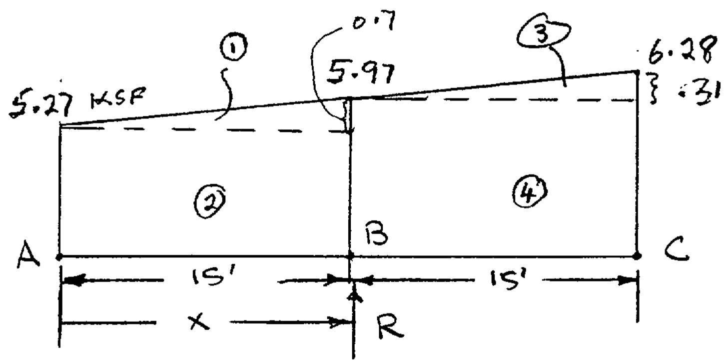
DYN BEARING CAPACITY OF PAD: 4-CASK CASE

DETERMINE ECCENTRICITY OF F_V WRT B, $e_{B_{F_V}}$

ALONG LINE AC



ΣM_A

AREA K/FT	MOMENT ARM (FT)	MOMENT
① $\frac{1}{2} \times 0.7 \frac{K}{FT^2} \times 15' = 5.25$	$2/3 \times 15 = 10'$	52.5
② $5.27 \frac{K}{FT^2} \times 15' = 79.05$	$1/2 \times 15 = 7.5'$	592.88
③ $\frac{1}{2} \times .31 \frac{K}{FT^2} \times 15' = 2.325$	$15 + 2/3 \times 15 = 25'$	58.125
④ $5.97 \frac{K}{FT^2} \times 15' = 89.55$	$15 + 1/2 \times 15 = 22.5$	2014.875
$F_V = \Sigma = 176.175$		$R_X = \Sigma = 2718.38$

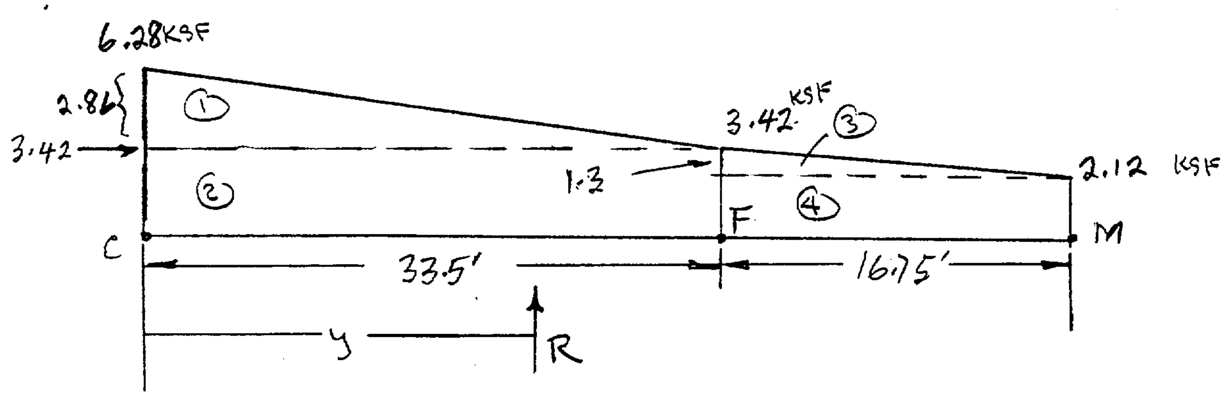
$$\therefore X = \frac{2718.38 \ K\text{-}FT/FT}{176.175 \ K/FT} = 15.4'$$

$$e_{B_{F_V}} = \frac{B}{2} - X = 15' - 15.4' = 0.4'$$

~~00877~~

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

DETERMINE ECCENTRICITY OF F_v WRT L, $e_{L_{Fv}}$



ΣM_A TO FIND y

	FORCE (K/')	MOMENT ARM (FT)	MOMENT K·FT/L.F.
1	$\frac{1}{2}\,2.86 \times 33.5' = 47.905$	$\frac{1}{3} \times 33.5' = 11.16'$	534.9
2	$3.42 \times 33.5 = 114.57$	$\frac{1}{2} \times 33.5' = 16.75'$	1919.0
3	$\frac{1}{2}\,1.30 \times 16.75 = 10.89$	$33.5' + \frac{1}{3}16.75' = 39.08$	425.6
4	$2.12 \times 16.75 = 35.51$	$33.5' + \frac{16.75}{2} = 41.875$	1487.0

$R = \Sigma F_v = 208.88$ K/' $Ry = \Sigma M = 4366.5$

$$y = \frac{\Sigma M}{\Sigma F_v} = \frac{4366.5 \text{ K-FT/LF}}{208.88 \text{ K/LF}} = 20.9 \text{ FT}$$

$$e_{L_{F_v}} = \frac{L}{2} - y = 25.125' - 20.9' = 4.23'$$

AS SHOWN ON NEXT PAGE

~~00878~~

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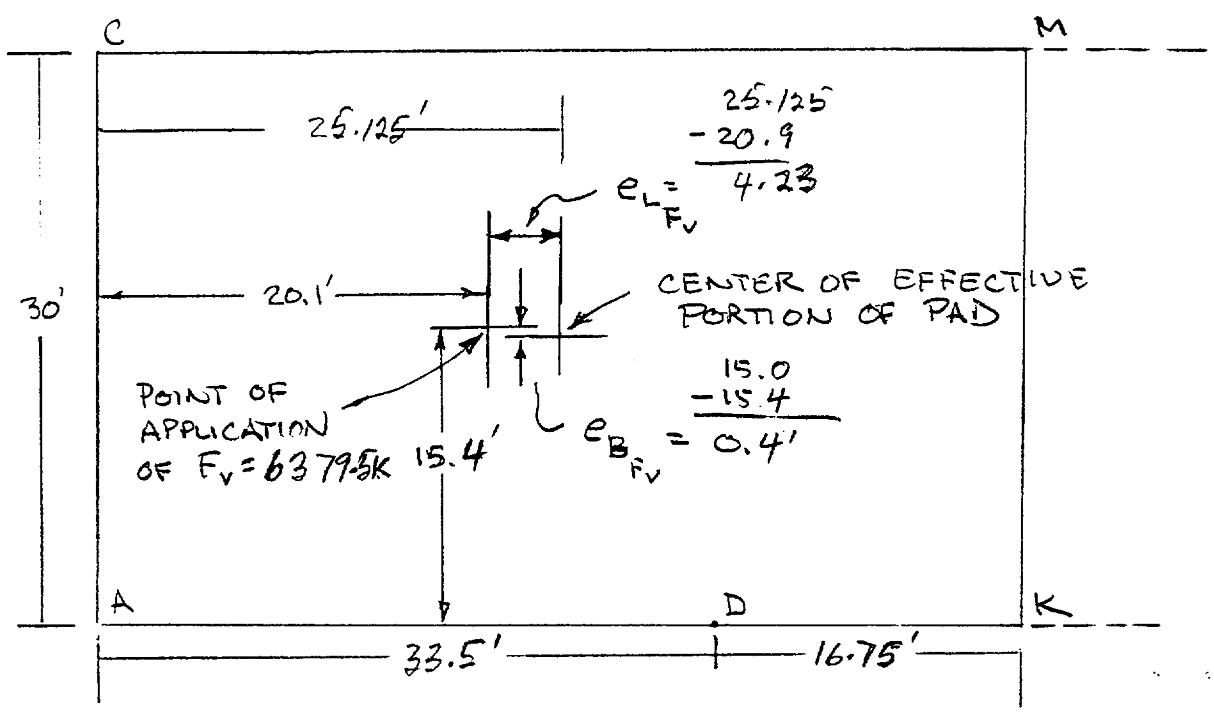
05996.02	G(B)	04-9		

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

PLAN VIEW OF PAD SHOWING
LOCATION OF VERTICAL FORCE
DUE TO VERTICAL STRESSES FOR
4-CASK LOADING CASE



~~00879~~

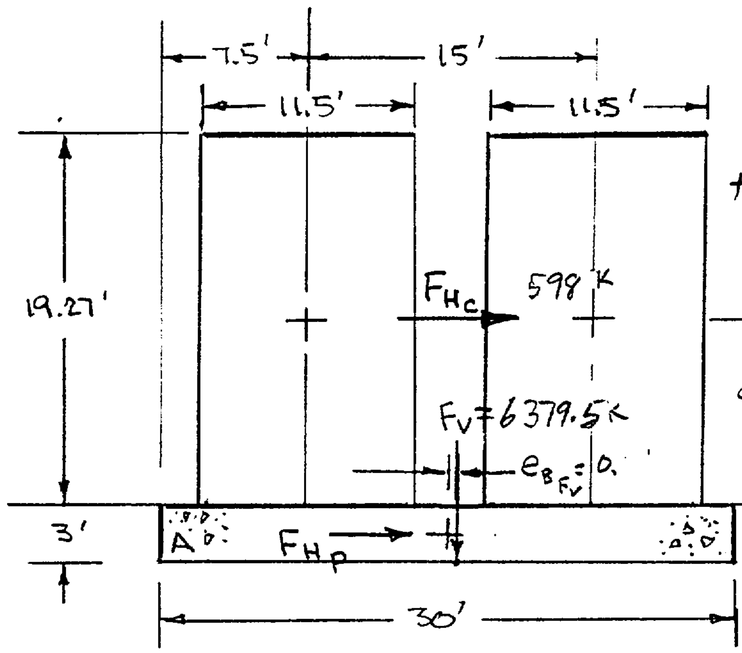
E	CALCULATION IDENTIFICATION NUMBER			PAGE 87
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-9	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

4-CASK Q_{xd} IN TABLE D-1(c)

Use 40% of $F_{H_{C_T}} = 1494K \times 0.4 = 598\,K$

$+\frac{3}{4} \times 40\% F_{H_p} = \frac{3}{4} \cdot 257 \Rightarrow 791\,k = EQ_{H\,E\text{-}W}$



CENTER OF GRAVITY OF CASK

9.83' = 118" (p C4 CALC 05996.01-G(B)-05-0)

40% $F_{H_p} = 257^K$ FOR 30'×67'

$= \dfrac{0.4 \times 0.711g \times 30' \times 67' \times 3' \times 0.15K/FT^3}{g}$

ΣM_{F_V} TO FIND LOC'N OF RESULTANT TO RESIST F_H's

3/4 PAD

40% 4-CASK TRANSVERSE F_H

$R\,\Delta b = 1.5' \times \frac{3}{4} \times 257^K + (3' + 9.83')\,598\,k$

$R = F_V = 6379.5^K$ ON 30×56.25' PORTION OF PAD

$$\therefore \Delta b = \frac{289 + 7{,}672\ \text{K-FT}}{6379.5^K} = 1.25' = e_{B_{F_H}}$$

$$e_b = e_{B_{F_V}} + e_{B_{F_H}} = 0.4' + 1.25' = 1.65'$$

$$B' = B - 2e_B = 30' - 2 \times 1.65' = \underline{26.7'}$$

~~00880~~

F	CALCULATION IDENTIFICATION NUMBER			PAGE 88
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

CALCULATE L' SIMILARLY FOR LONGITUDINAL F_H

$F_{H_{C_L}} = \overset{40\% \text{ of } 1237^K}{495^K} = Q_{yd_{MAX\ 4\ CASKS}}$ TABLE D-1(C) P B7 $F_{H_{C_L}}$

$+\frac{3}{4} \times 40\% F_{Hp} = \frac{3}{4}\, 257 \Rightarrow EQ_{H\ N\text{-}S} = 688\ K$

$\Sigma M_{F_V} \qquad R\,\Delta l = 1.5' \times \frac{3}{4} \times 257^K + (3' + 9.83')\ 495\ K$ ← 4 CASKS

$R = F_V = 6379.5^K$ ON EFFECTIVE PORTION OF PAD (30' x 50.25')

$$\Delta l = \frac{289^{K\text{-}FT} + 6{,}351^{K\text{-}FT}}{6379.5^K} = 1.04' = e_{L_{F_H}}$$

$$e_L = e_{L_{F_V}} + e_{L_{F_H}} = 4.23' + 1.04 = 5.27'$$

$$L' = L - 2e_L = 50.25' - 2 \times 5.27' = 39.71'$$

$$q_{ACTUAL} = \frac{F_V}{B' \times L'} = \frac{6379.5^K}{26.7 \times 39.71} = \underline{\underline{6.02\ KSF}}$$

CALC q_{ALLOW} FOR THE FOLLOWING: $B' = 26.7'$ $L' = 39.71'$

$F_V = 6379.5^K$ FOR 4-CASK CASE (STATIC + DYN)

$EQ_{H\ E\text{-}W} = \overset{PAD}{\frac{3}{4}\, 257} + \overset{F_{HC}}{598} = 791\ K$ E-W

$EQ_{H\ N\text{-}S} =$ " $+ 495 = 688$ N-S

$FS = 1.1 \quad \gamma_{SURCH} = 100\ PCF \quad \gamma = 80\ PCF \quad D_f = 3'$

$\phi = 0° \quad c = 2.2\ KSF$

~~00097~~ 6

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 4 CASKS

PSHA 2,000-Yr Earthquake: Case IVA — **40 % N-S, 100 % Vert, 40 % E-W**

				Footing Dimensions:	
Soil Properties:	$c =$	**2,200**	Cohesion (psf)		
	$\phi =$	**0.0**	Friction Angle (degrees)	B = **30.0**	Width - ft (E-W)
	$\gamma =$	**80**	Unit weight of soil (pcf)	L = **67.0**	Length - ft (N-S)
	$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)		
Foundation Properties:	$B' =$	**26.7**	Effective Ftg Width - ft (E-W)	L' = **39.7**	Length - ft (N-S)
	$D_f =$	**3.0**	Depth of Footing (ft)		

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

$F_V =$ **6,380** k (Includes EQ_V)

$EQ_{H\,E\text{-}W} =$ **791** k & $EQ_{H\,N\text{-}S} =$ **688** k → **1,048** 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)

Equation		Value	Reference
$N_c = (N_q - 1)\cot(\phi)$, 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.13	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.73	"
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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.04	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 $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.85 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.53	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 $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.87	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**11,879**	**psf =**	**11,579**	+	**300**	+	**0**

q_{all} = 10,790 psf = q_{ult} / FS

q_{actual} = 6,017 psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = 1.97 = q_{ult}/q_{actual} > **1.1 Hence OK**

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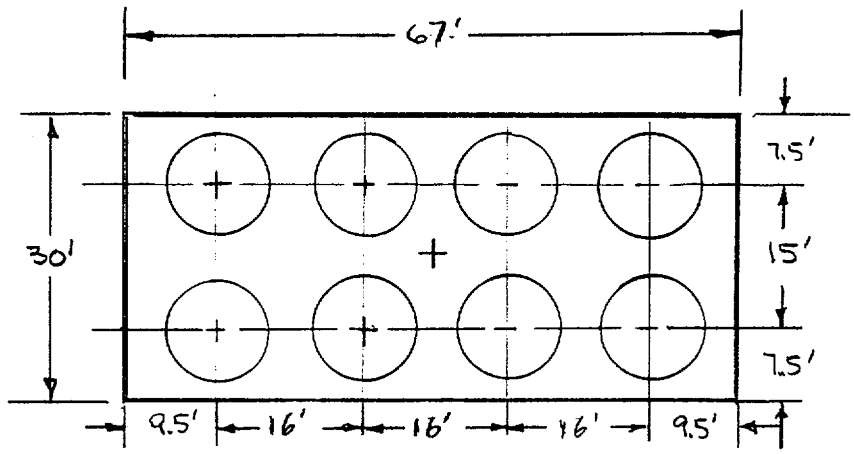
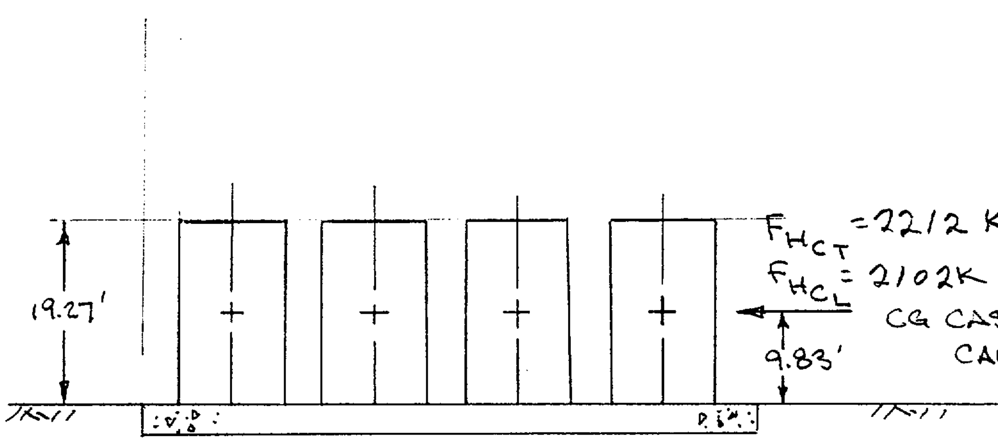
DYN BEARING CAPACITY OF PAD: 8-CASK CASE



PLAN



$F_{H_{CT}}$ = 2212 K E-W
$F_{H_{CL}}$ = 2102K N-S } p. B7

CG CASK (p C4 of CALC 05996.01-G(B)-05-0)

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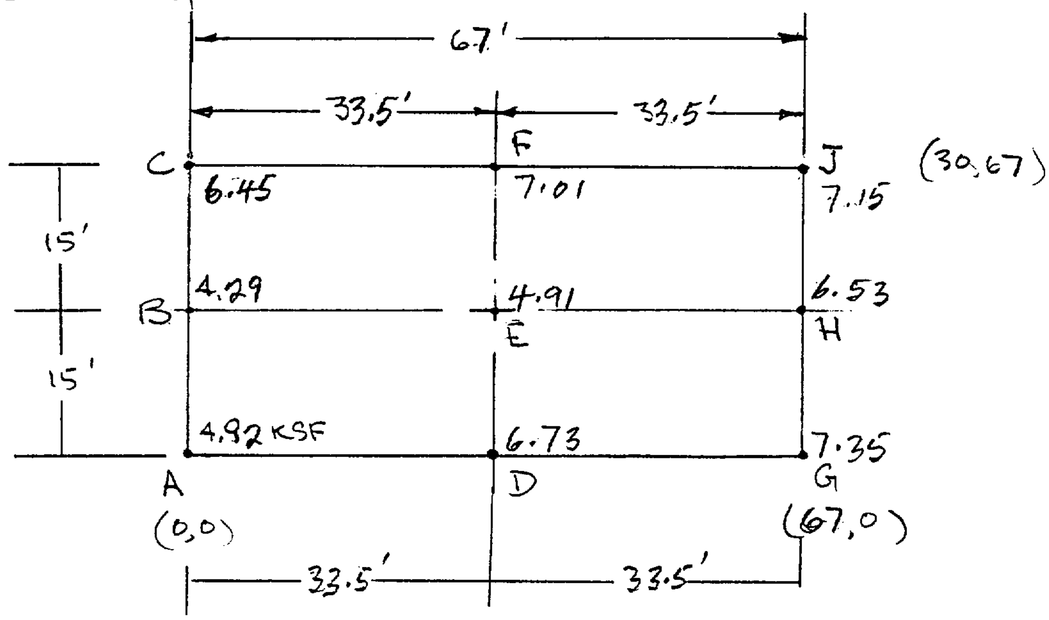
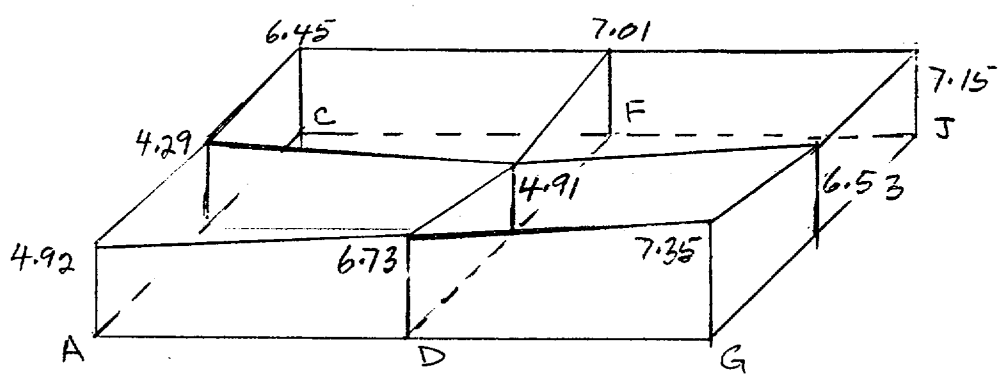
DYN BEARING CAPACITY OF PAD: 8-CASK CASE

SOIL BEARING PRESSURES FROM TABLE 1





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DYN BEARING CAPACITY OF PAD: 8-CASK CASE

CALCULATE F_V :

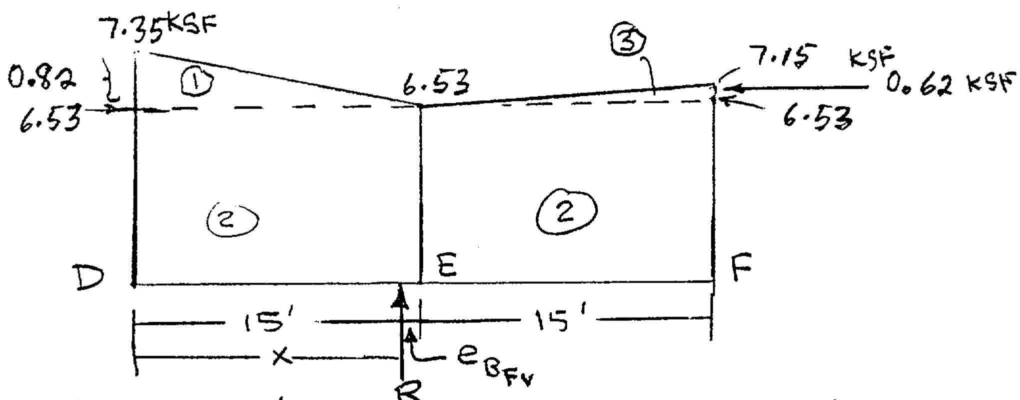
ALONG LINE	AREA = K/FT		F_V (K/FT)	q_{AVG} (KSF)
AC	$\frac{15}{2}(4.92 + 2\times 4.29 + 6.45)$	=	149.63	4.99
DF	$\frac{15}{2}(6.73 + 2\times 4.91 + 7.01)$	=	176.70	5.89
GJ	$\frac{15}{2}(7.35 + 2\times 6.53 + 7.15)$	=	206.7	6.89

$$F_V \sim \frac{33.5}{2}(149.63 + 2\times 176.7 + 206.7) = \underline{11{,}888 \text{ K}}$$

ESTIMATE LOCATION WHERE F_V ACTS.
DETERMINE ECCENTRICITY OF F_V WRT B, $e_{BF_V} = \frac{B}{2} - x$

ALONG LINE GJ, WHICH HAS THE GREATEST STRESSES

ΣM_D



AREA	K/FT	MOMENT ARM (FT)	MOMENT K·FT/FT
① $\frac{1}{2}0.82\text{KSF}\times 15'$	= 6.15	$\frac{1}{3}\times 15' = 5'$	30.75
② $6.53\times 30'$	= 195.9	$\frac{1}{2}\times 30' = 15'$	2938.5
③ $\frac{1}{2}0.62\times 15'$	= 4.65	$15' + \frac{2}{3}\times 15' = 25'$	116.25
	$R = \Sigma = 206.7$		$Rx = \Sigma = 3085.5$ ~~00886~~

$$\therefore x = \frac{Rx}{R} = \frac{3085.5 \text{ K-FT/FT}}{206.7 \text{ K/FT}} = 14.92' \qquad e_{BF_V} = \frac{30}{2} - 14.92 = 0.08'$$

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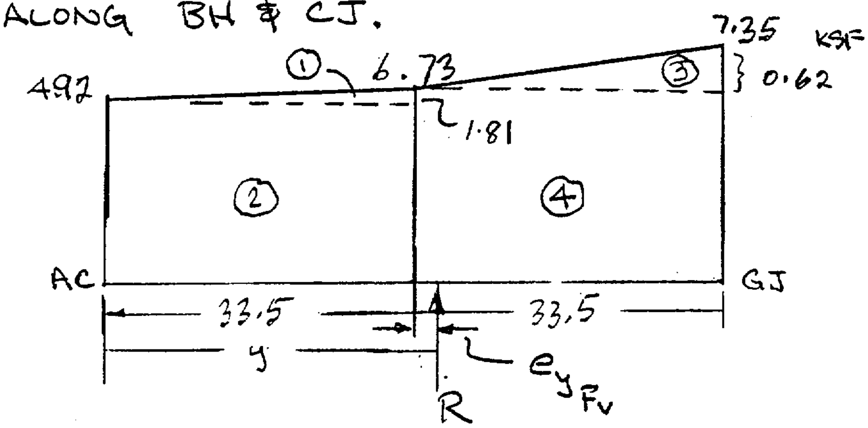
DYNAMIC BEARING CAPACITY OF PAD: 8-CASK CASE

DETERMINE ECCENTRICITY OF F_V WRT L, $e_{L_{F_V}} = \frac{L}{2} - y$

USE AVERAGE VALUES ALONG LINE AG, WHICH ARE GREATER THAN ALONG BH & CJ.



ΣM_{AC}

AREA	K/FT	MOMENT ARM FT	MOMENT K-FT/FT
①	$\frac{1}{2}(1.81\ \text{KSF}) \times 33.5 = 30.32$	$\frac{2}{3} \times 33.5 = 22.33'$	677.09
②	$4.92\ \text{KSF} \times 33.5 = 164.82$	$\frac{1}{2} \times 33.5 = 16.75'$	2,760.74
③	$\frac{1}{2}(7.35 - 6.73)^{KSF} \times 33.5' = 10.38$	$33.5' + \frac{2}{3} 33.5 = 55.83'$	580.07
④	$6.73\ \text{KSF} \times 33.5' = 225.46$	$33.5' + \frac{1}{2} 33.5 = 50.25'$	11,329.37
	R = 430.98 K/FT		Ry = 15,347.3 K-FT/FT

$$\therefore\ y = \frac{Ry}{R} = \frac{15,347.3\ \text{K-FT/FT}}{430.98\ \text{K/FT}} = 35.61'$$

$$e_{L_{F_V}} = y - \frac{L}{2} = 35.61' - \frac{67'}{2} = 2.11'$$

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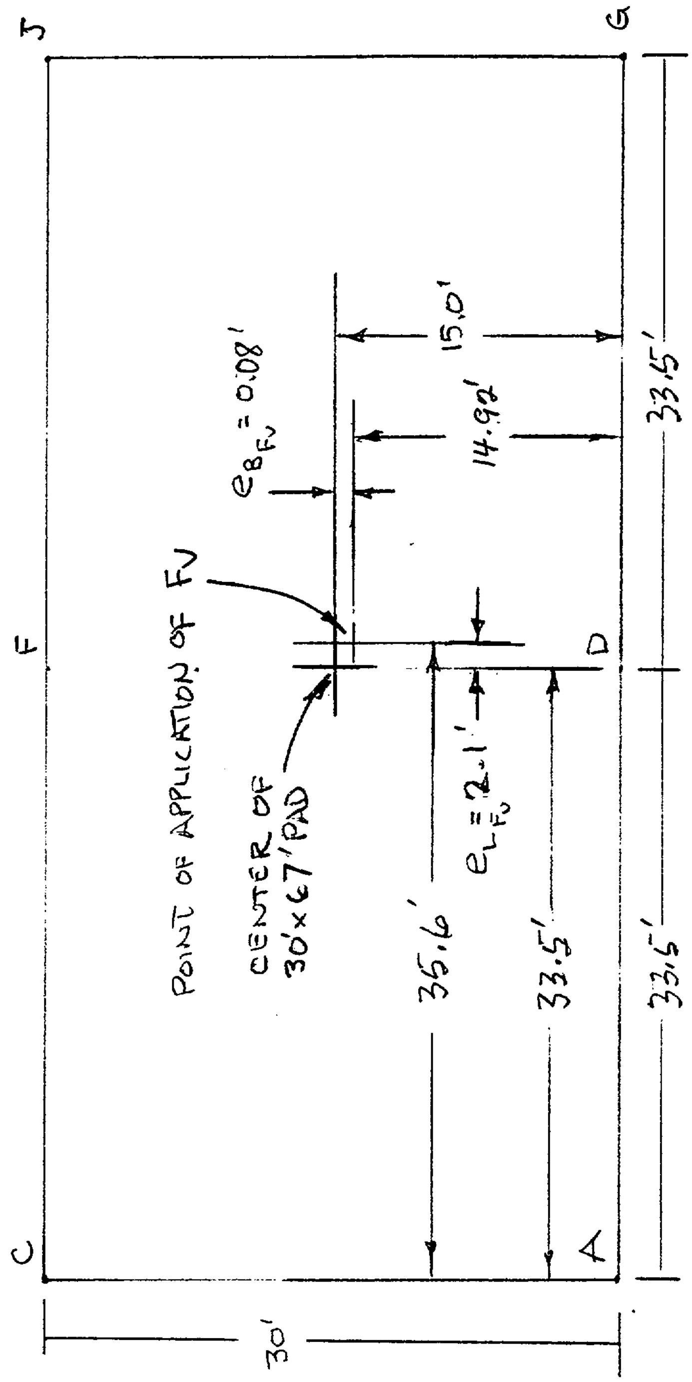
PLAN VIEW OF PAD SHOWING ~ LOCATION OF VERTICAL FORCE DUE TO VERTICAL STRESSES FROM STATIC AND DYNAMIC LOADS FOR 8-CASK LOADING CASE



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▲ 5010.65

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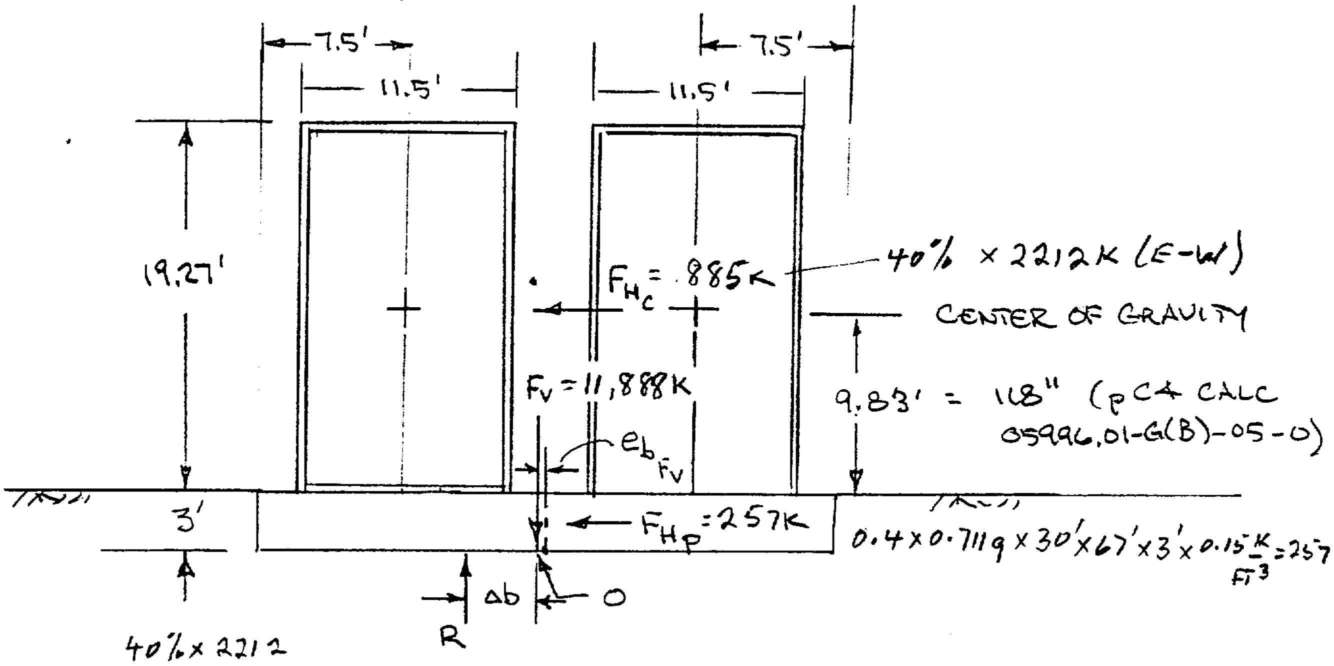
DYN BEARING CAPACITY OF PAD: 8-CASK CASE



F_{H_C} = 885 K FOR 8-CASK CASE (p B7)
TRANVERSE DIRECTION E-W

$$F_{H_P} = 0.4 \times \overset{a_h}{.711}\,g \times 30' \times 67' \times 3' \times \frac{0.15\ \text{KCF}}{g} = 257\ \text{K}$$

$$\Sigma = EQ_{H_{E-W}} = 1142\ \text{K}$$

ΣM_o TO FIND LOCATION OF R TO RESIST MOMENT DUE TO F_H's

$$\underset{=F_V}{R}\,\Delta b = \underset{\text{PAD}}{1.5' \times 257\ \text{K}} + \underset{\text{CASKS}}{(3'+9.83')\ 885\ \text{K}}$$

$$\Delta b = \frac{386 + 11,355\ \text{K-FT}}{11,888\ \text{K}} = 0.99\ \text{FT}$$

ADD $e_{b_{F_V}} = 0.08$ FT $\Rightarrow$ $e_b = 0.99 + 0.08 = 1.07$ FT

$$B' = B - 2e_b = 30' - 2 \times 1.07' = 27.86\ \text{FT}$$

~~00889~~

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DYN BEARING CAPACITY OF PAD: 8-CASK CASE

SIMILARLY FOR LONGITUDINAL DIRECTION

40% of 2102K

$F_{H_C} = 841$ K ← p 87 ADD $F_{HP} = 257$ K ⇒ $EQ_{H_{N-S}} = 1098$ K

ΣM_0 $\quad \underset{R=F_v}{11,888\text{ K}}\ \Delta l = \underset{\text{PAD}}{1.5' \times 257\text{ K}} + \underset{\text{CASKS}}{(3' + 9.83')(841\text{ K})}$

$$\Delta l = \frac{386 + 10,790 \text{ K-FT}}{11,888 \text{ K}} = 0.94'$$

ADD $e_{l_{F_v}} = 2.1$ FT ⇒ $e_l = 0.94' + 2.1' = 3.04'$

$L' = L - 2e_l = 67' - 2 \times 3.04' = 60.92$ FT

$$q_{ACTUAL} = \frac{F_v}{B' \times L'} = \frac{11,888 \text{ K}}{27.86' \times 60.92} = 7.00 \text{ KSF}$$

CALC q_{ALLOW} FOR FS = 1.1 $\quad B' = 27.86' \quad L' = 60.92'$

$F_v = 11,888$ K (STATIC + DYN 8 CASKS)

$EQ_{H_{E-W}} = \underset{F_{HP}}{257\text{ K}} + \underset{F_{HC}}{885\text{ K}} = 1142$ K

$EQ_{H_{N-S}} = 257$ K + 841 K = 1098 K

$\gamma_{SURCH} = 100$ PCF $\quad \gamma = 80$ PCF $\quad D_f = 3'$

$\phi = 0°$ $\quad C = 2.2$ KSF

$\boxed{6}$

~~00890~~

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DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 8 CASKS

PSHA 2,000-Yr Earthquake: Case IVA — **40 % N-S, 100 % Vert, 40 % 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:

$B' =$ **27.9** Effective Ftg Width - ft (E-W) $L' =$ **60.9** Length - ft (N-S)
$D_f =$ **3.0** Depth of Footing (ft)

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
$F_V =$ **11,888** k (Includes EQ_V)
$EQ_{H\ E\text{-}W} =$ **1,142** k & $EQ_{H\ N\text{-}S} =$ 1,098 k → **1,584** 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
	$s_c = 1 + (B/L)(N_q/N_c)$	= 1.09	Table 3.2
	$s_q = 1 + (B/L)\tan\phi$	= 1.00	"
	$s_\gamma = 1 - 0.4\,(B/L)$	= 0.82	"
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 $\phi = 0$:	$d_c = 1 + 0.4\,(D_f/B)$	= 1.04	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 $EQ_{H\ N\text{-}S} > 0$:	$\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	= 0.81 rad	
	$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	= 1.51	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 $\phi = 0$:	$i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	= 0.88	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	11,546	psf =	11,246	+	300	+	0
q_{all} =	10,490	psf = q_{ult} / FS					
q_{actual} =	7,004	psf = $(F_v + EQ_v)/(B' \times L')$					
FS_{actual} =	1.65	= q_{ult}/q_{actual}			> 1.1 Hence OK		

Table 2.6-8 presents a summary of the bearing capacity analyses that were performed using the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 2001) for the pad supporting 2 casks, 4 casks, and 8 casks. Details of these analyses are presented on the preceding pages. These analyses are performed for Load Case IVA, where 40% of the horizontal forces due to the earthquake are applied in both the N-S and the E-W directions and 100% of the vertical force is applied to obtain the maximum vertical load on the cask storage pad. The width (30 ft) is less in the E-W direction than the length N-S (67 ft); therefore, the E-W direction is the critical direction with respect to a bearing capacity failure.

As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the very conservative maximum dynamic cask driving forces due to the design basis ground motion is at least 10.5 ksf for the 2-cask, 4-cask, and 8-cask loading cases. The minimum allowable value was obtained for the 8-cask loading case. The actual factor of safety for this case was 1.6, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

CONCLUSIONS

Analyses presented herein demonstrate that the cask storage pads have adequate factors of safety against overturning, sliding, and bearing capacity failure for static and dynamic loadings due to the design basis ground motion. The following load cases are considered:

- Case I — Static
- Case II — Static + dynamic horizontal forces due to the earthquake
- Case III — Static + dynamic horizontal + vertical uplift forces due to the earthquake
- Case IV — Static + dynamic horizontal + vertical compression forces due to the earthquake

For Case II, 100% of the dynamic lateral forces in both the N-S and E-W directions are combined. For Cases III and IV, the effects of the three components of the design basis ground motion are combined in accordance with procedures described in ASCE (1986); i.e., 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the loading acts in the other two directions.

These results of these stability analyses are discussed in more detail in the following sections.

OVERTURNING STABILITY OF THE CASK STORAGE PADS

Analyses presented above indicate that the factor of safety against overturning due to dynamic loadings from the design basis ground motion is 5.6. This is greater than the criterion of 1.1 for the factor of safety against overturning due to dynamic loadings; therefore, the cask storage pads have an adequate factor of safety against overturning due to loadings from the design basis ground motion.

SLIDING STABILITY OF THE CASK STORAGE PADS

The cask storage pads will be constructed on and within soil cement, as shown in Figure 3. Analyses presented above demonstrate 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. This soil cement will be designed to have a minimum unconfined compressive strength of 40 psi. 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. The factor of safety against sliding between the concrete at the base of the pad and the surface of the underlying soil cement is greater than 1.98, which exceeds the factor of safety between the bottom of the soil cement and the underlying clayey soils. Therefore, the minimum factor of safety against sliding of the overall cask storage pad design is at least 1.27.

Since the resistance to sliding of the cask storage pads is provided by the strength of the bond at the interface between the concrete pad and the underlying soil cement and by the

bond between the soil cement under the pad and the in situ clayey soils, the sliding stability of the pads at the end of each column or row of pads are no different than that of the other pads. Therefore, the pads along the perimeter of the pad emplacement area also have an adequate factor of safety against sliding. Further, the soil-cement layer is continuous throughout the pad emplacement area; therefore, the area available to resist sliding of an entire column of pads greatly exceeds the sum of the areas of only the pads in the column. The factor of safety against sliding of an entire column of pads will, therefore, exceed that of an individual pad.

Additional analyses presented above demonstrate that even if the cohesion of the underlying soils is ignored along the interface between the soil cement and those soils, the resulting displacement of the pads would be minimal. This hypothetical case assumes resistance to sliding is comprised of only frictional resistance along base of pads and soil cement + passive resistance, using obviously conservative values of the friction angle for the underlying soils. Assuming the cask storage pads are founded directly on a layer of cohesionless soils with $\phi = 17°$, the resulting factor of safety is less than 1.1. The relative displacement of the pads due to the design basis ground motion was estimated using Newmark's method of estimating displacements of embankments and dams due to earthquakes. The analysis indicates that the maximum displacement of the pads ranges from ~2 inches to ~6 inches for this hypothetical case. There are several conservative assumptions that were made in determining these values for this hypothetical case, and, therefore, the estimated displacements represent upper-bound values. Even if the maximum horizontal displacement were to occur from an earthquake, there would be no safety consequence to the pads or the casks, since the pads and casks do not rely on any external "Important to Safety" connections.

Analyses presented above also address the possibility that sliding may occur along a deep slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces. To simplify the analysis, it was assumed that cohesionless soils extend above the 10 ft depth and, thus, the pads are founded directly on cohesionless materials. Because of the magnitude of the peak ground accelerations (0.71g) due to the design basis ground motion at this site, the frictional resistance available for cohesionless soils when the normal stress is reduced due to the uplift from the inertial forces applicable for the vertical component of the design basis ground motion is not sufficient to resist sliding. However, analyses were performed to estimate the amount of displacement that might occur due to the design basis ground motion for this case. These analyses, based on the method of estimating displacements of dams and embankments during earthquakes developed by Newmark (1965), indicate that even if these soils are cohesionless and even if they are conservatively located directly at the base of the pads, the estimated displacements would be ~2.2 inches. Whereas there are no connections between the ground and these pads or between the pads and other structures, this minor amount of displacement would not adversely affect the performance of these structures if it did occur.

Allowable Bearing Capacity of the Cask Storage Pads

Static Bearing Capacity of the Cask Storage Pads

Analyses of bearing capacity for static loads are summarized in Table 2.6-6. As indicated for Case IA, the factor of safety of the cask storage pad foundation is 7.0 using the undrained strength for the cohesive soils that was measured in the UU tests (s_u > 2.2 ksf) that were performed at depths of approximately 10 to 12 feet. The results for Case IB illustrates that the factor of safety against a bearing capacity failure increases to greater than 15 when the effective-stress strength of $\phi = 30°$ is used. The minimum gross allowable bearing capacity exceeds 4 ksf for static loads. Therefore, these analyses demonstrate that the factor of safety against a bearing capacity failure exceeds the minimum allowable value of 3 for static loads.

Dynamic Bearing Capacity of the Cask Storage Pads

Analyses of bearing capacity for dynamic loads are summarized in Tables 2.6-7 and 2.6-8. Table 2.6-7 presents the results of the bearing capacity analyses based on the inertial forces applicable for the peak ground accelerations from the design basis ground motion. Table 2.6-8 presents the results of the analyses based on the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 2001) for the pad supporting 2 casks, 4 casks, and 8 casks. These latter dynamic forces represent the maximum forces occurring at any time during the earthquake at each node in the model used to represent the cask storage pads. It is expected that these maximum forces will not occur at the same time for every node. These forces, therefore, represent an upper bound of the dynamic forces that could act at the base of the pad.

Table 2.6-7 presents the results of the dynamic bearing capacity analyses for the following cases, which include static loads plus inertial forces due to the earthquake.

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
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.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction

As indicated in Table 2.6-7, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the inertial loads due to the design basis ground motion exceeds 4.8 ksf for all loading cases identified above. The minimum allowable value was obtained for Load Case II, wherein 100% of the earthquake loads act in the N-S and E-W directions and 0% acts in the Vertical direction,

tending to rotate the cask storage pad about the N-S axis. The actual factor of safety for this condition was 1.2, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1). In Load Cases III and IV, the effects of the three components of the earthquake in accordance with procedures described in ASCE (1986) to account for the fact that the maximum response of the three orthogonal components of the earthquake do not occur at the same time. For these cases, 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the dynamic loading acts in the other two directions. For these load cases, the gross allowable bearing capacity of the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the inertial loads due to the design basis ground motion exceeds 6.7 and the factor of safety exceeds 2.1.

Table 2.6-8 presents a summary of the bearing capacity analyses that were performed using the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 2001) for the pad supporting 2 casks, 4 casks, and 8 casks. These analyses are performed for Load Case IVA, where 40% of the horizontal forces due to the earthquake are applied in both the N-S and the E-W directions and 100% of the vertical force is applied to obtain the maximum vertical load on the cask storage pad. The width (30 ft) is less in the E-W direction than the length N-S (67 ft); therefore, the E-W direction is the critical direction with respect to a bearing capacity failure.

As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the very conservative maximum dynamic cask driving forces due to the design basis ground motion is at least 10.5 ksf for the 2-cask, 4-cask, and 8-cask loading cases. The minimum allowable value was obtained for the 8-cask loading case. The actual factor of safety for this case was 1.6, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

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TABLE 1

Summary of Vertical Soil Bearing Pressures (ksf) from Calc 05996.02-G(PO17)-2, Rev. 3

Loading	Point	A (287)	B (293)	C (299)	D (144)	E (150)	F (156)	G (1)	H (7)	J (13)
2-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.345	1.352	1.345	0.185	0.199	0.185	0.00	0.00	0.00
	Pad EQ	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EQ	4.11	3.90	3.18	0.84	0.52	0.56	0.00	0.00	0.00
	100% Vert	**6.26**	**6.06**	**5.33**	**1.83**	**1.53**	**1.55**	**0.81**	**0.81**	**0.81**
4-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.71	1.71	1.71	0.76	0.76	0.76	0.00	0.00	0.00
	Pad EQ	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EQ	2.75	3.45	3.76	2.69	2.16	1.86	0.00	0.00	0.00
	100% Vert	**5.27**	**5.97**	**6.28**	**4.25**	**3.73**	**3.42**	**0.81**	**0.81**	**0.81**
8-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.402	1.402	1.402	1.514	1.516	1.514	1.402	1.402	1.402
	Pad EQ	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EQ	2.71	2.08	4.24	4.41	2.59	4.69	5.14	4.32	4.94
	100% Vert	**4.92**	**4.29**	**6.45**	**6.73**	**4.91**	**7.01**	**7.35**	**6.53**	**7.15**

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TABLE 2.6-6
SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS
Based on Static Loads

Case	F_V k	$EQ_{H\ N\text{-}S}$ k	$EQ_{H\ E\text{-}W}$ k	$\Sigma M_{@N\text{-}S}$ ft-k	$\Sigma M_{@E\text{-}W}$ ft-k	β_B $EQ_{H\ E\text{-}W}$ deg	β_L $EQ_{H\ N\text{-}S}$ deg	GROSS q_{ult} ksf	GROSS q_{all} ksf	e_B ft	e_L ft	EFFECTIVE B' ft	EFFECTIVE L' ft	EFFECTIVE q_{actual} ksf	FS_{actual}
IA - Static Undrained Strength	3,757	0	0	0	0	0.0	0.0	13.08	**4.36**	0.0	0.0	30.0	67.0	**1.87**	**7.0**
IB - Static Effective Strength	3,757	0	0	0	0	0.0	0.0	29.22	**9.73**	0.0	0.0	30.0	67.0	**1.87**	**15.6**

$\phi =$ **30** Effective stress friction angle (deg), c=0.

$c =$ **2,200** Undrained strength (psf), ϕ=0.

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

$B =$ **30** Footing width (ft)

$L =$ **67** Footing length (ft)

$D_f =$ **3.0** Depth of footing (ft)

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

$FS =$ **1.1** Factor of safety for static loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E\text{-}W}$ or $EQ_{H\ N\text{-}S}$

$\beta_B = \tan^{-1} [(EQ_{H\ E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(

$\beta_L = \tan^{-1} [(EQ_{H\ N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\,e_B$ $L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

[geot]\05996\calc\brng_cap\Pad\Wint_Fang-8.xls Table 2.6-6

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TABLE 2.6-7
SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS
Based on Inertial Forces Due to Design Earthquake: PSHA 2,000-Yr Return Period

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\ E\text{-}W}$	β_L $EQ_{H\ N\text{-}S}$	GROSS		e_B	e_L	EFFECTIVE			FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	q_{ult} ksf	q_{all} ksf	ft	ft	B' ft	L' ft	q_{actual} ksf	
II	3,757	2,671	2,671	26,982	26,982	35.4	35.4	5.34	**4.85**	7.2	7.2	15.6	52.6	**4.56**	**1.2**
IIIA	1,146	749	749	6,699	6,699	33.2	33.2	11.34	**10.31**	5.8	5.8	18.3	55.3	**1.13**	**10.0**
IIIB	2,712	1,068	2,077	19,361	10,793	37.4	21.5	8.51	**7.73**	7.1	4.0	15.7	59.0	**2.92**	**2.9**
IIIC	2,712	2,077	1,068	10,793	19,361	21.5	37.4	10.01	**9.10**	4.0	7.1	22.0	52.7	**2.33**	**4.3**
IVA	6,368	1,068	1,068	10,793	10,793	9.5	9.5	11.57	**10.51**	1.7	1.7	26.6	63.6	**3.76**	**3.1**
IVB	4,801	1,068	2,671	26,982	10,793	29.1	12.5	8.51	**7.73**	5.6	2.2	18.8	62.5	**4.09**	**2.1**
IVC	4,801	2,671	1,068	10,793	26,982	12.5	29.1	10.05	**9.13**	2.2	5.6	25.5	55.8	**3.38**	**3.0**

$c =$ **2,200** Undrained strength (psf)

$\phi =$ **0.0** Friction angle (deg)

$B =$ **30** Footing width (ft)

$L =$ **67** Footing length (ft)

$D_f =$ **3.0** Depth of footing (ft)

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

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

$FS =$ **1.1** Factor of safety for dynamic loads.

F_V = Vertical load ($F_{V\ Static} + EQ_V$)

EQ_H = Earthquake: Horizontal force. $F_H = SQRT[EQ_H{}^2{}_{E\text{-}W} + EQ_H{}^2{}_{N\text{-}S}]$

$\beta_B = \tan^{-1}[(EQ_{H\ E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\ N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\quad$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\,e_B$ $\quad$ $L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

0.711 $g = a_H$

0.695 $g = a_V$

[geol]\05996\calc\brng_cap\Pad\Wint_Fang-8.xls Table 2.6-7

TABLE 2.6-8

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Maximum Cask Driving Forces Due to Design Earthquake: PSHA 2,000-Yr Return Period for Loading Case IV: 40% N-S, 100% Vertical, and 40% E-W

Case IV	F_V k	$EQ_{H\,N\text{-}S}$ k	$EQ_{H\,E\text{-}W}$ k	$\Sigma M_{@N\text{-}S}$ ft-k	$\Sigma M_{@E\text{-}W}$ ft-k	β_B $EQ_{H\,E\text{-}W}$ deg	β_L $EQ_{H\,N\text{-}S}$ deg	GROSS q_{ult} ksf	GROSS q_{all} ksf	e_B ft	e_L ft	EFFECTIVE B' ft	EFFECTIVE L' ft	EFFECTIVE q_{actual} ksf	FS_{actual}
2 Casks	3,790	429	506	6,443	16,183	7.6	6.5	12.42	**11.28**	1.70	4.27	25.0	26.6	**5.71**	**2.2**
4 Casks	6,380	688	791	10,526	33,620	7.1	6.2	11.88	**10.79**	1.65	5.27	26.7	39.7	**6.02**	**2.0**
8 Casks	11,888	1,098	1,142	12,720	36,140	5.5	5.3	11.55	**10.49**	1.07	3.04	27.9	60.9	**7.00**	**1.6**

$c =$	**2,200**	Undrained strength (psf)
$\phi =$	**0.0**	Friction angle (deg)
$B =$	**30**	Footing width (ft)
$L =$	**Varies**	Footing length (ft)
$D_f =$	**3.0**	Depth of footing (ft)
$\gamma =$	**80**	Unit weight of soil (pcf)
$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)
$FS =$	**1.1**	Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$\Sigma M_{@N\text{-}S} = e_B \times F_V$ $\qquad \Sigma M_{@E\text{-}W} = e_L \times F_V$

$B' = B - 2\,e_B$ $\qquad L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

[geotl]\05996\calc\brng_cap\Pad\Wint_Fang-8.xls Table 2.6-8

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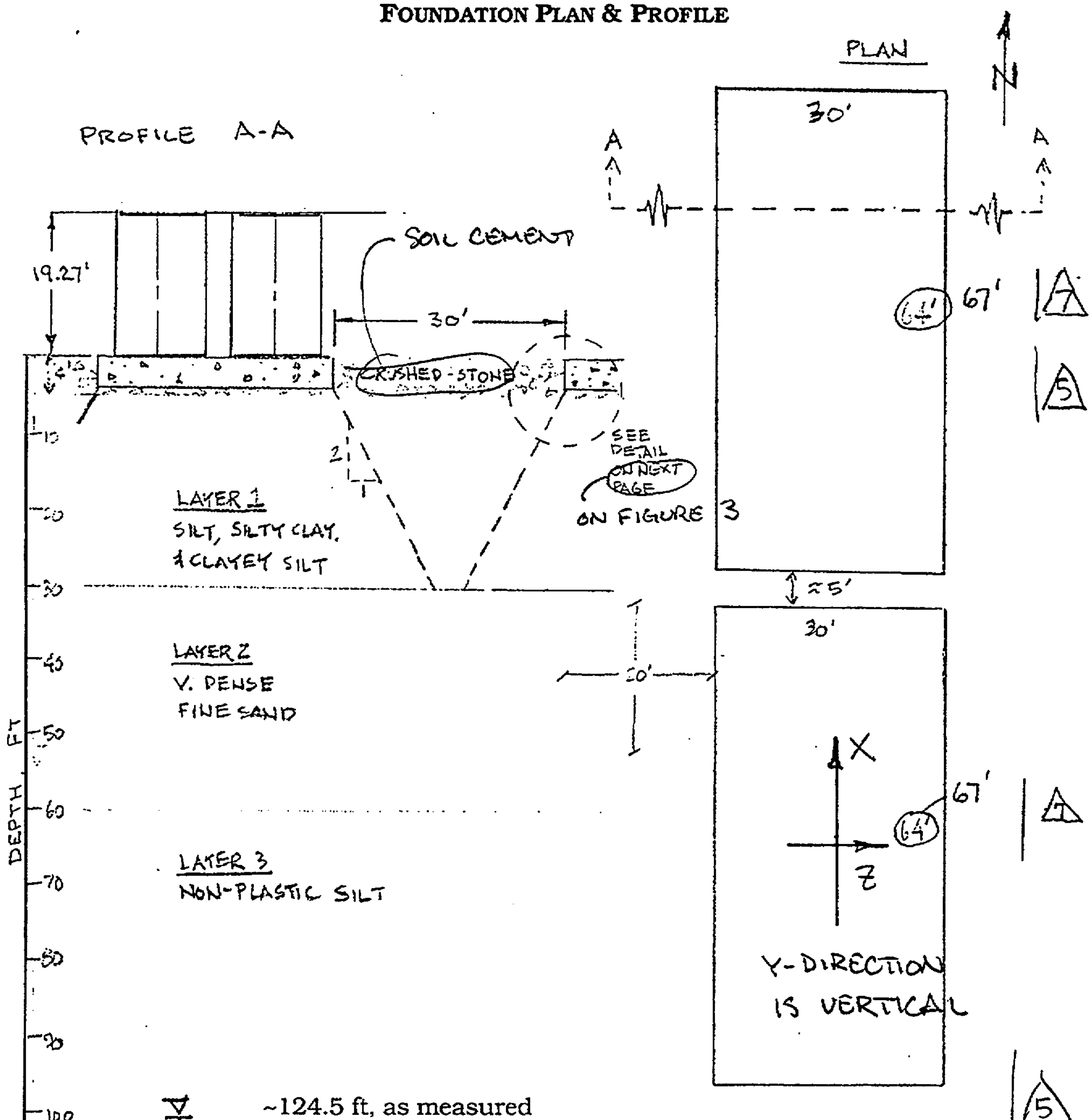
FIGURE 1

FOUNDATION PLAN & PROFILE



~124.5 ft, as measured in Observation Well in Boring CTB-5 OW

Note: Plan view of pad from SWEC Drawing 0599601-EY-2E.
Cask details from Attachment C of Calc 05996.02-G(B)-05-1.

FIGURE 2

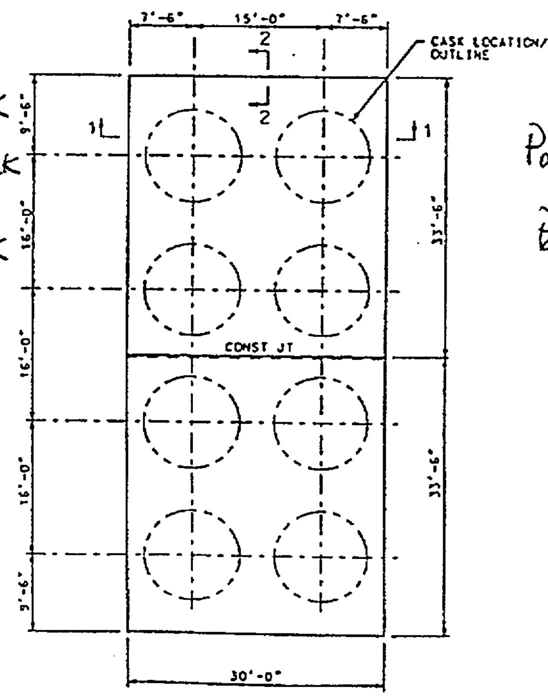
STATIC FOUNDATION LOAD / PRESSURE

Total Load

8 Casks @ $356.5^K = 2852^K$

$30' \times 67' \times 3' \times 0.15\,^K/_{ft^3} = 904.5^K$

$\therefore$ Total Load $= 3756.5^K$



PLAN
CASK STORAGE PAD
(500 REQD)

Bearing Pressure:

P_{actual} = Load/Area

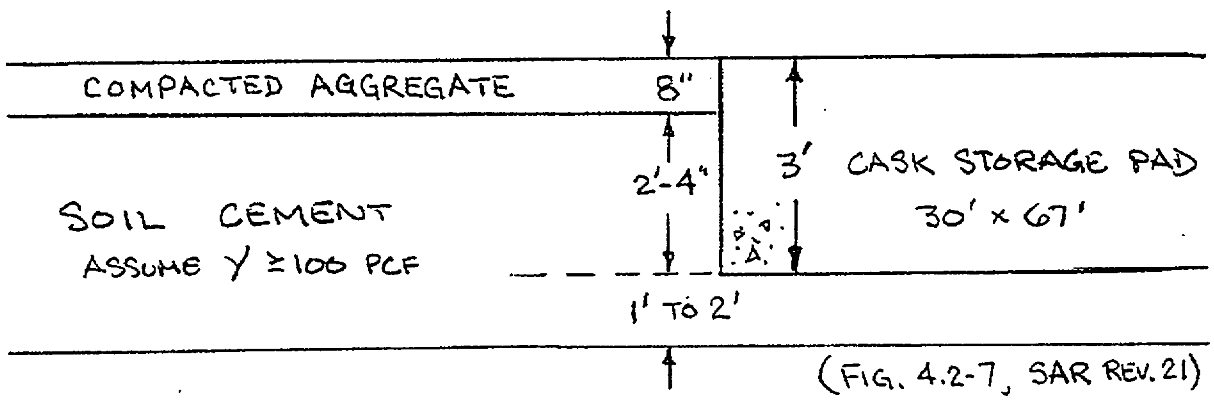
$P_{actual} = 3756.5 / 30' \times 67'$

$P_{actual} = 1.87$ KSF

Cask weight = 356.5K based on heaviest assembly weight shown on HI-STORM TSAR Table 3.2.1 (overpack with fully loaded MPC-32). See p C3 of Calc 05996.02-G(B)-05-1 for copy.

FIGURE 3

DETAIL OF SOIL CEMENT UNDER & ADJACENT TO CASK STORAGE PADS

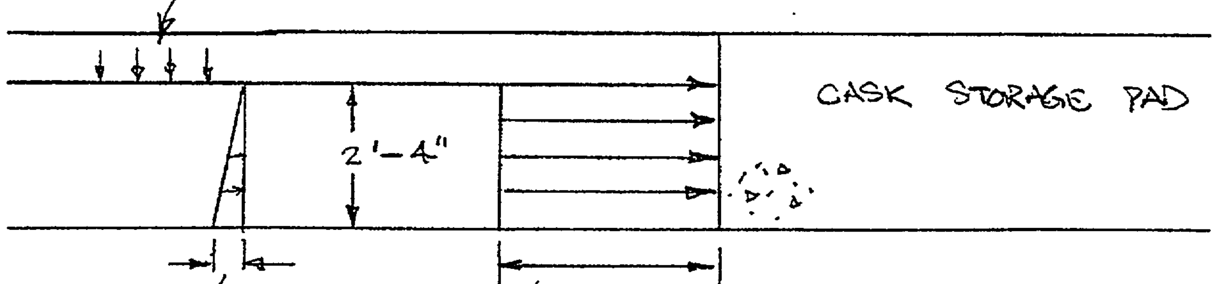


(FIG. 4.2-7, SAR REV. 21)

FIGURE 4

PASSIVE PRESSURE ACTING ON CASK STORAGE PADS

$$q_s = 0.125 \frac{K}{FT^3} \times \frac{8\ IN.}{12\ IN./FT} = 0.08\ KSF$$



$$p_p = \gamma H K_p$$

$$p_p = (2\bar{c} + q_s) H \sqrt{K_p}$$

$$= (2\bar{c} + 0.08)\ KSF \times 2.33' \quad \text{FOR } \phi = 0, \ K_p = 1.0$$

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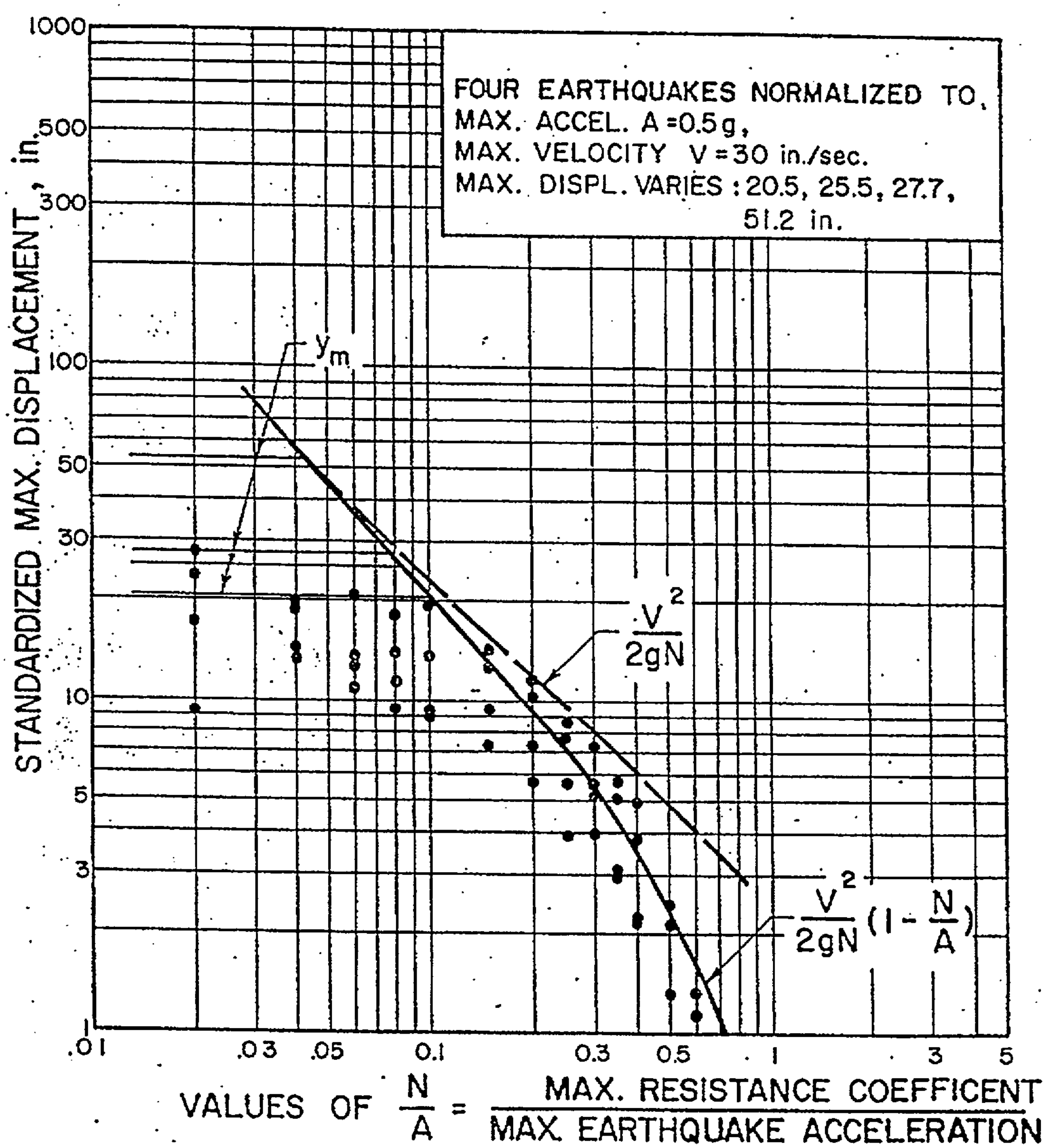
FIGURE 5

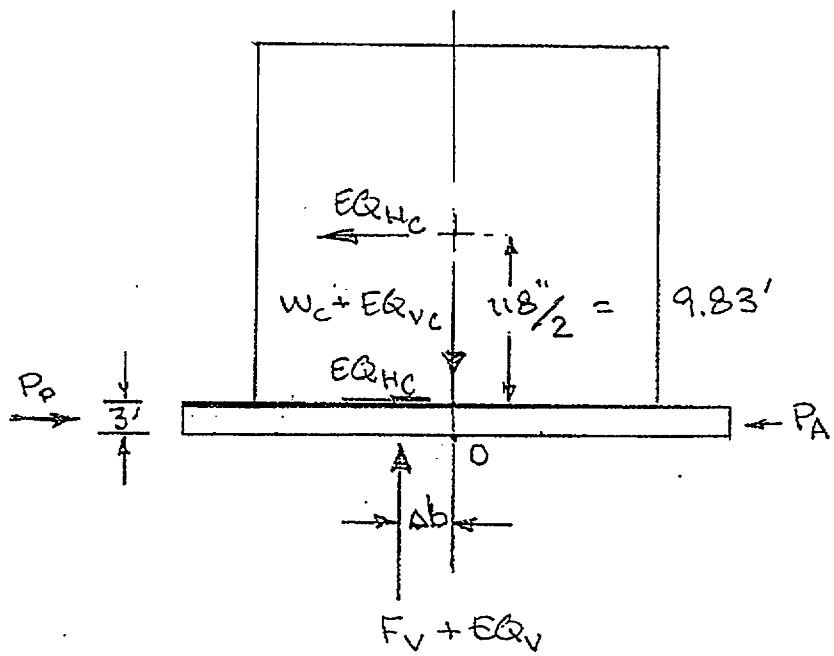
STANDARDIZED DISPLACEMENT FOR NORMALIZED EARTHQUAKES (SYMMETRICAL RESISTANCE)



From Newmark (1965)

FIGURE 6

DETERMINATION OF MOMENTS ACTING ON PAD DUE TO EARTHQUAKE LOADS FROM CASKS



$P_A << P_P$; therefore, it's conservative to ignore both in ΣM.

Vertical reaction of cask load acts on the pad at an offset = Δb from the centerline of the cask.

$$\sum M_{@\,\text{centerline}} \text{ to find } \Delta b.$$

$$\Delta b \times (W_c + EQ_{VC}) = 9.83\text{ ft} \times EQ_{HC}$$

$$\sum M_{@O} \text{ to find } \sum M_{@N\text{-}S}$$

$$\sum M_{@N\text{-}S} = \underbrace{1.5\text{ ft} \times EQ_{HP}}_{\text{pad}} + \underbrace{3\text{ ft} \times EQ_{HC}}_{\text{cask horiz}} + \underbrace{\Delta b \times (W_c + EQ_{VC})}_{\text{cask vert}}.$$

Note: Moment arm of 3 ft is used for determining moment due to cask horizontal force, because casks are only resting on the pads — No connection exists to transmit moment to the pad.

ED JUN 1 2 2000 P. J. Trudeau

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

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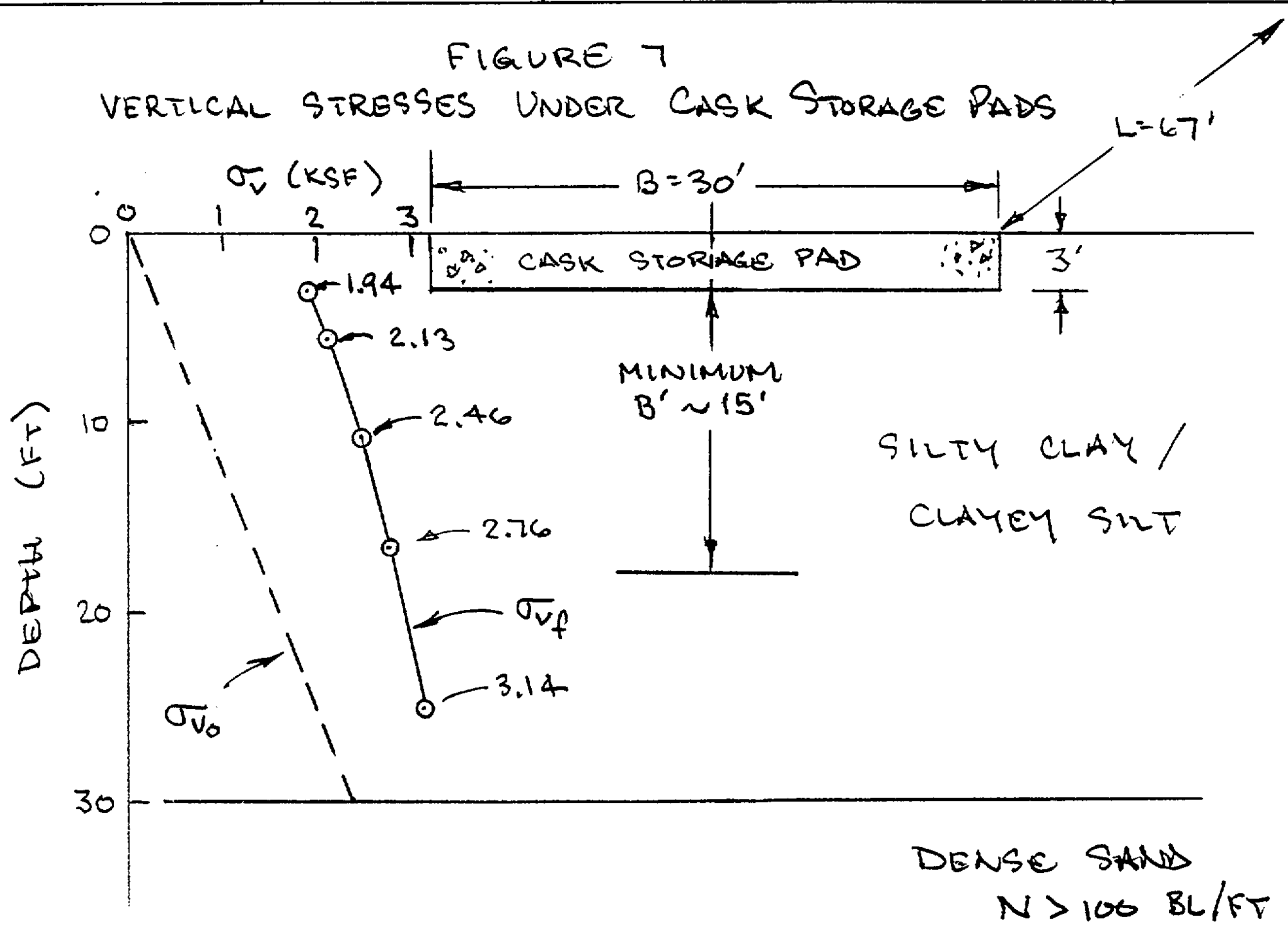
FIGURE 7

VERTICAL STRESSES UNDER CASK STORAGE PADS



NOTE: AVG γ_m ~ 80 PCF FOR SOILS THAT WERE TESTED UNDER THE PAD EMPLACEMENT AREA, AS INDICATED ON p 4 OF CALC 05996.02-G(B)-05-1.

REF: CALC 05996.02-G(B)-03-3:

TABLE 3 => σ_{vf} AS $f(z)$

FIG 1 => σ_v AT BOTTOM OF 3' PAD = 1.935 KSF

FIG 7 => SUBLAYERS USED IN DET'G σ_v AS $f(z)$

STONE & WEBSTER, INC.
CALCULATION SHEET

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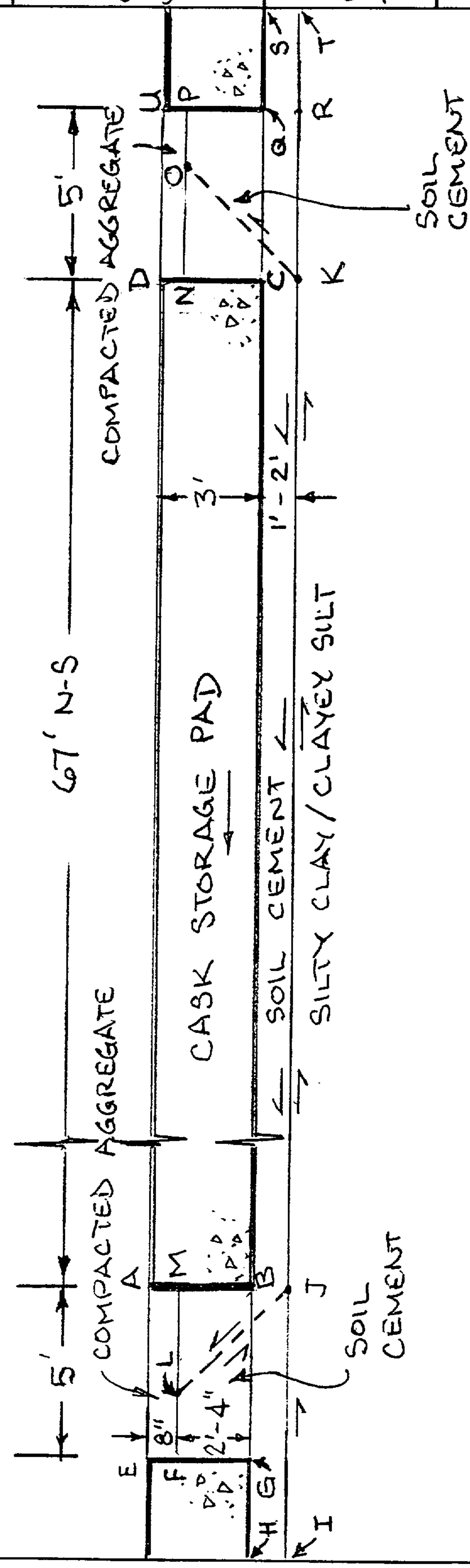
FIGURE 8

ELEVATION VIEW OF COLUMN OF CASK STORAGE PADS - LOOKING EAST



ATTACHMENT A TO CALC 05996.02-G(B)-04-9 p A1/1

NOTES OF TELEPHONE CONVERSATION

JO No. 05996.01

PRIVATE FUEL STORAGE, LLC
PRIVATE FUEL STORAGE FACILITY

Date: 06-19-97
Time: 2:45 PM EDT

FROM:	Stan M. Macie	SWEC-Denver 1E		Tie Line 321-7305
	Wen Tseng	(ICEC)	Voice	(510) 841-7328
			(FAX)	(510) 841-7438
TO:	Paul J. Trudeau	SWEC-Boston 245/03		(617) 589-8473

SUBJECT: DYNAMIC BEARING CAPACITY OF PAD

DISCUSSION:

WTseng reported that his pad design analyses are being prepared for three loading cases: 2 casks, 4 casks, and 8 casks. The dynamic loads that he is using are based on the forcing time histories he received from Holtec. These forcing time histories were developed using a coefficient of friction between the cask and the pad of 0.2 and 0.8, where 0.2 provides the lower bound and 0.8 provides the upper bound loads from the cask to the pad.

He indicated that the bearing pressures at the base of the pad are greatest for the 2-cask dynamic loading case for $\mu = 0.8$ between the cask and the pad, because of eccentricity of the loading. For this case, the vertical pressures at the 30' wide loaded end of the pad are 5.77 ksf at one corner and 3.87 ksf at the other. He reported that it is reasonable to assume this pressure decreases linearly to 0 at a distance of ~32 ft; i.e., approximately half of the pad is loaded in this case. He also indicated that the horizontal pressure at the base of the pad is 1.04 ksf at the 30' wide end of the pad that is loaded by the 2 casks, and that this pressure decreases linearly over a distance of ~40' from the loaded end. He noted that the vertical pressures include the loadings (DL + dynamic loadings) of the casks and the pad, but the horizontal pressures apply only to the casks. Therefore, the inertia force of the whole pad must be added to the horizontal loads calculated based on the horizontal pressure distribution described above.

Since the table of allowable bearing pressures as a function of coefficient of friction between the cask and the pad that is in the design criteria does not include a value for $\mu = 0.8$, WTseng asked PJTrudeau to provide the allowable bearing pressure for this case.

SUPERSEDED BY ATT B 3

ACTION ITEMS:

PJTrudeau to determine the dynamic allowable bearing pressure for the 2-cask loading case.

COPY TO: NTGeorges Boston 245/03
SMMacie Denver 1E

~~00901~~

CEC

CALCULATION SHEET

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5.3 Soil Pressures

5.3.1 Static Soil Pressure

Calculations of static soil pressure due to dead load (DL) and cask live load (LL) are given in Table S-1 and S-2, respectively.

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Table S-1
Maximum Vertical Displacements and Soil Bearing Pressures
Dead Load

	k_s = 2.75 kcf	k_s = 26.2 kcf
Z_w(ft) =	0.164	0.017
q_{zw}(ksf) =	0.45	0.45

Notes:

1. Z_w = maximum vertical displacement due to dead load (wt. of the pad only) obtained from CECSAP analysis results.
2. q_{zw} = vertical soil bearing pressure = k_s x Z_w, where k_s = subgrade modulus=2.75 and 26.2 kcf for lower-bound and upper-bound soils,respectively.

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Table S-2
Maximum Vertical Displacements and Soil Bearing Pressures
Live Load

Node No.	(Z_i)max (x10^{-2} ft.)							
	subgrade modulus = 2.75 kcf				subgrade modulus = 26.2 kcf			
	2 Casks	4 Casks	8 Casks	7 Casks + OLT	2 Casks	4 Casks	8 Casks	7 Casks + OLT
1	13.06	11.29	-50.97	-57.81	0.61	1.16	-4.83	-5.30
7	13.02	11.28	-50.97	-41.84	0.59	1.14	-4.84	-4.42
13	13.06	11.29	-50.97	-25.83	0.61	1.16	-4.83	-3.50
144	-11.82	-26.36	-52.73	-78.21	-0.70	-2.89	-5.78	-7.95
150	-11.93	-26.35	-52.71	-61.06	-0.76	-2.89	-5.79	-6.31
156	-11.82	-26.36	-52.71	-43.87	-0.70	-2.89	-5.78	-4.65
287	-42.54	-62.26	-50.97	-100.20	-5.13	-5.98	-4.83	-11.81
293	-42.59	-62.25	-50.97	-80.88	-5.16	-5.98	-4.84	-8.48
299	-42.54	-62.26	-50.97	-61.84	-5.13	-5.98	-4.83	-5.47
	Maximum Soil Bearing Pressure q_{zi}(1) (ksf)							
1	0	0	-1.402	-1.590	0	0	-1.264	-1.390
7	0	0	-1.402	-1.151	0	0	-1.267	-1.159
13	0	0	-1.402	-0.710	0	0	-1.264	-0.917
144	-0.325	-0.725	-1.450	-2.151	-0.185	-0.757	-1.514	-2.082
150	-0.328	-0.725	-1.450	-1.679	-0.199	-0.758	-1.516	-1.653
156	-0.325	-0.725	-1.450	-1.206	-0.185	-0.757	-1.514	-1.219
287	-1.170	-1.712	-1.402	-2.756	-1.345	-1.567	-1.264	-3.094
293	-1.171	-1.712	-1.402	-2.224	-1.352	-1.565	-1.267	-2.222
299	-1.170	-1.712	-1.402	-1.701	-1.345	-1.567	-1.264	-1.434

Notes:

1. $q_{zi} = k_s \times Z_i$, where k_s = 2.75 and 26.2 kcf for lower-bound and upper-bound subgrade moduli, respectively, and Z_i are obtained from CECSAP analysis results (Att. A)
2. Negative displacements imply downward movements.
3. The locations of nodes listed are shown in Figure 5.1-1.
4. For snow load, the soil bearing pressures is .045 ksf (Ref. 11).

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5.3.2 Dynamic Horizontal and Vertical Soil Pressures

Calculations of lateral and vertical soil pressures due to dynamic cask loadings resulting from 2000-year event earthquake are given in the following tables:

Table D-1(a) shows calculation of horizontal dynamic soil pressures in the X-direction (short direction of pad).

Table D-1(b) shows calculation of horizontal dynamic soil pressures in the Y-direction (long direction of pad).

Table D-1(c) shows a summary of averaged horizontal dynamic soil reactions.

Table D-1(d) shows calculation of vertical dynamic soil pressures.

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Table D-1(a)
Averaged Maximum Horizontal Soil Reactions in the X Direction
Dynamic Load

Node No.	Maximum Displacement Xd ($x10^{-3}$ ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	3.512	2.409	17.160	1.624	1.177	9.076	0.798	0.547	3.597
7	3.515	2.405	17.180	1.625	1.170	9.085	0.801	0.552	3.625
13	3.512	2.409	17.190	1.624	1.177	9.060	0.799	0.550	3.618
144	4.461	9.712	17.460	2.021	4.241	9.127	1.017	2.325	3.952
150	4.461	9.729	17.470	2.021	4.242	9.156	0.999	2.294	3.951
156	4.467	9.733	17.470	2.029	4.244	9.171	0.982	2.272	3.947
287	12.800	21.490	17.510	6.201	9.504	8.860	3.345	5.306	4.514
293	12.800	21.490	17.530	6.186	9.512	8.886	3.360	5.341	4.566
299	12.800	21.470	17.530	6.173	9.516	8.886	3.381	5.349	4.565
Avg =	6.925	11.205	17.389	3.278	4.976	9.034	1.720	2.726	4.037
Kxd =	1.14E+05	1.14E+05	1.14E+05	2.33E+05	2.33E+05	2.33E+05	5.48E+05	5.48E+05	5.48E+05
Qxd =	789	1277	1982	764	1159	2105	943	1494	2212

Notes:
1. Avg = {sum $(Xd)_i$}/N; Xd = max. x-displ.; i = nodes 1, 7, 13, 144, 150, 156, 287, 293, 299; and N = 9.
2. Qxd = Kxd x Avg = averaged maximum horizontal-x soil reaction in Kips due to dynamic loading.
3. Kxd for LB, BE, and UB soils are dynamic horizontal-x soil spring stiffnesses given below:

(Kxd)LB =	9.51E+06 lb/in 1.14E+05 Kips/ft	(Kxd)BE =	1.94E+07 lb/in 2.33E+05 Kips/ft	(Kxd)UB =	4.57E+07 lb/in 5.48E+05 Kips/ft

4. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.
5. Xd are obtained from CECSAP analysis results given in Att. A.

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Table D-1(b)
Averaged Maximum Horizontal Soil Reactions in the Y Direction
Dynamic Load

Node No.	Max. Displacement Yd ($x10^{-3}$ ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	5.107	8.657	13.550	2.194	4.059	8.393	1.413	2.578	3.979
7	3.916	7.318	14.030	2.055	4.313	8.173	1.195	1.962	4.056
13	4.303	7.097	14.510	2.567	4.664	7.937	1.337	2.161	4.109
144	5.231	8.763	13.450	2.332	4.187	8.430	1.513	2.714	3.975
150	3.946	7.447	13.960	2.122	4.429	8.132	1.267	2.133	4.042
156	4.379	7.207	14.450	2.690	4.767	7.834	1.442	2.301	4.121
287	5.389	8.870	27.260	2.449	4.357	8.396	1.651	2.821	3.926
293	4.016	7.584	13.840	2.253	4.556	8.048	1.464	2.380	4.013
299	4.476	7.253	14.370	2.877	4.846	7.795	1.657	2.334	4.097
Avg =	4.529	7.800	15.491	2.393	4.464	8.126	1.438	2.376	4.035
Kyd =	1.08E+05	1.08E+05	1.08E+05	2.21E+05	2.21E+05	2.21E+05	5.21E+05	5.21E+05	5.21E+05
Qyd =	491	846	1680	528	986	1794	749	1237	2102

Notes:
1. Avg = {sum (Yd)i}/N; Yd = max. y-displ.; i = nodes 1, 7, 13, 144, 150, 156, 287, 293, 299; and N = 9.
2. Qyd = Kyd x Avg = averaged maximum horizontal-y soil reaction in Kips due to dynamic loading.
3. Kyd for LB, BE, and UB soils are dynamic horizontal-y soil spring stiffnesses given below:

(Kyd)LB =	9.04E+06 lb/in 1.08E+05 Kips/ft	(Kyd)BE =	1.84E+07 lb/in 2.21E+05 Kips/ft	(Kyd)UB =	4.34E+07 lb/in 5.21E+05 Kips/ft

4. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.
5. Yd are obtained from CECSAP analysis results given in Att. A.

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Table D-1(c)
Summary of Total Maximum Horizontal Soil Reactions
Dynamic Load

	Max. Soil Reaction (Kips)									
	LB			BE			UB			
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	
Qxd =	789	1277	1982	764	1159	2105	943	1494	2212	E-W
Qyd =	491	846	1680	528	986	1794	749	1237	2102	N-S

Notes:
1. Qxd, and Qyd shown are obtained from Tables D-1(a), and (b), respectively.
2. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.

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			CALC. NO. G(PO17)-2	REV. NO.	3
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Table D-1(d)
Maximum Vertical Soil Bearing Pressures
Dynamic Load

Node No.	Maximum Displacement Zd (x10^{-3} ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	4.051	9.396	-31.02	1.806	4.158	-23.66	0.406	1.654	-15.92
7	3.900	7.973	-24.23	1.964	3.648	-21.18	0.439	1.024	-13.36
13	4.788	11.470	-31.22	2.115	4.636	-17.88	0.528	1.560	-15.31
144	-9.195	-22.58	-34.05	-5.939	-16.84	-22.66	-1.861	-8.34	-13.66
150	-5.063	-15.2	-12.71	-3.683	-11.13	-12.39	-1.332	-6.698	-8.016
156	-6.565	-15.9	-32.24	-2.988	-9.447	-18.42	-1.734	-5.773	-14.53
287	-29.18	-24.39	-17.51	-14.54	-15.67	-18.88	-12.72	-8.52	-8.38
293	-15.57	-16.97	-19.21	-9.019	-12.42	-12.22	-12.08	-10.68	-6.446
299	-21.85	-26.09	-28.04	-12.87	-16.35	-17.02	-9.835	-11.63	-13.12
	Maximum Soil Bearing Pressure q_{zd} (Kips/ft^2)								
1	0	0	-2.22	0	0	-3.35	0	0	-5.14
7	0	0	-1.74	0	0	-3.00	0	0	-4.32
13	0	0	-2.24	0	0	-2.53	0	0	-4.94
144	-0.66	-1.62	-2.44	-0.84	-2.38	-3.21	-0.60	-2.69	-4.41
150	-0.36	-1.09	-0.91	-0.52	-1.57	-1.75	-0.43	-2.16	-2.59
156	-0.47	-1.14	-2.31	-0.42	-1.34	-2.61	-0.56	-1.86	-4.69
287	-2.09	-1.75	-1.25	-2.06	-2.22	-2.67	-4.11	-2.75	-2.71
293	-1.12	-1.22	-1.38	-1.28	-1.76	-1.73	-3.90	-3.45	-2.08
299	-1.57	-1.87	-2.01	-1.82	-2.31	-2.41	-3.18	-3.76	-4.24

Notes:
1. q_{zd} = maximum soil bearing pressure = (Kzd x Z_d)/A, where A = 67' x 30' = 2010 ft^2.
2. Kzd for LB, BE, and UB soils are vertical-z dynamic soil spring stiffnesses given below:

(Kzd)LB =	1.20E+07 lb/in	(Kzd)BE =	2.37E+07 lb/in	(Kzd)UB =	5.41E+07 lb/in
	1.44.E+05 Kips/ft		2.84.E+05 Kips/ft		6.49.E+05 Kips/ft

3. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.
4. Zd are obtained from CECSAP analysis results given in Att. A.
5. Negative displacements imply downward movements.
6. The maximum values of Zd shown may not be concurrent. However, they are assumed to be concurrent values and concurrent signs are assigned to them.
7. Node numbers are shown in Figure 5.1-1.

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6.2 Vertical Soil Bearing Pressures and Horizontal Soil Shear Stresses

Vertical soil bearing pressures for individual loadings and combined loadings are Summarized in Table 4.

Horizontal soil shear stresses are shown in Tables D-1(a) and (b), and the total horizontal soil reactions (shear forces) in both the short (x) and long (y) directions of the pad are summarized in Table D-1(c).

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			CALC. NO. G(PO17)-2	REV. NO.	3
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PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
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Table 4
Summary of Vertical Soil Bearing Pressures (ksf)

Loading	Point	A 287	B 293	C 299	D 144	E 150	F 156	G 1	H 7	J 13
2 - Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.345	1.352	1.345	0.185	0.199	0.185	0	0	0
	Pad EQ	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EQ	4.11	3.9	3.18	0.84	0.52	0.56	0	0	0
	100% Vert	6.26	6.06	5.33	1.83	1.53	1.55	0.81	0.81	0.81
4-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.712	1.712	1.712	0.757	0.758	0.757	0	0	0
	Pad EQ	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EQ	2.75	3.45	3.76	2.69	2.16	1.86	0	0	0
	100% Vert	5.27	5.97	6.28	4.25	3.73	3.42	0.81	0.81	0.81
8-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.402	1.402	1.402	1.514	1.516	1.514	1.402	1.402	1.402
	Pad EQ	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EQ	2.71	2.08	4.24	4.41	2.59	4.69	5.14	4.32	4.94
	100% Vert	4.92	4.29	6.45	6.73	4.91	7.01	7.35	6.53	7.15

Notes:
1. Values for Pad DL are obtained from Table S-1.
2. Values for snow LL are obtained from Table S-2.
3. Values for Cask LL are obtained from Table S-2.
4. Pad EQ pressure = (pad wt.)xa_v, where pad wt.=904.5 kips, and a_v=.695g.
5. Values for Cask EQ are obtained from Table D-1(d).
6. EQ pressures listed are the envelopes of results for all soil conditions.
7. Node numbers are shown in Figure 5.1-1.

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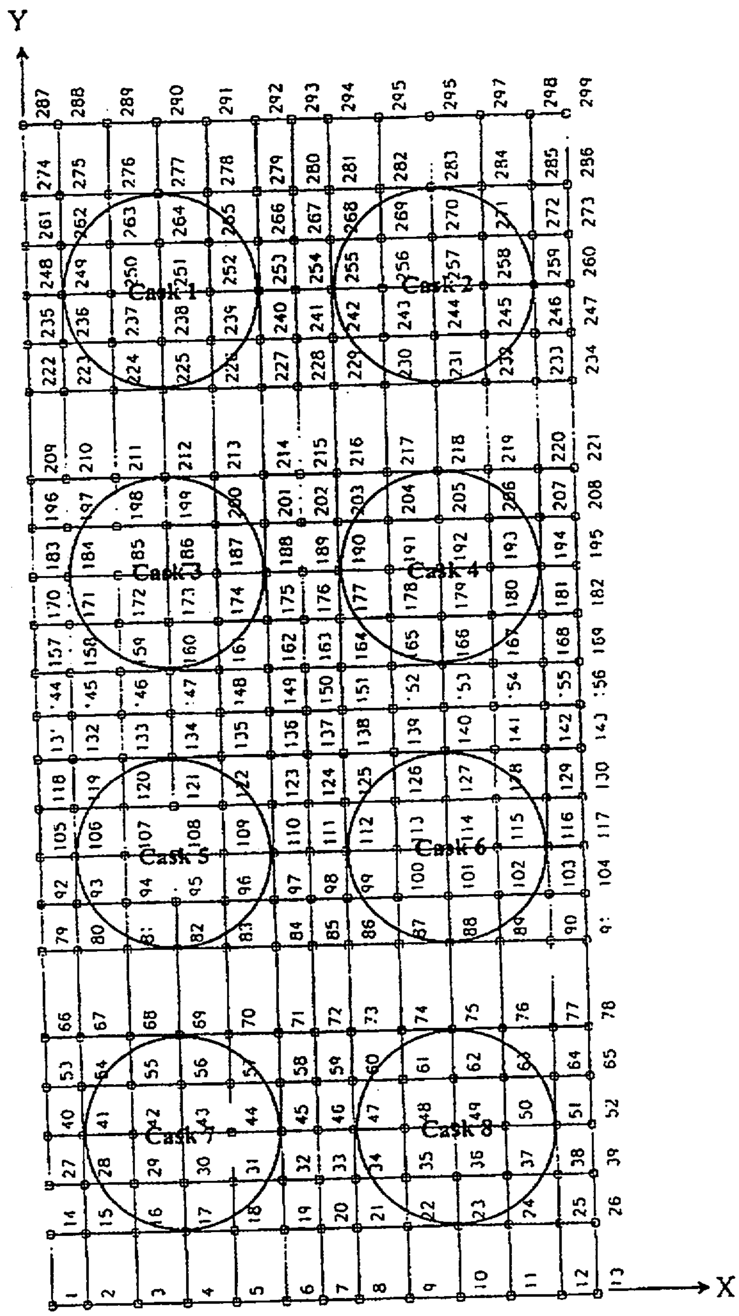

Figure 5.1-1 CECSAP Finite-Element Model with Node Numbers

fig5 1-1

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SUBJECT Storage Pad Analysis and Design			SHEET NO. 2.1

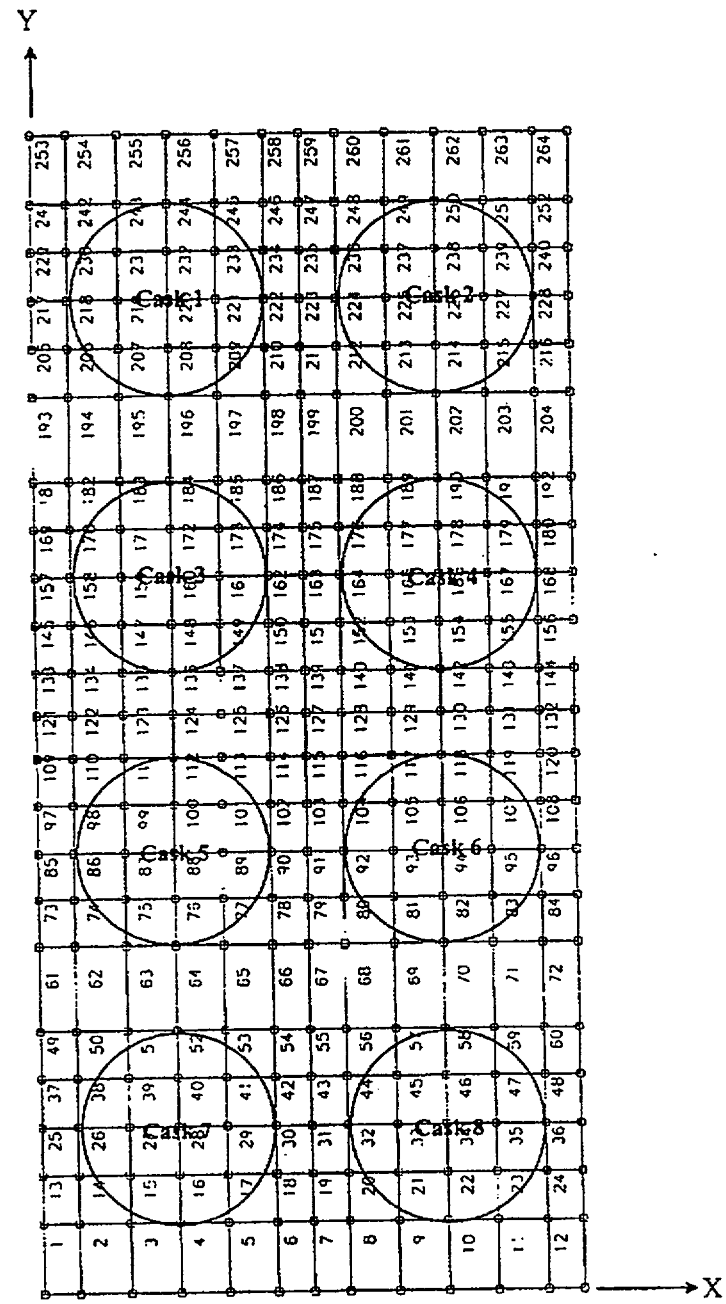

Figure 5.1-2 CECSAP Finite-Element Model with Element Numbers

CALCULATION COVER SHEET

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JOB NO. 1101-000
FILE NO.
CALC NO. G(PO17)-2
NO. OF SHEETS 289

PROJECT Private Fuel Storage Facility (PFSF)
SUBJECT Storage Pad Analysis and Design

RECORD OF ISSUES

NO.	DESCRIPTION	BY	DATE	CHKD	DATE	APPRD	DATE
0	Initial Issue	msy DH awm KL	10/18/99	msy DH awm KL	10/18/99	WT	10/18/99
1	Revision 1 (see notes below)	DH KL	12/6/99	DH KL	12/6/99	WT	12/6/99
2	Revision 2 (see notes below)	DH	2/4/00	msy	2/4/00	WT	2/4/00
3	Revision 3 (see notes on Sheet ii)	awm KL msy DH	4/5/01	awm msy KL DH	4/5/01	WT	4/5/01

☒ Nuclear Quality Assurance Category ☐ Non-Nuclear Quality Assurance Category

This set of calculations documents the engineering analyses and detailed calculations required for structural design of the reinforced-concrete spent-fuel cask storage pads to be constructed at the Private Fuel Storage Facility (PFSF) project site.

This set of calculations has been prepared in accordance with CEC's quality assurance procedure for nuclear projects.

Revision 1 was made to correct (1) typographical errors on Pages 5, 29, and A-3 and (2) insert computer output file names and explanation notes on Pages 43 and 51.

Revision 2 was made to correct typographical errors and to include additional clarifications on Pages 17, 21, 28, 236, 298, and 312.

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CALCULATION COVER SHEET

		JOB NO.	1101-000
PROJECT	Private Fuel Storage Facility (PFSF)	FILE NO.	
SUBJECT	Storage Pad Analysis and Design	CALC NO.	G(PO17)-2
		SHEET	ii

Revision 3 was made to incorporate the following: (1) PGA of 0.711g and 0.695g for horizontal and vertical components of the new design ground motions, (2) Revised dynamic soil properties for lower-bound, best-estimate, and upper-bound soils provided by Geomatrix, (3) Revised cask force time-histories provided by Holtec, (4) Revised pad size to 30 ft by 67 ft with cask spacing in the long axis of the pad changed to 16 ft and cask spacing in the short axis of the pad remained at 15 ft, (5) Pad founded in soil cement with about 3 ft under the pad and 2 ft thick on its side walls, and (6) Revised transporter weight to 145 kips.

TABLE 6

SUMMARY OF TRIAXIAL TEST RESULTS FOR SOILS WITHIN ~10 FT OF GROUND SURFACE AT THE SITE

Boring	Sample	Depth ft	Elev ft	w %	ATTERBERG LIMITS LL	PL	PI	USC Code	γ_m pcf	γ_d pcf	e_o	σ_c ksf	s_u ksf	ε_a %	Type	Date
B-1	U-2C	5.9	4453.9	47.1	66.1	33.4	32.7	MH	79.3	53.9	2.15	0.0	2.03	1.7	CU	Nov '99
B-1	U-2B	5.3	4454.5	52.9	80.6	40.9	39.7	MH	70.8	46.3	2.67	1.0	2.21	6.0	CU	Nov '99
B-4	U-3D	10.4	4462.1	27.4	42.5	24.7	17.8	CL	85.5	67.1	1.53	1.3	2.18	4.0	UU	Jan '97
C-2	U-2D	11.1	4453.4	35.6	See U-2C & E[1]			CL	78.5	57.9	1.93	1.3	2.39	11.0	UU	Jan '97
CTB-1	U-3D	8.7	4463.7	47.9	See U-3C[2]			CH	91.9	62.1	1.73	1.7	2.84	5.0	CU	June '99
CTB-4	U-2D	9.5	4465.5	45.2	See U-2E[2]			CH	87.7	60.4	1.81	1.7	3.11	6.0	CU	June '99
CTB-6	U-3D	8.3	4467.9	52.7				CH	85.7	56.2	2.02	1.7	2.70	7.0	CU	June '99
CTB-N	U-1B	5.7	4468.4	30.1	41.3	22.5	18.8	CL	100.6	77.3	1.20	1.7	3.00	8.0	CU	Nov '98
CTB-N	U-2B	7.7	4466.4	65.4	See U-2A[2]			MH	74.6	45.1	2.76	1.7	2.41	13.0	CU	June '99
CTB-N	U-3D	10.5	4463.6	52.2	61.1	30.8	30.3	CH	86.3	56.7	1.98	1.7	2.73	7.0	CU	June '99
CTB-S	U-1B	5.8	4468.7	73.6	66.2	40.9	25.3	MH	78.0	44.9	2.78	1.7	2.05	12.0	CU	Nov '98
CTB-S	U-2D	8.4	4466.1	54.6	57.9	28.9	29.0	CH	90.0	58.2	1.92	1.7	2.40	5.0	CU	June '99
B-1	U-2D	6.5	4453.3	45.2	59.8	34.7	25.1	MH	76.7	52.8	2.22	2.1	3.26	15.0	CU	Mar '99
B-3	U-1B	5.2	4463.0	33.5	52.4	25.2	27.2	MH	90.6	67.9	1.50	2.1	3.55	8.0	CU	Mar '99
C-2	U-1D	6.3	4458.2	50.5	70.3	41.3	29.0	MH	74.5	49.5	2.43	2.1	3.03	12.0	CU	Mar '99

NOTES 1 Attachment 2 of SAR Appendix 2A.
2 Attachment 6 of SAR Appendix 2A.

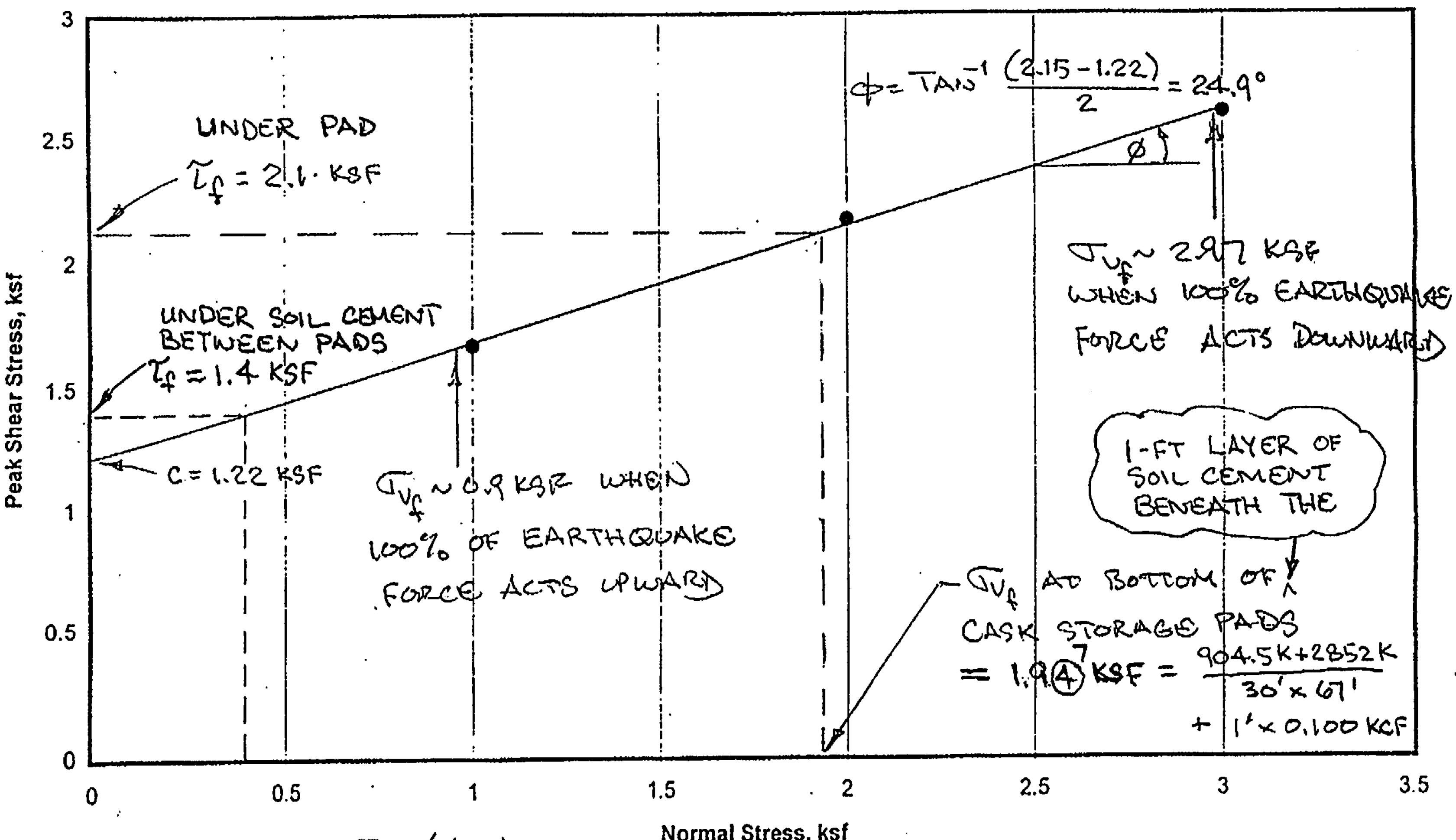
ATTACHMENT C p C1/3
CALC 05996.02-G(B)-04-9

FIGURE 7

DIRECT SHEAR TEST

Boring C-2, Sample U-1C

PAD EMPLACEMENT AREA



BETWEEN PADS $\bar{\sigma}_V \sim (3'+1')\ 0.100$ KCF $= 0.4$ KSF

REF SAR APP 2A ATT 7

ATTACHMENT C p C2

CALC 05996.02-G(B)-04-9

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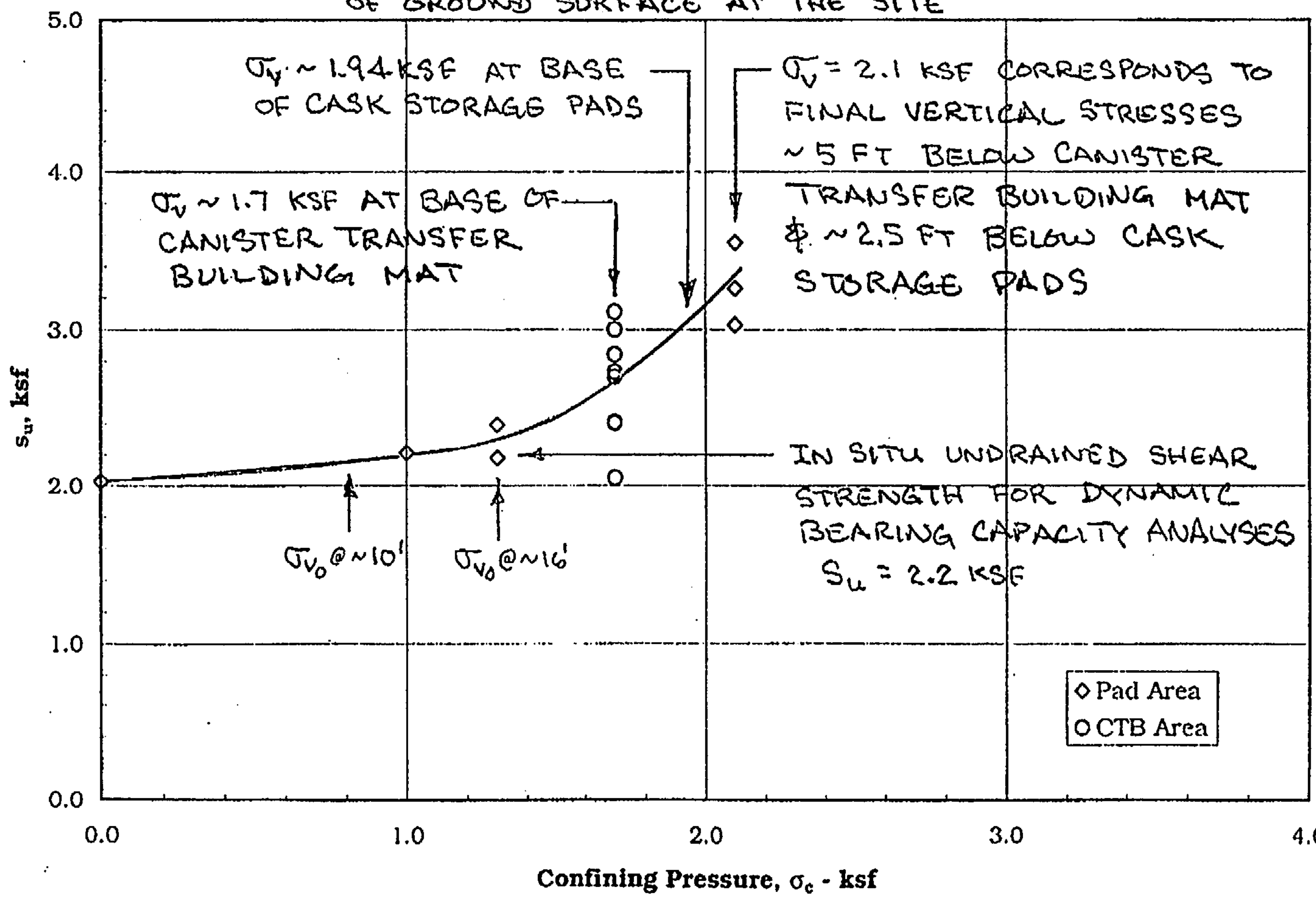
Figure 11
Summary of Triaxial Test Results for Soils Within Depth of ~10 ft
OF GROUND SURFACE AT THE SITE



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ATTACHMENT C p C3/3
CALC 05996.02-G(B)-04-9

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CALCULATION SHEET

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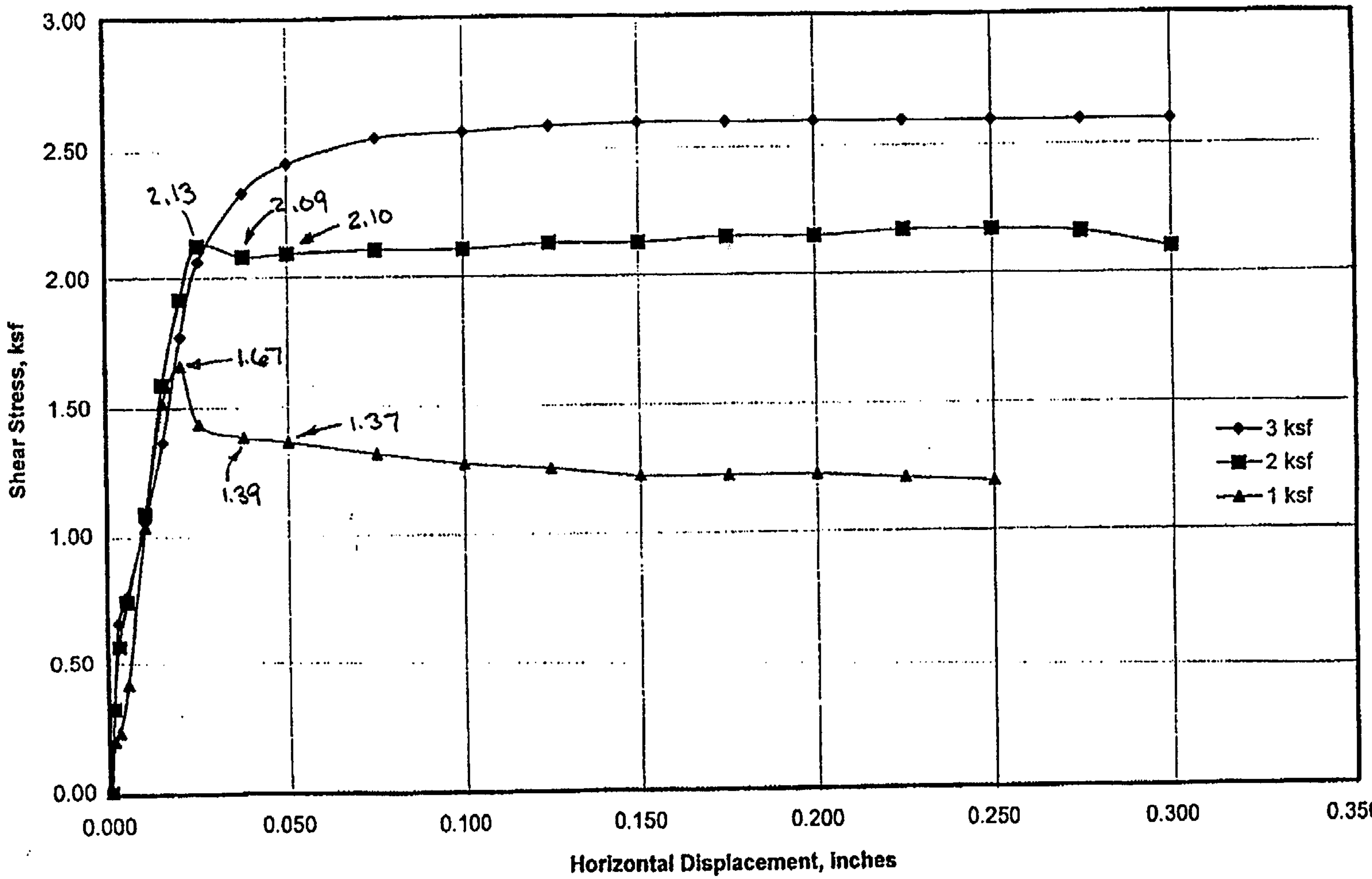
5010.65

DIRECT SHEAR TEST
Boring C-2, Sample U-1C



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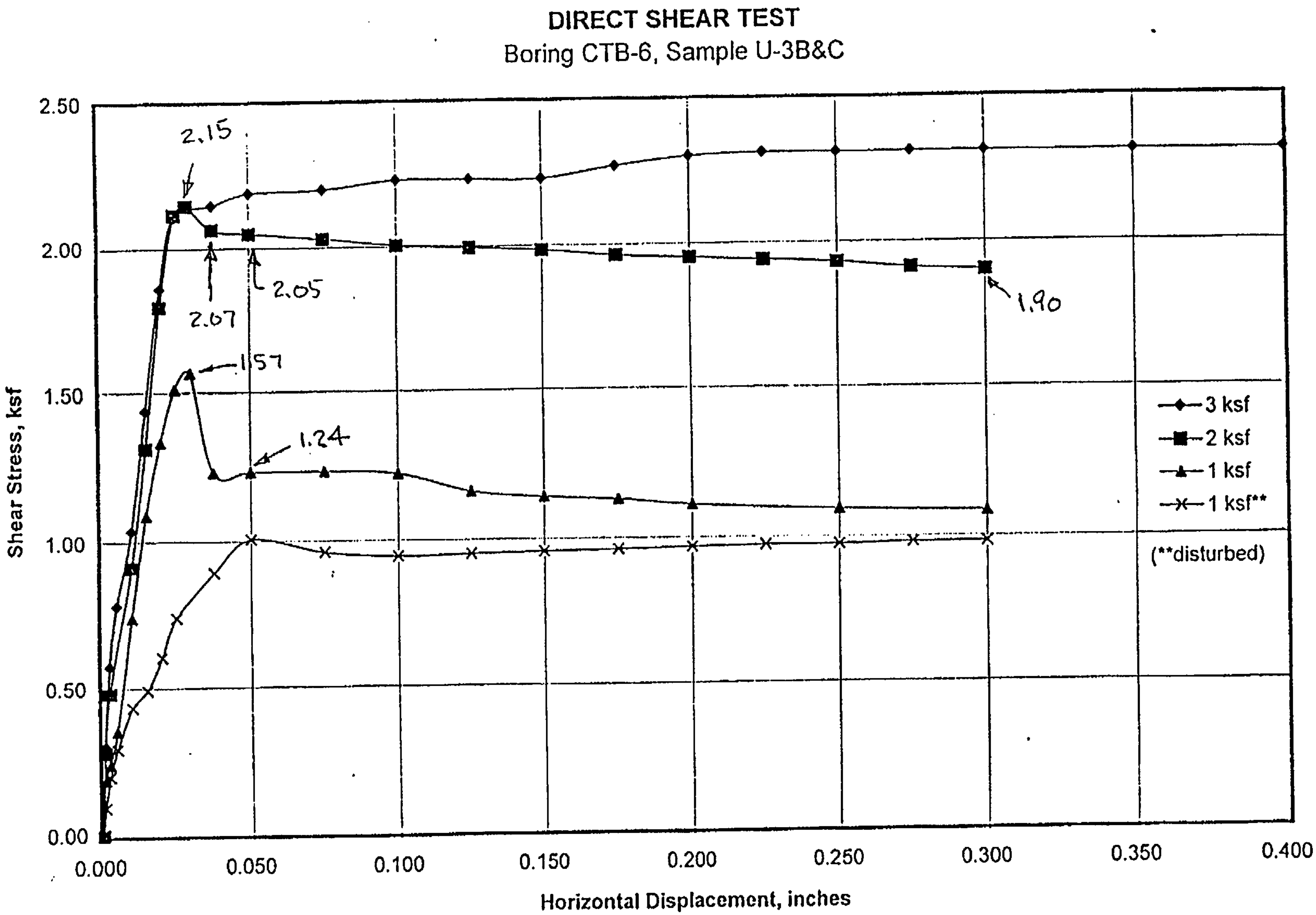

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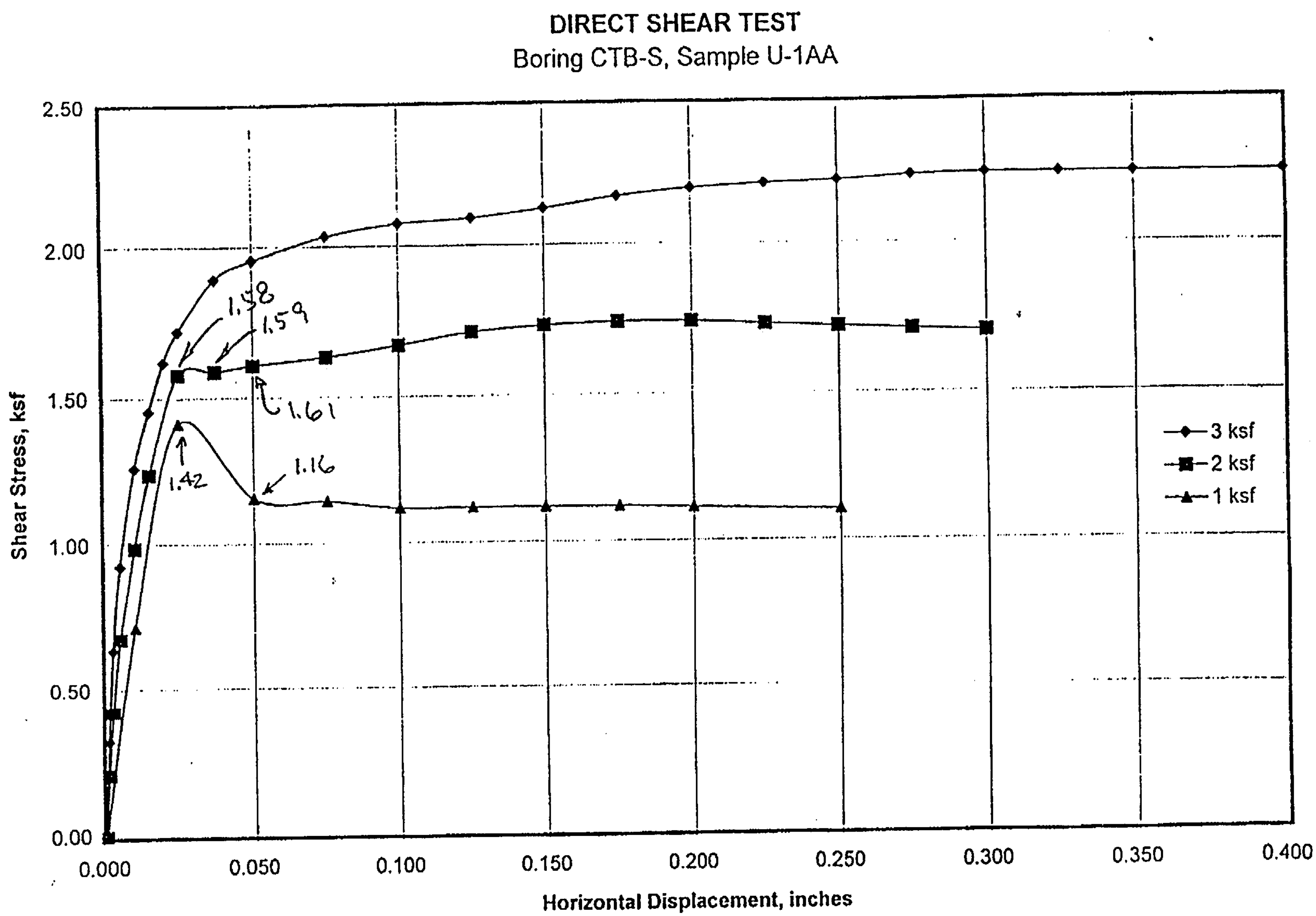
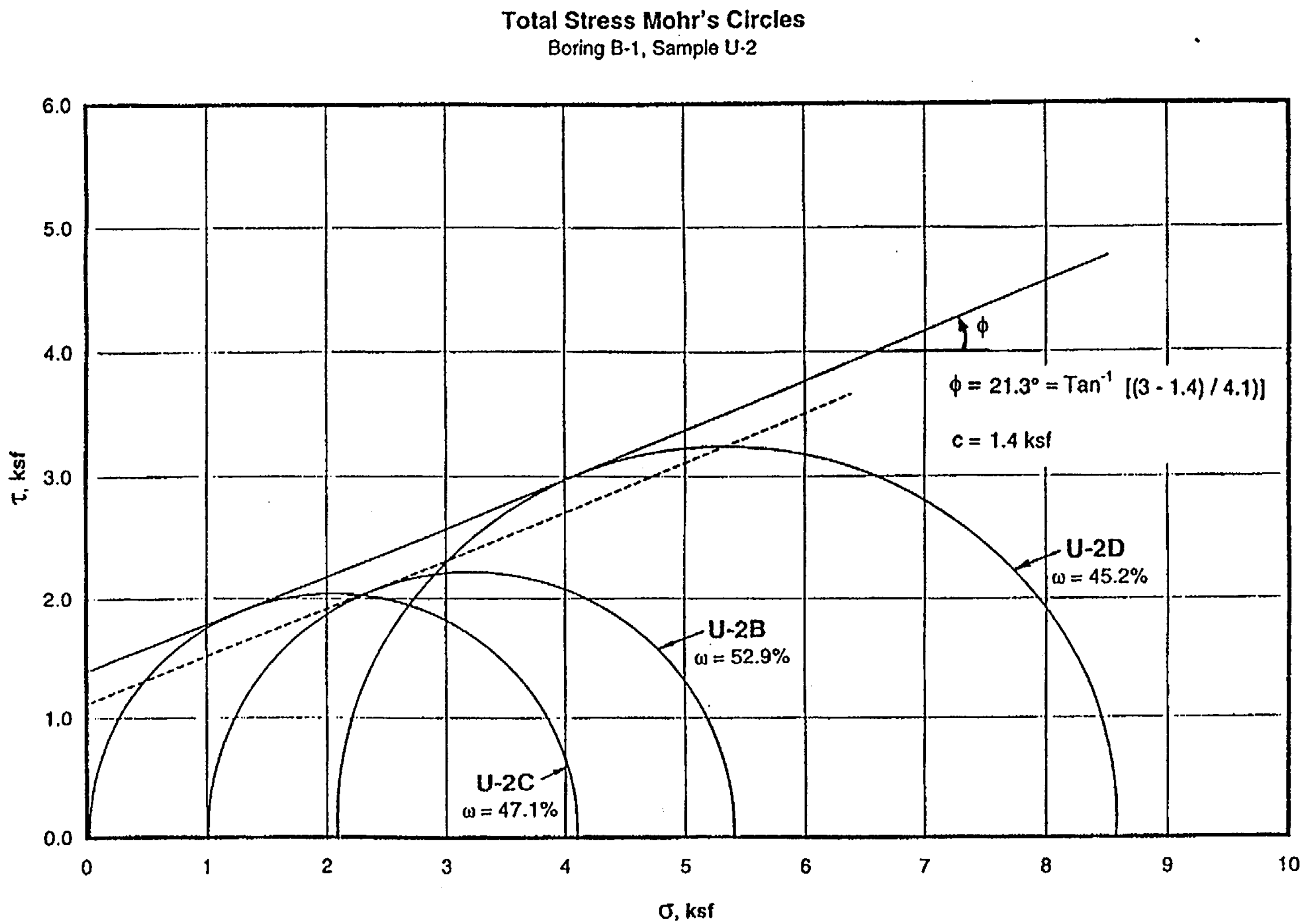


ATTACHMENT D TO CALC 05996.02-G(B)-04-9 p D3

STONE & WEBSTER ENGINEERING CORPORATION



Private Fuel Storage, LLC
PFSF, Skull Valley, UT
SAR APP 2A ATT 8

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JO 05996.02
November 1999

STONE & WEBSTER, INC.

5010.64

CALCULATION SHEET

CLIENT & PROJECT	PAGE 1 OF 59
PRIVATE FUEL STORAGE, LLC – PFSF	+ 6 pp of ATTACHMENTS

CALCULATION TITLE	QA CATEGORY (✓)
STABILITY ANALYSES OF CANISTER TRANSFER BUILDING	☑ 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)	13		

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: 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 *See page 2-1 for ID of*	Original Signed By: SYBoakye / 9-3-99 DLAloysius / 9-3-99 *Prepared / Reviewed By*	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 Paul J. Trudeau	TYChang / 7-26-01 Thomas Y. Chang	TYChang / 7-26-01 Thomas Y. Chang	6	5		✓

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RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1

Page count increased from 37 to 63.

- Revised seismic loadings to correspond to the PSHA 2,000-yr return period earthquake (p. 9-1)
- Added section on dynamic strength of soils (p. 9-3)
- Added section on seismic sliding resistance of the mat foundation (p. 9-5)
- Added section on evaluation of sliding on a deep slip surface (p. 9-8)
- Updated bearing capacity analysis using revised seismic loadings (p. 34-1)
  - Added additional loading combination: static + 40% seismic uplift + 100% in x (N-S) direction + 40% in z (E-W) direction
- Added additional references (p. 36-1)

NOTE:
SYBoakye prepared/DLAloysius reviewed pp. 9-8 through 9-12. Remaining pages prepared by DLAloysius and reviewed by SYBoakye.

REVISION 2

Major re-write of the calculation.

1. Renumbered pages and figures to make the calculation easier to follow.
2. Changed effective length of mat to 265 ft to make it consistent with Calculation 05996.02-SC-4, Rev 1 (SWEC, 1999a).
3. Added overturning analysis.
4. Corrected calculation of moments for joints 3 and 6 in Table 2.6-11 and incorporated revised seismic loads in calculations of overturning stability and dynamic bearing capacity.
5. Revised dynamic bearing capacity analyses to utilize only total strength parameters because these partially saturated soils will not have time to drain fully during the rapid cycling associated with the design basis ground motion. See Calculation 05996.02-G(B)-05-1 (SWEC, 1999b) for additional details.
6. Updated references to current issues of drawings.

7. Added references to foundation profiles through Canister Transfer Building area presented in SAR Figures 2.6-21 through 23.
8. Deleted analyses of bearing capacity on layered profile, as adequate factors of safety are obtained conservatively assuming that the total strengths measured for the clayey soils in the upper ~25' to 30' layer apply for the entire profile under the Canister Transfer Building and revised all of the detailed bearing capacity analyses.
9. Changed "Load Combinations" to "Load Cases" and defined these cases to be consistent throughout the various stability analyses included herein. These are the same cases as are used in the stability analyses of the cask storage pads, Calculation 05996.02-G(B)-04-5 (SWEC, 2000).
10. Added analysis of sliding on a deep plane at the top of silty sand/sandy silt layer, incorporating passive resistance acting on the block of clayey soil and the foundation mat overlying this interface.
11. Revised Conclusions to reflect results of these changes.

REVISION 3

1. Added a 1-ft deep key around the perimeter of the Canister Transfer Building mat to permit use of the cohesive strength of the in situ silty clay/clayey silt in resisting sliding due to loads from the design basis ground motion.
2. Revised shear strength used in the sliding stability analyses of the Canister Transfer Building mat supported on the in situ silty clay to be the strength measured in the direct shear tests performed on samples obtained from elevations approximately at the bottom of the 1-ft deep perimeter key. The shear strength used in this analysis equaled that measured for stresses corresponding to the vertical stresses at the bottom of the mat following completion of construction.
3. Removed static and dynamic bearing capacity analyses based on total-stress strengths.
4. The relative strength increase noted for the deeper lying soils in the cone penetration testing that was performed within the Canister Transfer Building footprint was used to determine a weighted average undrained strength of the soils in the entire upper layer for use in the bearing capacity analyses, since the soils within a depth equal to approximately the width of the foundation are effective in resisting bearing failures. This resulted in the average undrained strength for the bearing capacity analyses of the upper layer equal to 3.18 ksf.
5. Removed dynamic analyses based on increasing strengths of the cohesive soils that were measured in static tests to reflect well known phenomenon that the strength of cohesive soils increases as the rate of loading decreases.
6. Revised undrained shear strength of the clay block overlying the cohesionless layer to 2.2 ksf, based on the UU tests that were performed at confining pressures of 1.3 ksf (reported in Attachment 2 of Appendix 2A of the SAR) in the analysis of sliding of the Canister Transfer Building on deep plane of cohesionless soils.

7. Added shearing resistance available on the ends of the block of clay, since this soil must be sheared along these planes in order for the Canister Transfer Building to slide on a deep plane of cohesionless soils.
8. Revised method of calculating the inclination factor in the bearing capacity analyses to that presented by Vesic in Chapter 3 of Winterkorn and Fang (1975). Vesic's method expands upon the theory developed by Hansen for plane strain analyses of footings with inclined loads. 0Vesic's method permits a more rigorous analysis of inclined loads acting in two directions on rectangular footings, which more closely represents the conditions applicable for the Canister Transfer Building.
9. Replaced Tables 2, 2.6-9, and 2.6-10 with revised results for the changes in shear strength of the in situ soils noted above and deleted Table 3.

REVISION 4

1. Updated stability analyses to reflect revised design basis ground motions (a_H = 0.711g & a_V = 0.695g, per Table 1 of Geomatrix, 2001).
2. Resisting moment in overturning stability analysis calculated based on resultant of static and dynamic vertical forces.
3. Updated dimensions of foundation mat to 240 ft (E-W) x 279.5 ft (N-S), and changed the depth of the perimeter key to 1.5 ft, in accordance with design change identified in Figure 4.7-1 (3 sheets), "Canister Transfer Building," of SAR Revision 21 (based on S&W Drawings 0599602-EC-404A-B & 404B-B).
4. Added definition of "m" used in the inclination factors for calculating allowable bearing capacity.
5. Updated references to supporting calculations.
6. Updated discussions and conclusions to incorporate revised results.

REVISION 5

1. Shear strength of clayey soils beneath the building for resisting sliding was changed from 1.8 ksf to 1.7 ksf to reflect lower final effective stresses under the mat after changing size of mat to 240 ft x 279.5 ft.
2. Added sliding analysis that includes both shear resistance along bottom of the plane of the clayey soils enclosed within the perimeter key at the base of the mat and the full passive resistance from the soil cement placed adjacent to the mat. Used residual strength measured in the direct shear tests that were performed on these clayey soils for this case.

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REVISION 6

1. Expanded description of soil cement properties.
2. Added discussion to clarify use of peak strengths measured in the direct shear tests along with one-half of passive resistance and residual strengths along with full passive resistance in sliding stability analysis.
3. Added calculation of horizontal displacement of the building due to elastic theory.
4. Expanded discussion of residual strengths of the clayey soils underlying the building.

OBJECTIVE

To determine the stability against overturning, sliding, and static and dynamic bearing capacity failure of the Canister Transfer Building supported on a mat foundation.

ASSUMPTIONS/DATA

The footprint of the Canister Transfer Building foundation mat is shown on SAR Figure 4.7-1, "Canister Transfer Building," and S&W Drawing 0599602-EC-404A-B & 404B-B, Canister Transfer Building – Conc Mat Foundation Plan, Sheets 1 & 2. The elevation view of the structure is shown on Sheets 2 & 3 of SAR Figure 4.7-1. The foundation mat is 240 ft (E-W) x 279.5 ft (N-S) x 5 ft thick, with a 6.5-ft wide x 1.5-ft deep foundation key along the perimeter of the mat.

Figure 1 presents a schematic view of the foundation and identifies the coordinate system used in these analyses. Figure 2 presents the stick model used in the structural analysis of the Canister Transfer Building.

The various static and dynamic loads and load combinations used in these analyses were obtained from Calculation 05996.02-SC-5-2 (S&W, 2001). All loads are transferred to the bottom of the mat. Moments, when transferred to the bottom of the mat, result in eccentricity of the applied load with respect to the center of gravity of the mat. Lateral loads, when combined with the vertical load, result in inclination of the vertical load, which decreases the allowable bearing capacity.

The generalized soil profile at the site is shown on Figure 3. The soil profile consists of ~30 ft of silty clay/clayey silt with sandy silt/silty sand layers (Layer 1), overlying ~30 ft of very dense fine sand (Layer 2), overlying extremely dense silt (N ≥100 blows/ft, Layer 3). SAR Figures 2.6-21 through 23 present foundation profiles showing the relationship of the Canister Transfer Building with respect to the underlying soils. These profiles, located as shown in SAR Figure 2.6-18, provide more detailed stratigraphic information, especially within the upper ~30-ft thick layer at the site.

The bearing capacity analyses assume that Layer 1, which consists of silty clay/clayey silt with some sandy silt/silty sand, is of infinite thickness and has strength properties based on those measured for the clayey soils within the upper layer. These assumptions simplify the analyses and they are very conservative. The strength of the sandy silt/silty sand in the upper layer is greater than that of the clayey soils, based on the increases in Standard Penetration Test (SPT) blow counts (N-values) and the increased tip resistance (see SAR Figure 2.6-5, Sheet 1) in the cone penetration testing (ConeTec, 1999) measured for these soils. The underlying soils are even stronger, based on their SPT N-values, which generally exceed 100 blows/ft.

Geotechnical Properties

Based on laboratory test results presented in Table 3 of Calculation 05996.02-G(B)-5-2 (SWEC, 2000a), γ_{moist} = 80 pcf above the bottom of the mat and 90 pcf below the mat.

Table 6 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A) summarizes the results of the triaxial tests that were performed within depths of ~10 ft. The undrained shear strengths (s_u) measured in these tests are plotted vs confining pressure in Figure 6. 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 partially saturated, fine-grained soils will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. As indicated in Figure 6, 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 6 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 bearing capacity analyses of these structures.

The bearing capacity of the structures are dependant primarily on the strength of the soils in the upper ~25 to ~30-ft layer at the site. All of the borings drilled at the site indicate that the soils underlying this upper layer are very dense fine sands overlying silts with standard penetration test blow counts that exceed 100 blows/ft. The results of the cone penetration testing, presented in ConeTec(1999) and plotted in SAR Figure 2.6-5, Sheets 1 to 14, illustrate that the strength of the soils in the upper layer are much greater at depths below ~10 ft than in the range of ~5 ft to ~10 ft, where most of the triaxial test specimens were obtained.

In determining the bearing capacity of the foundation, the average shear strength of the soils along the anticipated bearing capacity failure slip surface should be used. This slip surface is normally confined to the zone within a depth below the footing equal to the minimum width of the footing. For the Canister Transfer Building, the effective width of the footing is decreased because of the large eccentricity of the load on the mat due to the seismic loading. As indicated in Table 2.6-10, the minimum effective width of the Canister Transfer Building occurs for Load Case IIIA, where B' = 119.5 ft. This is greater than the depth of the upper layer (~30 ft). Therefore, it is conservative to use the average strength of the soils in the upper layer in the bearing capacity analyses, since all of the soils in the upper layer will be effective in resisting failure along the anticipated bearing capacity slip surface.

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% (Δs_u = 100% x [(2.20 tsf – 1.34 tsf) / 1.34 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 ksf – 1.46 ksf) ÷ 0.09 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

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 ϕ = 30° 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: ϕ = 0° & c = 3.18 ksf.

Case IB Static using effective-stress strength parameters: ϕ = 30° & c = 0.

and dynamic bearing capacity analyses are performed using ϕ = 0° & 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

of these soils with varying amounts of cement and the testing of compacted specimens of soil-cement to determine moisture-density relationships, freeze/thaw and wet/dry characteristics, compressive and tensile strengths, and permeability of compacted soil-cement specimens. The entire laboratory testing program is being conducted in full compliance with the Quality Assurance (QA) Category I requirements of the ESSOW.

As part of this effort, PFS is performing so-called durability testing. These tests are performed in accordance with ASTM D559 and D560 to measure the durability of soil cement specimens exposed to 12 cycles of wet/dry and freeze/thaw conditions. As indicated on p. 16 of PFS Calculation 05996.02-G(B)-04-8:

> *"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)."*

PFS is performing these tests to determine the amounts of cement and water that must be added to the site soils and to determine the compaction requirements to ensure that the soil cement will be durable and will withstand exposure to the elements. As indicated on p. 8 of PCA[1]:

> *"The freeze-thaw and wet-dry tests were designed to determine whether the soil-cement would stay hard or whether expansion and contraction on alternate freezing-and-thawing and moisture changes would cause the soil-cement to soften."*

And on p. 32:

> *"The principle requirement of a hardened soil-cement mixture is that it withstand exposure to the elements. Thus the primary basis of comparison of soil-cement mixtures is the cement content required to produce a mixture that will withstand the stresses induced by the wet-dry and freeze-thaw tests. The service record of projects in use proves the reliability both of the results based on these tests and of the criteria given below.*
>
> *The following criteria are based on considerable laboratory test data, on the performance of many projects in service, and on information obtained from the outdoor exposure of several thousand specimens. The use of these criteria will provide the minimum cement content required to produce hard, durable soil-cement, suitable for base-course construction of the highest quality.*
>
> 1. *Soil-cement losses during 12 cycles of either the wet-dry test or freeze-thaw test shall conform to the following limits:*
>
>   *Soil Groups A-1, A-2-4, A-2-5, and A-3, not over 14 percent;*
>
>   *Soil Groups A-2-6, A-2-7, A-4, and A-5, not over 10 percent;*
>
>   *Soil Groups A-6 and A-7, not over 7 percent.*

[1] Portland Cement Association, "Soil-Cement Laboratory Handbook," Skokie, IL, 1971.

2. *Compressive strengths should increase both with age and with increases in cement content in the ranges of cement content producing results that meet requirement 1."*

The on-going laboratory testing program will also include additional tests to confirm that the bond at the interfaces between lifts of soil-cement and soil-cement and the site soils will exceed the strength of the in situ clayey soils. These tests will include direct shear tests, performed on specimens prepared from the site soils at various cement and moisture contents, in a manner similar to that used by DeGroot[2] in his testing of bond along soil-cement interfaces. This testing will include direct shear tests to be performed in the laboratory in the near-term (pre-construction) during the soil-cement mix development to demonstrate that the required interface strengths can be achieved (p. 2.6-113 of SAR) and during construction to demonstrate that the required interface strengths are achieved (p. 2.6-114 of SAR). In addition, PFS has committed to augmenting this field testing program by performing additional site-specific testing of the strengths achieved at the interface between the bottom of the soil cement and the underlying soils.

[2] DeGroot, G., 1976, "Bonding Study on Layered Soil Cement", REC-ERC-76-16, U.S. Bureau of Reclamation, Denver, CO, September 1976.

METHOD OF ANALYSIS

Load cases analyzed consist of combinations of vertical static, vertical dynamic (compression and uplift, Y-direction), and horizontal dynamic (in X and Z-directions) loads.

The following load combinations are analyzed:

Case I	Static
Case II	Static + dynamic horizontal forces due to the earthquake
Case III	Static + dynamic horizontal + vertical uplift forces due to the earthquake
Case IV	Static + dynamic horizontal + vertical compression forces due to the earthquake

For Case II, 100% of the dynamic lateral forces in both X and Z directions are combined. For Cases III and IV, 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the dynamic loading acts in the other two directions. For these cases, the suffix "A" is used to designate 40% in the X direction (N-S for the Canister Transfer Building, as shown in Figure 1), 100% in the Y direction (vertical), and 40% in the Z direction (E-W). Similarly, the suffix "B" is used to designate 40% in the X direction, 40% in the Y, and 100% in the Z, and the suffix "C" is used to designate 100% in the X direction and 40% in the other two directions. Thus,

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.

The negative sign for the vertical direction in Case III indicates uplift forces due to the earthquake. Case IV is the same as Case III, but the vertical forces due to the earthquake act downward in compression; therefore, the signs on the vertical components are positive.

Combining the effects of the three components of the design basis ground motion in this manner is in accordance with ASCE-4 (1986).

ANALYSIS OF OVERTURNING STABILITY

The factor of safety against overturning is defined as:

$$FS_{OT} = \Sigma M_{Resisting} \div \Sigma M_{Driving}$$

The overturning stability of the Canister Transfer Building is determined using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads are listed in Table 2.6-11, and they were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (S&W, 2001) and described in SAR Section 4.7.1.5.3. The masses and accelerations of the joints (see Figure 2 for locations of the joints) used in the model of the Canister Transfer Building in Calculation 05996.02-SC-5 are listed on the left side of Table 2.6-11, and the resulting inertial forces and associated moments are listed on the right. Based on building geometry shown schematically in Figure 1 and the forces and moments shown in Table 2.6-11, overturning is more critical about the N-S axis (279.5 ft) than about the E-W axis (240 ft). Page 37 of Calculation 05996.02-SC-5 indicates that the moment due to angular (rotational) acceleration of the structure is 465,729 ft-K about the N-S axis and 1,004,332 ft-K about the E-W axis.

The vertical force due to the earthquake can act upward or downward. However, when it acts downward, it acts in the same direction as the weight, tending to stabilize the structure with respect to overturning stability. The minimum factor of safety against overturning will occur when the maximum dynamic vertical force acts in the upward direction, tending to unload the mat and reduce the resisting moment. Therefore, calculate the factor of safety for Case III.

CHECKING OVERTURNING ABOUT THE N-S AXIS

For Case IIIA, where 40% of the horizontal force due to the earthquake act in the N-S and E-W directions and 100% acts vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 79,779 K, (i.e., Weight – Total $F_{V\ Dyn}$), as shown in Table 2.6-11. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of 240 ft, or 120 ft. Therefore,

$$\Sigma M_{Resisting} = (97,749 - 79,779)\text{ K} \times 120\text{ ft} = 2,156,400\text{ ft-K.}$$

This ignores the eccentricities of the vertical masses with respect to the center of the mat. Incorporating these eccentricities, which are included in Attachment A of Calc 05996.02-SC-5, Rev. 2, the resulting resisting moment is calculated as follows:

JOINT	EL.	MASS Y k-sec²/ft	AY g's	Z (E-W) ft	Moment Arm E-W ft	$SM_{@N-S}$ ft-K
0	94.25	260.1	0.783	0	120.00	218,002
1	95	1,908.0	0.783	-0.73	119.27	1,589,353
2	130	420.4	0.821	-2.02	117.98	285,292
3	170	304.3	0.913	-3.14	116.86	99,412
4	190	117.1	0.928	0	120.00	32,638
5	190	27.6	1.840	0	120.00	-89,478
6	170	1.0	0	0	120.00	3,860
					Total =	**2,139,080**

The driving moments include 40% of the ΣM acting about the N-S axis, $\Sigma M_{@X}$ in Table 2.6-11, which is 0.4 x 2,706,961.4 = 1,082,785 ft-K, and 40% of the moment about the N-S axis due to angular (rotational) acceleration of the structure, which is 0.4 x 465,729 = 186,292 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the E-W axis do not contribute to overturning about the N-S axis; therefore,

$$\sum M_{Driving} = \sqrt{1{,}082{,}785^2 + (186{,}292)^2} = 1{,}098{,}694 \quad \text{ft} - \text{K}$$

and **FS_{OT}** = 2,156,400 ÷ 1,098,694 = **1.96**

about the N-S axis for Case IIIA without including eccentricities of vertical masses.

Including the effect of the eccentricities of the vertical masses, the resulting factor of safety against overturning is:

FS_{OT} = 2,139,080 ÷ 1,098,694 = **1.95** **<u>(Minimum)</u>**

For Case IIIB, where 100% of the horizontal force due to the earthquake acts in the E-W direction and 40% acts in the N-S direction and vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 40% of 79,779 K, (i.e., Weight – Total $F_{V\,Dyn}$), as shown in Table 2.6-11. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of 240 ft, or 120 ft. Therefore,

$\Sigma M_{Resisting}$ = (97,749 – 0.4 x 79,779) K x 120 ft = 7,900,488 ft-K.

CALCULATION IDENTIFICATION NUMBER				PAGE 16
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

The driving moments include 100% of the ΣM acting about the N-S axis, $\Sigma M_{@X}$ in Table 2.6-11, which is 2,706,961.4 ft-K, and 100% of the moment about the N-S axis due to angular (rotational) acceleration of the structure, which is 465,729 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the E-W axis do not contribute to overturning about the N-S axis; therefore,

$$\sum M_{Driving} = \sqrt{2{,}706{,}961.4^2 + 465{,}729^2} = 2{,}746{,}733 \quad \text{ft} - \text{K}$$

and FS_{OT} = 7,900,488 ÷ 2,746,733 = **2.88** about the N-S axis for Case IIIB.

Case IIIC, where 100% of the horizontal force due to the earthquake acts in the N-S direction and 40% acts in the E-W direction and vertically upward, **is less critical** for overturning about the N-S axis than Case IIIB.

CHECKING OVERTURNING ABOUT THE E-W AXIS

For Case IIIA, where 40% of the horizontal force due to the earthquake act in the N-S and E-W directions and 100% acts vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 79,779 K, (i.e., Weight – Total $F_{V\ Dyn}$), as shown in Table 2.6-11. For overturning about the E-W axis, the moment arm for the resisting moment equals ½ of 279.5 ft, or 139.75 ft. Therefore,

$\Sigma M_{Resisting}$ = (97,749 – 79,779) K x 139.75 ft = 2,511,308 ft-K.

This ignores the eccentricities of the vertical masses with respect to the center of the mat. Incorporating these eccentricities, the resulting resisting moment is calculated as follows:

JOINT	EL.	MASS Y k-sec²/ft	AY g's	Moment Arm N-S ft	$SM_{@E\text{-}W}$ ft-K
0	94.25	260.1	0.783	139.75	253,882
1	95	1,908.0	0.783	138.08	1,840,009
2	130	420.4	0.821	131.46	317,889
3	170	304.3	0.913	143.18	121,802
4	190	117.1	0.928	139.75	38,010
5	190	27.6	1.840	139.75	-104,205
6	170	1.0	0	139.75	4,496
				Total =	**2,471,883**

The driving moments include 40% of the ΣM acting about the E-W axis, $\Sigma M_{@z}$ in Table 2.6-11, which is 0.4 x 2,849,703 = 1,139,881 ft-K, and 40% of the moment about the E-W axis due to angular (rotational) acceleration of the structure, which is 0.4 x 1,004,322 = 401,729 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the N-S axis do not contribute to overturning about the E-W axis; therefore,

$$\sum M_{Driving} = \sqrt{1{,}139{,}881^2 + 401{,}729^2} = 1{,}208{,}601 \quad ft-K$$

and FS_{OT} = 2,511,308 ÷ 1,208,601 = **2.07**

about the E-W axis for Case IIIA without including eccentricities of vertical masses.

Including the effect of the eccentricities of the vertical masses, the resulting factor of safety against overturning is:

FS_{OT} = **2,471,883** ÷ 1,208,601 = **2.05 (Minimum @ E-W Axis)**

For Case IIIC, where 100% of the horizontal force due to the earthquake acts in the N-S direction and 40% acts in the E-W direction and vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 40% of 79,779 K, (i.e., Weight – Total $F_{V\,Dyn}$), as shown in Table 2.6-11. For overturning about the E-W axis, the moment arm for the resisting moment equals ½ of 279.5 ft, or 139.75 ft. Therefore,

$\Sigma M_{Resisting}$ = (97,749 – 0.4 x 79,779) K x 139.75 ft = 9,200,777 ft-K.

The driving moments include 100% of the ΣM acting about the E-W axis, $\Sigma M_{@z}$ in Table 2.6-11, which is 2,849,703.4 ft-K, and 100% of the moment about the E-W axis due to angular (rotational) acceleration of the structure, which is 1,004,322 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the N-S axis do not contribute to overturning about the E-W axis; therefore,

$$\sum M_{Driving} = \sqrt{2{,}849{,}703^2 + 1{,}004{,}322^2} = 3{,}021{,}501 \quad ft-K$$

and FS_{OT} = 9,200,777 ÷ 3,021,501 = **3.05** about the E-W axis for Case IIIC.

Case IIIB is less critical for overturning about the N-S axis than Case IIIC.

Analysis of Sliding Stability

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

$$FS = \text{Resisting Force} \div \text{Driving Force} = T \div V$$

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

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

where, N (normal force) = $\sum F_v = F_{v\,Static} + F_{v\,Eqk}$

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

c = 1.7 ksf, as discussed above under "Geotechnical Properties."

B = 240 feet

L = 279.5 feet

The driving force, V, is calculated as follows:

$$V = \sqrt{F_{HN-S}^2 + F_{HE-W}^2}$$

Sliding Stability of the Canister Transfer Building on In Situ Clayey Soils

Based on Half of the Passive Resistance of the Soil Cement and the Peak Strength of the Clayey Soils Under the Building

The sliding stability of the CTB was evaluated using the foundation loadings developed in the soil-structure interaction analyses (Calculation 05996.02-SC-5, S&W, 2001). In this case, the strength of the clayey soils at the bottom of the 1.5-ft deep key around the CTB mat was based on the average of the two sets of direct shear tests performed on samples of soils obtained from beneath the CTB, approximately at the elevation proposed for founding the structure. The results of these tests are included in Attachments 7 and 8 of Appendix 2A of the SAR, and Figures 7 and 8 present plots of peak shear stress vs normal stress measured in these tests. As discussed above under Geotechnical Properties, $\phi = 0°$ and a shear strength of 1.7 ksf were used for the clayey soils underlying the Canister Transfer Building in determining resisting forces for the earthquake loading combinations.

The unconfined compressive strength of the soil cement adjacent to the Canister Transfer Building will be at least 250 psi. These analyses assume that the peak shear strength of the clayey soils under the Canister Transfer Building are available to resist sliding along with up to half of the passive resistance of the soil cement.

The backfill to be placed around the Canister Transfer Building mat and 1.5-ft deep key will be soil cement, constructed from the eolian silt and silty clay that was excavated from the area. For soil cement constructed using these soils, it is reasonable to assume the lower bound value of γ is 100 pcf, $\phi = 0°$ & c = 125 psi.

For the soil cement, $P_p = 2c \times D_f \times (B \text{ or } L)$

For 5' of soil cement, using a factor of safety of 2 applied to the passive resistance,

$$P_p = \frac{2 \times c \times D_f \times w}{FS} = \frac{2 \times 125 \frac{\#}{in.^2} \times \frac{144 \cdot in.^2}{ft^2} \times \frac{K}{1{,}000\#} \times 5\,ft \times 1 \frac{ft}{LF}}{2} = 90 \frac{K}{LF}$$

The CTB mat is 240' wide in the E-W direction and 279.5' long in the N-S direction; therefore, the passive force available to resist sliding is at least 240' x 90 K/LF = 21,600 K acting in the N-S direction in the analyses that use half of the passive resistance of the soil cement adjacent to the mat.

The effects of wall movement on wall pressure are defined in DM-7[3] (p. 7.2-60) as the ratio of horizontal displacement to the height of the wall. For stiff cohesive soils, the wall rotation or yield ratio, y/H, required to fully mobilize passive resistance is 0.02, or 2%. For dense cohesionless soils, even less movement is required to reach full passive, ~0.2%. Lambe & Whitman (1969, p 166) also indicates that little horizontal compression, ~0.5%, is required to reach half of full passive resistance for dense sands. The soil cement will be compacted to a dense state, and once it cures, it is expected to be stiffer than dense sand, requiring less displacement to reach full passive resistance. Therefore, it is conservative to assume that half of the total passive resistance is available to resist sliding of the building.

Note, if we assume that the soil cement is comparable in stiffness to stiff cohesive soil, the figure from DM-7 cited above indicates that yield ratio, y/H, required to fully mobilize passive resistance is 2%. It is reasonable to use a yield ratio of half of this, or ~1% of the 5 ft height of the mat + 1.5-ft deep key, to reach half of passive resistance for the soil cement adjacent to the mat. This indicates that a horizontal displacement of the mat = 0.01 x 6.5 ft x 12 in./ft = 0.78 in. would be sufficient to reach half of the passive resistance. Since there are no safety-related systems that would be severed or otherwise impacted by movements of this small magnitude, it is reasonable to use this passive thrust to resist sliding. The following analysis demonstrates that it is also reasonable to use the resistance provided by the peak shear strength of the clayey soils enclosed within the perimeter key at the base of the mat to resist sliding in this case, because this amount of horizontal displacement can be obtained from elastic deformation of the clayey soils underlying the building.

The horizontal displacement of the Canister Transfer Building is estimated using elastic theory, as described in Section 4.3, "Rectangles Subjected to Shear Loading," of Poulos and Davis[4].

$$\rho = \frac{q \times a \times I}{E}$$ Eq. 4.9 Poulos & Davis

3 NAVFAC (1986), DM 7.2, "Foundations and Earth Structures," Dept of the Navy, Naval Facilities Eng'g. Command, Alexandria, VA.

4 Poulos, H. G., and Davis, E. H., Elastic Solutions for Soil and Rock Mechanics, John Wiley & Sons, New York, NY, 1974.

$$G_s = \rho \times V_s^2 = \frac{80\text{ pcf}}{32.2\,\frac{\text{ft./sec}^2}{g}} \times (540\text{ ft/sec})^2 = 724{,}472\text{ psf} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 = 5{,}031\text{ psi}$$

$$E_s = 2 \times (1+\nu) \times G_s = 2 \times (1+0.4) \times 5{,}031\text{ psi} = 14{,}087\text{ psi}$$

In the E-W direction (See Table 2.6-11 for horizontal shear values):

$$q = \frac{99{,}997\text{ K}}{240\text{ ft} \times 279.5\text{ ft}} = 1.49\text{ ksf} \times \frac{1{,}000\text{ lbs}}{\text{K}} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 = 10.4\text{ psi}$$

$$\frac{h}{b} = \frac{6.5\text{ ft}}{279.5\text{ ft}} = 0.023$$

$$\frac{b}{a} = \frac{279.5\text{ ft}}{240\text{ ft}} = 1.17$$

In the N-S direction:

$$q = \frac{111{,}108\text{ K}}{240\text{ ft} \times 279.5\text{ ft}} = 1.66\text{ ksf} \times \frac{1{,}000\text{ lbs}}{\text{K}} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 = 11.5\text{ psi}$$

$$\frac{h}{b} = \frac{6.5\text{ ft}}{240\text{ ft}} = 0.027$$

$$\frac{b}{a} = \frac{240\text{ ft}}{279.5\text{ ft}} = 0.859$$

From Figure 4.17 of Poulos & Davis, estimate the horizontal displacement factor for the corners for horizontal shear of a horizontal rectangle. For the h/b and b/a values shown above, $I_{E\text{-}W} = 0.62$ and $I_{N\text{-}S} = 0.59$.

$$\rho_{E-W} = \frac{10.4\text{ psi} \times 240\text{ ft} \times 12\,\frac{\text{in.}}{\text{ft}} \times 0.62}{14{,}087\text{ psi}} = 1.32\text{ inches} \quad \text{Eq. 4.9 Poulos \& Davis}$$

$$\text{Yield Ratio} = \frac{\rho}{H} = \frac{1.32\text{ in.}}{6.5\text{ ft} \times 12\,\frac{\text{in.}}{\text{ft}}} = 0.017\text{, or }1.7\%$$

$$\rho_{N-S} = \frac{11.5\text{ psi} \times 279.5\text{ ft} \times 12\frac{\text{in.}}{\text{ft}} \times 0.59}{14{,}087\text{ psi}} = 1.62\text{ inches} \quad \text{Eq. 4.9 Poulos \& Davis}$$

$$\text{Yield Ratio} = \frac{\rho}{H} = \frac{1.62\text{ in.}}{6.5\text{ ft} \times 12\frac{\text{in.}}{\text{ft}}} = 0.021\text{, or } 2.1\%$$

Thus, based on the shear modulus estimated from the shear wave velocity of the surficial silty clay/clayey silt, the horizontal displacement of the CTB subjected to the full horizontal earthquake load is calculated to be about 1.3 to 1.6 inches using the elastic solution of a buried horizontal rectangle subjected to shear in an elastic half-space. This horizontal displacement corresponds to a yield ratio, defined as horizontal displacement ÷ height of wall, of 2% from translation of the 6.5 ft height of the CTB foundation mat adjacent to the soil cement. This yield ratio is larger than the yield ratio required to mobilize one half of full passive resistance for dense sand or stiff cohesive soils. This displacement is sufficient to develop full passive resistance in the soil cement adjacent tot he mat; therefore, it is conservative to use one-half of the passive resistance in these analyses

The results of the sliding stability analysis of the Canister Transfer Building for this case are presented in Table 2.6-13. 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 1.15, 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 are conservative, because they assume that only one-half of the passive pressures are available to resist sliding and no credit is taken for the fact that the strength of cohesive soils increases as the rate of loading increases. Note, Newmark and Rosenblueth (1973) indicate:

> *"In all cohesive soils reported to date, strength and stiffness increase markedly with strain rate (Figs. 13.6 and 13.7). An increase of the order of 40 percent is common for the usual strain rates of earthquakes, above the strength and stiffness of static tests."*

Schimming et al, (1966), Casagrande and Shannon (1948, and Das (1993) all report similar increases in strength of cohesive soils due to rapid loading. Therefore, since these results are based on static shear strengths, they represent conservative lower-bound values of the factor of safety against sliding of the Canister Transfer Building founded on in situ silty clay/clayey silt with soil-cement backfill around the mat.

Based on the Full Passive Resistance of the Soil Cement and the Residual Strength of the Clayey Soils Under the Building

Before a complete sliding failure can occur, the full passive resistance of the soil cement must be engaged. Because the horizontal displacements associated with reaching the full passive state typically are large for soils, in the analyses where the full passive resistance of the soil cement adjacent to the mat is used, the shear strength of the clayey soils under the building is reduced to a conservative estimate of the residual shear strength based on the results of the direct shear tests.

The results of the direct shear tests, presented as plots of shear stress vs horizontal displacement in Attachment 7 of Appendix 2A of the SAR (annotated copies are included in Attachment C of this calculation), illustrate that the residual strength of these soils is nearly equal to the peak strength for those specimens that were tested at confining stresses of 2 ksf. For example, for Sample U-1C from Boring C-2, at horizontal displacements of ~0.025" past the peak strength, there is ~1.5% reduction in the shear strength indicated. The results for Sample U-1AA from Boring CTB-S showed no decrease in shear strength following the peak at ~0.025" horizontal displacement, and Samples U-3B&C from Boring CTB-6 showed a decrease of ~5%. The specimens that were tested at confining stresses of 1 ksf all show reductions of ~20% at horizontal displacements of ~0.025" past the peak.

The final effective vertical stresses at the base of the Canister Transfer Building, σ'_v, are ~1.5 ksf, now that the mat has been changed to 240 ft x 279.5 ft. This value is approximately half-way between the confining stresses of 1 and 2 ksf used for several of the direct shear tests. The residual strength of the clayey soils beneath the building are expected to show reductions from the peak strength of ~10% to ~12.5%; i.e., approximately half-way between the reductions observed for the specimens tested at confining stresses of 1 ksf and 2 ksf, since the final effective stresses under the building are ~1.5 ksf; i.e., approximately half-way between confining stresses used in these tests (1 ksf and 2 ksf). Therefore, it is reasonable to assume that the peak strength of the clayey soils enclosed within the perimeter key at the base of the Canister Transfer Building mat should be reduced to account for horizontal displacement required to reach full passive resistance of the soil cement adjacent to the mat. Based of the results of the direct shear tests performed on samples of the site soils, it would be reasonable to use a reduction of ~10% to ~12.5% to obtain the residual strength applicable for the final vertical stresses at the base of the Canister Transfer Building. The analyses that follow, however, reduce the peak strength even more than this, by a total of 20%, to provide additional conservatism.

The following table illustrates further that using a reduction of the peak strength equal to 20% provides a conservative estimation of the residual strength of these soils. This table presents the peak strengths measured in the direct shear tests at normal stresses of 1 ksf and 2 ksf. It also lists the final shear strengths measured in these tests, which were generally obtained at horizontal displacements of 0.25 inches or 0.30 inches. The table also lists the calculated post-peak strength reduction for these test results, as well as the average post-peak strength reduction for normal stress of 1.5 ksf, which is applicable for

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 Displace-ment	Post-Peak Strength Reduction	Peak Strength	Strength at Maximum Shear Displace-ment	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.

SLIDING STABILITY OF THE CANISTER TRANSFER BUILDING ON COHESIONLESS SOILS

The Canister Transfer Building will be founded on clayey soils that have an adequate amount of cohesive strength to resist sliding due to the dynamic forces from the design basis ground motion. As shown in SAR Figures 2.6-21 through 2.6-23, however, some of the soils underlying the building are cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. Analyses presented on the next six pages address the possibility that sliding may occur along a deeper slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

The resistance to sliding is greatly reduced for frictional materials when the dynamic forces due to the earthquake act upward. The normal forces act downward for Case IV loadings and, hence, the resisting forces will be much greater than those for Case III. Therefore, these analyses are performed only for Load Cases IIIA, IIIB, and IIIC. As described above, these load cases are defined as follows:

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.

As shown in SAR Figures 2.6-21 through 2.6-23, the top of the cohesionless layer varies from about 5 ft to about 9 ft below the mat, and it generally is at a depth of about 6 ft below the mat. These analyses include the passive resistance acting on a plane extending from grade down to the top of the cohesionless layer, plus the shear strength available at the ends of the silty clay block under the mat, plus the frictional resistance available along the top of the cohesionless layer. The weight of the clayey soils existing between the top of the cohesionless soils and the bottom of the mat is included in the normal force used to calculate the frictional resistance acting along the top of the cohesionless layer.

A review of the cone penetration test results (ConeTec, 1999) obtained within the top 2 ft of the layer of nonplastic silt/silty sand/sandy silt underlying the Canister Transfer Building indicated that $\phi = 38°$ is a reasonable minimum value for these soils. This review is presented on the next page.

The next five pages illustrate that the factor of safety against sliding along the top of this layer is >1.1 for all load cases (i.e., Load Cases IIIA, IIIB, and IIIC). These analyses include several conservative assumptions. They are based on static strengths of the silty clay block under the Canister Transfer Building mat, even though, as reported in Das (1993), experimental results indicate that the strength of cohesive soils increases as the rate of loading increases. For rates of strain applicable for the cyclic loading due to the design basis ground motion, Das indicates that for most practical cases, one can assume that $c_{u\ dynamic} \sim 1.5 \times c_{u\ static}$. In addition, the silty sand/sandy silt layer is not continuous under the Canister Transfer Building mat, and this analysis neglects cementation of these soils that was observed in the samples obtained in the borings. Therefore, sliding is not expected to occur along the surface of the cohesionless soils underlying the Canister Transfer Building.

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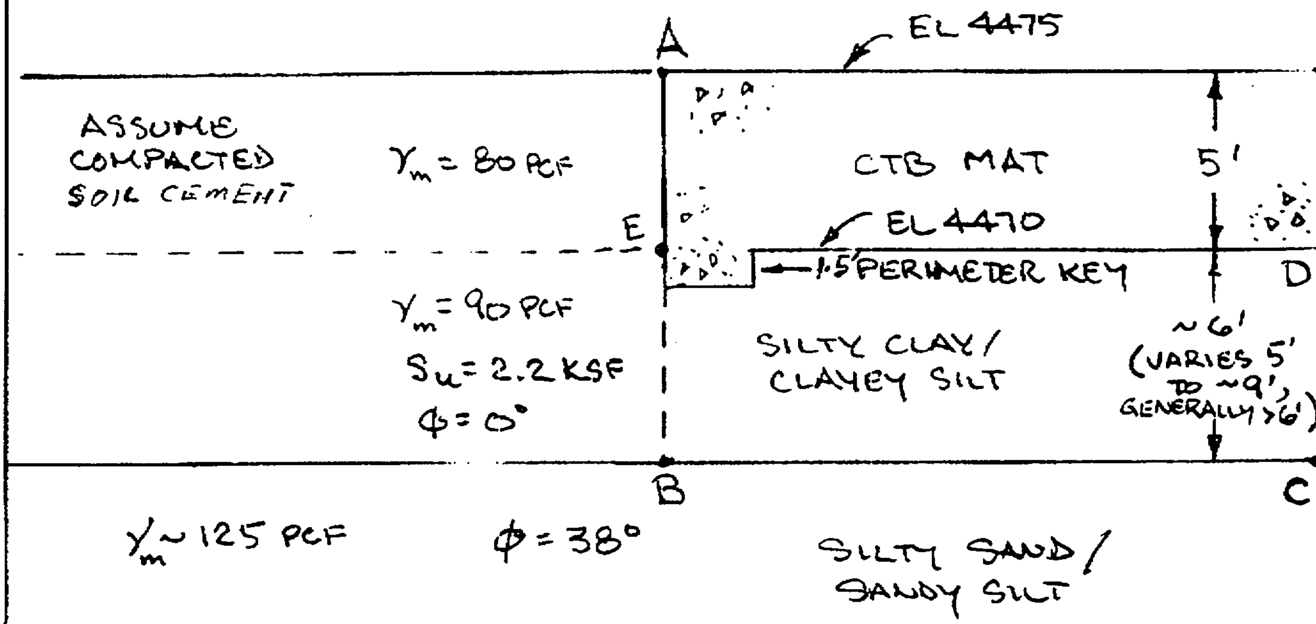
SLIDING ON DEEP PLANE AT TOP OF SILTY SAND / SANDY SILT LAYER



NOTE: VALUE OF ϕ BASED ON ϕ DATA FROM CPT-37 & 38. PRESENTED IN CONETEC (1999)

ID	~DEPTH OF SILTY SAND	MIN ϕ	MAX ϕ	AVG ϕ	MEDIAN ϕ	ϕ IN TOP 2'
CPT-37	~11.6' TO ~18.7'	36*	44	40	40	~38
CPT-38	~11' TO ~18'	38	46	43	44	~38

PASSIVE PRESSURES ACTING ON PLANE AB WILL INCREASE AS B GETS DEEPER IN THE SILTY SAND / SANDY SILT LAYER; ∴ USE ϕ NEAR THE TOP OF THE LAYER. ⇒ $\phi = 38°$.

N VALUES ARE HIGH, GENERALLY >> 20 BL/FT; ∴ $\phi = 38°$ IS REASONABLE

* EXCLUDING SINGLE VALUE OF $\phi = 34°$ AT $z = 13.8'$

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SLIDING ON DEEP COHESIONLESS PLANE

$$FS_{SLIDING} = \frac{\Sigma \text{ RESISTING FORCES}}{\Sigma \text{ DRIVING FORCES}}$$

RESISTING FORCES INCLUDE: PASSIVE RESISTANCE AVAILABLE ALONG AB + SHEAR RESISTANCE ALONG ENDS OF BLOCK BCDE + FRICTION ALONG BC.

(1) PASSIVE RESISTANCE AVAILABLE ALONG AB

INCLUDES $(2 \times 5 \times 125 \times .144 \frac{K}{FT^2}) \times (5') = 180$ K/LF FOR COMPACTED 5' SOIL-CEMENT ADJACENT TO 5' MAT

$+ \frac{1}{2} \gamma H^2 K_p + q_s H K_p + 2cH\sqrt{K_p}$ FOR 5' BLOCK OF SILTY CLAY UNDERLYING THE COMPACTED SOIL-CEMENT

$$\frac{1}{2}\left(0.090 \frac{K}{FT^3}\right) \times (5 \text{ FT})^2 \times \underset{K_p}{1.0} + \underset{H}{6 \text{ FT}} \times \underset{\gamma}{0.080 \frac{K}{FT^3}} \times 5 \text{ FT} \times \underset{K_p}{1.0}$$

$$+ 2 \times \underset{c}{2.2} \frac{K}{FT^2} \times 5 \text{ FT} \times \sqrt{1.0} = 1.125 + 2.40 + 22.0 = 25.52 \frac{K}{FT}$$

$\therefore$ TOTAL PASSIVE RESISTANCE AVAILABLE ALONG AB

$= 180 + 25.52 = 205.52$ K/LF

(2) ESTIMATE ADDITIONAL RESISTANCE TO SLIDING AVAILABLE AT THE ENDS OF THE BLOCK OF SILTY CLAY THAT MUST SHEARED BEFORE THE CTB CAN SLIDE. INCLUDE ONLY THE PORTION BELOW THE CTB MAT; i.e., BCDE SHOWN ON PAGE 19.

$S_u = 2.2$ KSF, = MINIMUM S_u MEASURED IN UU TRIAXIAL TESTS AT $\sigma_c = 1.3$ KSF

$$\text{AREA BCDE} = 6\,\text{FT} \times 240\,\text{FT}_{E\text{-}W} = 1440\,\frac{\text{FT}^2}{\text{END}}$$

$$\therefore\ \Delta T_{\text{ENDS}_{E\text{-}W}} = 2\ \text{ENDS} \times 1440\,\frac{\text{FT}^2}{\text{END}} \times 2.2\,\frac{\text{K}}{\text{FT}^2} = 6{,}336\ \text{K}_{E\text{-}W}$$

$$\Delta T_{\text{END}_{N\text{-}S}} = 2\ \text{ENDS} \times 6' \times \underset{L_{N\text{-}S}}{279.5'} \times 2.2\,\frac{\text{K}}{\text{FT}^2} = 7{,}379\ \text{K}_{N\text{-}S}$$

(3) FRICTIONAL RESISTANCE ALONG PLANE BC:

ADD WEIGHT OF SILTY CLAY BLOCK BETWEEN BOTTOM OF MAT & TOP OF SILTY SAND/SANDY SILT TO THE NORMAL FORCE AT BOTTOM OF THE MAT.

$$\Delta N_{\text{CLAY}} = \underset{\Delta H}{6'} \times \underset{\gamma}{0.090\,\frac{\text{K}}{\text{FT}^3}} \times \underset{B \times L}{240' \times 279.5'} = 36{,}223\ \text{K}$$

(4) SINCE THE MATERIAL FROM GROUND SURFACE TO THE TOP COHESIONLESS SILTY SAND/SANDY SILT ARE ALL COHESIVE (SOIL CEMENT, SILTY CLAY), THE ACTIVE EARTHQUAKE PRESSURE IS SMALL AND NEGLIGIBLE.

NOTE: FRICTIONAL RESISTANCE WILL BE LOWER WHEN VERT EARTHQUAKE FORCES ACT UPWARD. ∴ CHECK CASES III A, B & C

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SLIDING ON DEEP PLANE

	N-S	VERT	E-W
CASE IIIA:	40% IN X	−100% IN Y	40% IN Z
FROM TABLE 1	$0.4 \times 111{,}108^K$	$-79{,}779K$	$\underbrace{0.4 \times 99{,}997}_{L = V_{E\text{-}W}} = 39{,}999^K$

$$\therefore N = \underset{\text{CTB DL}}{97{,}749} - \underset{F_{VD} \text{ or } EQ_V}{79{,}779^K} + \underset{\Delta N_{CLAY}}{36{,}223^K} - 54{,}193\ K$$

$$N \tan\phi = 54{,}193^K \tan 38^\circ = 42{,}340\ K$$

$$\therefore FS_{SLIDING_{N\text{-}S}} = \frac{205.52^K/_{LF} \times \underset{B_{E\text{-}W}}{240'} + 7{,}379^K + 42{,}340^K}{0.4 \times 111{,}108\ K} = 1.78$$

$$FS_{SLIDING_{E\text{-}W}} = \frac{205.52 \frac{K}{LF} \times \overset{L}{279.5} + [illegible]6^K + 42{,}340\ K}{0.4 \times 99{,}997\ K} = 1.65 > 1.1$$

$\therefore$ OK

	N-S	VERT	E-W
CASE IIIB	40% IN X	−40% IN Y	100% IN Z
FROM TABLE 1	$0.4 \times 111{,}108\ K$	$-0.4 \times 79{,}779^K$	$99{,}997^K$ $L = V_{E\text{-}W}$

$$\therefore N = \underset{\text{CTB DL}}{97{,}749\ K} - \underset{F_{VD}}{0.4 \times 79{,}779^K} + \underset{\Delta N_{CLAY}}{36{,}223^K} = 102{,}060\ K$$

$$\Rightarrow T = \underbrace{\left(\overset{P_P}{180 \frac{K}{L.F.}} + 25.52 \frac{K}{L.F.}\right)_K \times \overset{L_{N\text{-}S}}{279.5'}}_{57{,}443} + 6{,}336\ K_{E\text{-}W}$$

$$+ \underbrace{\overset{N}{102{,}060}\ K \tan \overset{\phi}{38^\circ}}_{79{,}738} = 143{,}517\ K$$

$$FS = \frac{\text{RESISTING}}{\text{DRIVING}} = \frac{143{,}517^K}{99{,}997\ K} = 1.44 > 1.1$$

$\therefore$ O.K

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SLIDING ON DEEP PLANE

	N-S	VERT	E-W
CASE III C	100% IN X	−40% IN Y	40% IN Z
FROM TABLE I	111,108 K	−0.4 × 79,779 K	0.4 × 99,997

$$\therefore\ N = \overset{\text{CTR}\ ℄}{97,749} - \underset{31,912}{\overset{F_{VD}}{0.4 \cdot 79,779\,K}} + \overset{\Delta N_{CLAY}}{36,223} = 102,060\,K$$

$$T_{N\text{-}S} = 205.52\,\tfrac{K}{LF} \times \overset{B}{240'} + \overset{\Delta T_{N\text{-}S}}{7,779\,K} + 102,060 \tan 38^\circ = 136,442\,K$$

$$FS_{SLIDING} = \frac{T}{V_{N\text{-}S}} = \frac{136,442\,K}{111,108\,K} = 1.23 \quad > 1.1 \ \therefore OK$$

THE FACTOR OF SAFETY AGAINST SLIDING ON A DEEP PLANE OF COHESIONLESS SOIL IS > 1.1 FOR LOAD CASES III A, III B, & III C. THEREFORE THERE IS NO SLIDING ON A DEEP PLANE OF COHESIONLESS SOIL.

ALLOWABLE BEARING CAPACITY

Bearing capacity calculations are performed using the method for determining general bearing capacity failure, as presented in Winterkorn and Fang (1975). Local bearing capacity (punching shear) failure is ruled out due to the large size of the mat, 240' x 279.5'.

The general bearing capacity equation is a modification of Terzaghi's bearing capacity equation, which was developed for strip footings and which indicates that $q_{ult} = cN_c+qN_q+1/2\ \gamma BN_\gamma$. For this relationship, the ultimate bearing capacity of soil consists of three components: 1) cohesion, 2) surcharge, and 3) friction, which are represented by bearing capacity factors N_c, N_q, and N_γ. Terzaghi's bearing capacity equation has been enhanced by various investigators to incorporate shape, depth, and load inclination factors for different foundation geometries and loads as follows:

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

where

q_{ult} = ultimate bearing capacity

c = cohesion or undrained strength

q = effective surcharge at bottom of foundation, = γD_f

γ = unit weight of soil

B = foundation width

s_c, s_q, s_γ = shape factors, which are a function of foundation width to length

d_c, d_q, d_γ = depth factors, which account for embedment effects

i_c, i_q, i_γ = load inclination factors

N_c, N_q, N_γ = bearing capacity factors, which are a function of ϕ.

γ in the third term is the unit weight of soil below the foundation, whereas the unit weight of the soil above the bottom of the footing is used in determining q in the second term.

BEARING CAPACITY FACTORS

Bearing capacity factors computed based on relationships proposed by Vesic (1973), which are presented in Chapter 3 of Winterkorn and Fang (1975).

$$N_q = e^{\pi \tan\varphi} \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$N_c = (N_q - 1) \cot\phi \text{, but} = 5.14 \text{ for } \phi = 0.$$

$$N_\gamma = 2 \ (N_q + 1) \ \tan\phi$$

SHAPE FACTORS

$$s_c = 1 + \frac{B}{L} \cdot \frac{N_q}{N_c}$$

$$s_q = 1 + \frac{B}{L} \tan\phi$$

$$s_\gamma = 1 - 0.4 \cdot \frac{B}{L}$$

DEPTH FACTORS

For $\frac{D_f}{B} \leq 1$:

$$d_c = d_q - \frac{(1 - d_q)}{N_q \cdot \tan\phi} \text{ for } \phi > 0 \text{ and } d_c = 1 + 0.4\left(\frac{D_f}{B}\right) \text{ for } \phi = 0.$$

$$d_q = 1 + 2\tan\phi \cdot (1 - \sin\phi)^2 \cdot \left(\frac{D_f}{B}\right)$$

$$d_\gamma = 1$$

INCLINATION FACTORS

$$i_q = \left(1 - \frac{F_H}{F_V + B'L'c\cot\phi}\right)^m$$

$$i_c = i_q - \frac{(1 - i_q)}{N_c \cdot \tan\phi} \text{ for } \phi > 0 \text{ and } i_c = 1 - \left(\frac{m F_H}{B'L'c N_c}\right) \text{ for } \phi = 0$$

$$i_\gamma = \left(1 - \frac{F_H}{F_V + B'L'c\cot\phi}\right)^{m+1}$$

Where: F_H and F_V are the total horizontal and vertical forces acting on the footing and
$m_B =$ (2 + B/L) / (1 + B/L)
$m_L =$ (2 + L/B) / (1 + L/B)

STATIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

The following pages present the details of the bearing capacity analyses for the static load cases. These cases are identified as follows:

Case IA Static using undrained strength parameters ($\phi = 0°$ & c = 3.18 ksf).

Case IB Static using effective-stress strength parameters ($\phi = 30°$ & c = 0).

Table 2.6-9 presents the results of the bearing capacity analyses for these static load cases. The minimum factor of safety required for static load cases is 3.

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 6.5 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and c = 3.18 ksf, the average undrained strength for the soils in the upper layer at the site, to model the end of construction. Using the estimated effective-stress strength of $\phi = 30°$ and c = 0 results in higher allowable bearing pressures. As shown in Table 2.6-9, the gross allowable bearing capacity of the Canister Transfer Building for static loads for these soil strengths is 56.6 ksf.

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Static Analysis: **Case IA - Static** **0 % in N-S, 0 % in Vert 0 % in E-W**

Soil Properties:
$s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer
$\phi =$ **0** Friction Angle (degrees)
$\gamma =$ **90** Unit weight of soil (pcf)
$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:
$B' =$ **240.0** Footing Width - ft (E-W) $L' =$ **279.5** Length - ft (N-S)
$D_f =$ **5** Depth of Footing (ft)
$\beta =$ **0.0** Angle of load inclination from vertical (degrees)
$FS =$ **3** Factor of Safety required for $q_{allowable}$.
$F_V =$ **97,749** k $EQ_V =$ 0 k
$EQ_{H\ E\text{-}W} =$ **0** k + $EQ_{H\ N\text{-}S} =$ 0 k = 0 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.17	Table 3.2
$s_q = 1 + (B/L) \tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.66	"
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 $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.01	Eq 3.27

No inclined loads; therefore, $i_c = i_q = i_\gamma = 1.0$.

			N_c term		N_q term		N_γ term	
Gross q_{ult} =	19,635	psf =	19,235	+	400	+	0	
q_{all} =	6,540	psf = q_{ult} / FS						
q_{actual} =	1,457	psf = $(F_v + EQ_v) / (B' \times L')$						
FS_{actual} =	13.47	= q_{ult} / q_{actual}			>	3	Hence OK	

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Static Analysis: **Case** **IB - Static** | **0 % in N-S, 0 % in Vert 0 % in E-W**

Soil Properties:
- $s_u =$ **0** Average undrained strength (psf) in upper ~30' layer
- $\phi =$ **30** Friction Angle (degrees)
- $\gamma =$ **90** Unit weight of soil (pcf)
- $\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:
- $B' =$ **240.0** Footing Width - ft (E-W) $L' =$ **279.5** Length - ft (N-S)
- $D_f =$ **5** Depth of Footing (ft)
- $\beta =$ **0.0** Angle of load inclination from vertical (degrees)
- $FS =$ **3** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **97,749** k $EQ_V =$ **0** k
- $EQ_{H\ E\text{-}W} =$ **0** k + $EQ_{H\ N\text{-}S} =$ **0** k = **0** 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)

Expression		Value	Reference
$N_c = (N_q - 1)\cot(\phi)$, but $= 5.14$ for $\phi = 0$	=	30.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	=	18.40	Eq 3.6
$N_\gamma = 2 (N_q + 1) \tan(\phi)$	=	22.40	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.52	Table 3.2
$s_q = 1 + (B/L) \tan\phi$	=	1.50	"
$s_\gamma = 1 - 0.4 (B/L)$	=	0.66	"
For $D_f/B \le 1$: $d_q = 1 + 2 \tan\phi\ (1 - \sin\phi)^2 D_f/B$	=	1.01	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q) / (N_q \tan\phi)$	=	1.01	
For $\phi = 0$: $d_c = 1 + 0.4 (D_f/B)$	=	N/A	Eq 3.27

No inclined loads; therefore, $i_c = i_q = i_\gamma = 1.0$.

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**169,921**	**psf =**	**0**	**+**	**11,076**	**+**	**158,845**

q_{all} = 56,640 **psf = q_{ult} / FS**

q_{actual} = 1,457 **psf = $(F_v + EQ_v) / (B' \times L')$**

FS_{actual} = 116.61 **= q_{ult} / q_{actual}** **> 3 Hence OK**

DYNAMIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

The following pages present the details of the bearing capacity analyses for the dynamic load cases. These analyses use the dynamic loads for the building that were developed in Calculation 05996.02-SC-5, (S&W, 2001). The development of these dynamic loads is described in Section 4.7.1.5.3 of the SAR. As in the structural analyses discussed in SAR Section 4.7.1.5.3., the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions. The resulting dynamic loading cases are identified as follows:

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
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.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction.

Table 2.6-10 presents the results of the bearing capacity analyses for these cases, which include static loads plus dynamic loads due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the average undrained strength applicable for the soils within the upper layer (ϕ = 0° and c = 3.18 ksf). As indicated above, for these cases including dynamic loads from the design basis ground motion, the minimum acceptable factor of safety is 1.1.

Table 2.6-10 indicates the minimum factor of safety against a dynamic bearing capacity failure was obtained for Load Case II, the load combination of full static, 100% of the seismic forces acting in the N-S direction and the E-W direction and 0% in the upward direction. This load case resulted in an actual soil bearing pressure of 2.4 ksf, compared with an ultimate bearing capacity of 13.2 ksf. The resulting factor of safety against a bearing capacity failure for this load case is ~5.5, which is much greater than 1.1, the minimum allowable factor of safety for seismic loading cases. In these analyses, no credit was taken for the fact that strength of cohesive soil increases as the rate of loading increases. Therefore, the Canister Transfer Building has an adequate factor of safety against a dynamic bearing capacity failure.

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case II

100 % in N-S, 0 % in Vert 100 % in E-W

Soil Properties:	s_u =	**3,180**	Average undrained strength (psf) in upper ~30' layer
	ϕ =	**0**	Friction Angle (degrees)
	γ =	**90**	Unit weight of soil (pcf)
	γ_{surch} =	**80**	Unit weight of surcharge (pcf)
Foundation Properties:	B' =	**184.6**	Footing Width - ft (E-W) L' = **221.2** Length - ft (N-S)
	D_f =	**5**	Depth of Footing (ft)
	β =	**45.7**	Angle of load inclination from vertical (degrees)
	FS =	**1.1**	Factor of Safety required for $q_{allowable}$.
	F_V =	**97,749** k	EQ_V = **0** k
	$EQ_{H\ E\text{-}W}$ =	**99,997** k	+ $EQ_{H\ N\text{-}S}$ = **111,108** k = **149,480** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.16	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.67	"
For $D_f/B \leq 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.54	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.46	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.73	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.50	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.66	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**13,171**	**psf =**	**12,771**	**+**	**400**	**+**	**0**

q_{all} = 11,970 psf = q_{ult} / FS

q_{actual} = 2,394 psf = $(F_v + EQ_v) / (B' \times L')$

FS_{actual} = 5.50 = q_{ult} / q_{actual} **> 1.1 Hence OK**

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IIIA

40 % in N-S, -100 % in Vert 40 % in E-W

Soil Properties:

$s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer

$\phi =$ **0** Friction Angle (degrees)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **119.5** Footing Width - ft (E-W) $L' =$ **152.6** Length - ft (N-S)

$D_f =$ **5** Depth of Footing (ft)

$\beta =$ **65.8** Angle of load inclination from vertical (degrees)

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

$F_V =$ **97,749** k $EQ_V =$ **-79,779** k

$EQ_{H\ E\text{-}W} =$ **39,999** k + $EQ_{H\ N\text{-}S} =$ **44,443** k = **59,792** 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)

Equation		Value	Reference
$N_c = (N_q - 1)\cot(\phi)$, 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.15	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.69	"
For $D_f/B \leq 1$: $d_q = 1 + 2\tan\phi\,(1-\sin\phi)^2 D_f/B$	=	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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.02	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.54	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.46	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.73 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.50	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.70	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**13,804**	**psf =**	**13,404**	+	**400**	+	**0**

q_{all} = 12,540 psf = q_{ult} / FS

q_{actual} = 985 psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = 14.01 = q_{ult}/q_{actual} **> 1.1 Hence OK**

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IIIB — **40 % in N-S, -40 % in Vert 100 % in E-W**

Soil Properties:

s_u = **3,180** Average undrained strength (psf) in upper ~30' layer

ϕ = **0** Friction Angle (degrees)

γ = **90** Unit weight of soil (pcf)

γ_{surch} = **80** Unit weight of surcharge (pcf)

Foundation Properties:

B' = **157.8** Footing Width - ft (E-W) L' = **244.9** Length - ft (N-S)

D_f = **5** Depth of Footing (ft)

β = **56.6** Angle of load inclination from vertical (degrees)

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

F_V = **97,749** k EQ_V = **-31,912** k

$EQ_{H\ E\text{-}W}$ = **99,997** k + $EQ_{H\ N\text{-}S}$ = **44,443** k = **109,429** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.13	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.74	"
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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.54	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.46	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	1.15	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.53	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.74	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**14,103**	**psf =**	**13,703**	+	**400**	+	**0**
q_{all} =	**12,820**	**psf = q_{ult} / FS**					
q_{actual} =	**1,704**	**psf = $(F_v + EQ_v)/(B' \times L')$**					
FS_{actual} =	**8.28**	**= q_{ult}/q_{actual}**			**> 1.1 Hence OK**		

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CALCULATION IDENTIFICATION NUMBER				PAGE 39
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IIIC — **100 % in N-S, -40 % in Vert 40 % in E-W**

Soil Properties:

$s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer

$\phi =$ **0** Friction Angle (degrees)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **207.1** Footing Width - ft (E-W) $L' =$ **192.9** Length - ft (N-S)

$D_f =$ **5** Depth of Footing (ft)

$\beta =$ **31.3** Angle of load inclination from vertical (degrees)

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

$F_V =$ **97,749** k $EQ_V =$ **-31,912** k

$EQ_{H\,E\text{-}W} =$ **39,999** k + $EQ_{H\,N\text{-}S} =$ **111,108** k = **118,088** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.21	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.57	"
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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.54	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.46	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.35 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.47	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.73	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**15,045**	**psf =**	**14,645**	+	**400**	+	**0**

q_{all} = 13,670 psf = q_{ult} / FS

q_{actual} = 1,648 psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = 9.13 = q_{ult}/q_{actual} > **1.1 Hence OK**

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CALCULATION IDENTIFICATION NUMBER				PAGE 40
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 13-6	OPTIONAL TASK CODE N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IVA — **40 % in N-S, 100 % in Vert 40 % in E-W**

Soil Properties:

$s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer

$\phi =$ **0** Friction Angle (degrees)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **227.8** Footing Width - ft (E-W) $L' =$ **266.7** Length - ft (N-S)

$D_f =$ **5** Depth of Footing (ft)

$\beta =$ **12.7** Angle of load inclination from vertical (degrees)

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

$F_V =$ **97,749** k $EQ_V =$ **79,779** k

$EQ_{H\,E\text{-}W} =$ **39,999** k + $EQ_{H\,N\text{-}S} =$ **44,443** k = **59,792** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.17	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.66	"
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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.54	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.46	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.73	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.50	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.91	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**17,897**	**psf =**	**17,497**	**+**	**400**	**+**	**0**

q_{all} = 16,260 psf = q_{ult} / FS

q_{actual} = 2,923 psf = $(F_v + EQ_v) / (B' \times L')$

FS_{actual} = 6.12 = q_{ult} / q_{actual} **> 1.1 Hence OK**

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CALCULATION IDENTIFICATION NUMBER				PAGE 41
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IVB — 40 % in N-S, 40 % in Vert 100 % in E-W

Soil Properties:
- $s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer
- $\phi =$ **0** Friction Angle (degrees)
- $\gamma =$ **90** Unit weight of soil (pcf)
- $\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:
- $B' =$ **198.2** Footing Width - ft (E-W) $L' =$ **261.9** Length - ft (N-S)
- $D_f =$ **5** Depth of Footing (ft)
- $\beta =$ **37.6** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **97,749** k $EQ_V =$ **31,912** k
- $EQ_{H\,E\text{-}W} =$ **99,997** k + $EQ_{H\,N\text{-}S} =$ **44,443** k = **109,429** 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)

Equation		Value	Reference
$N_c = (N_q - 1)\cot(\phi)$, 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.15	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.70	"
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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.54	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.46	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	1.15 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.53	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.80	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**15,616**	**psf =**	**15,216**	+	**400**	+	**0**

q_{all} = 14,190 psf = q_{ult} / FS

q_{actual} = 2,497 psf = $(F_v + EQ_v) / (B' \times L')$

FS_{actual} = 6.25 = q_{ult} / q_{actual} > **1.1 Hence OK**

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CALCULATION IDENTIFICATION NUMBER				PAGE 42
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IVC

100 % in N-S, 40 % in Vert 40 % in E-W

Soil Properties:

s_u =	3,180	Average undrained strength (psf) in upper ~30' layer	
ϕ =	0	Friction Angle (degrees)	
γ =	90	Unit weight of soil (pcf)	
γ_{surch} =	80	Unit weight of surcharge (pcf)	

Foundation Properties:

B' =	223.3	Footing Width - ft (E-W)	L' = 235.5 Length - ft (N-S)
D_f =	5	Depth of Footing (ft)	
β =	17.1	Angle of load inclination from vertical (degrees)	
FS =	1.1	Factor of Safety required for $q_{allowable}$.	
F_V =	97,749 k	EQ_V =	31,912 k
$EQ_{H\,E\text{-}W}$ =	39,999 k	+ $EQ_{H\,N\text{-}S}$ =	111,108 k = 118,088 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.18	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.62	"
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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.54	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.46	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.35	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.47	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.80	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**15,987**	**psf =**	**15,587**	**+**	**400**	**+**	**0**

q_{all} = 14,530 psf = q_{ult} / FS

q_{actual} = 2,465 psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = 6.49 = q_{ult}/q_{actual} **> 1.1 Hence OK**

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CONCLUSIONS

Overturning Stability of the Canister Transfer Building

The overturning stability of the Canister Transfer Building is analyzed on Pages 14 to 17 using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads, listed in Table 2.6-11, were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (S&W, 2001) and are described in SAR Section 4.7.1.5.3. This calculation demonstrates that the factor of safety against overturning of the Canister transfer Building is >1.1; therefore, the Canister Transfer Building has an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion. The minimum factor of safety against overturning is 1.95, and it applies to overturning about the north-south axis.

Sliding Stability of the Canister Transfer Building

The Canister Transfer Building (CTB) will be founded on clayey soils. The sliding stability of the CTB was evaluated using the loads developed in Calculation 05996.02-SC-5 (S&W, 2001). The static strength of the clayey soils at the bottom of the CTB mat was based on the average of two sets of direct shear tests performed on samples of soils obtained from beneath the Canister Transfer Building at the elevation proposed for founding the mat.

The results of the sliding stability analysis are presented in Table 2.6-13 of this calculation, and they indicate that for all load combinations examined, the factors of safety were acceptable. The lowest factor of safety was 1.15, which applies for Case IIIC, where 100% of the dynamic earthquake forces act in the N-S direction and 40% act in the other two directions. These results assume that only one-half of the passive pressures are available resist sliding and no credit is taken for the fact that the strength of cohesive soils increases as the rate of loading increases (Newmark and Rosenblueth, 1971, Schimming et al, 1966, Casagrande and Shannon, 1948, and Das, 1993); therefore, they represent a conservative lower-bound value of the sliding stability of the Canister Transfer Building founded on in situ silty clay/clayey silt with 5 ft of soil-cement backfill around the foundation.

Additional sliding stability analyses are included that demonstrate that there is additional margin available to resist sliding of the building due to the earthquake loads. In these analyses, it is recognized that the ultimate sliding failure of the building cannot occur until after the full passive resistance of the soil cement adjacent to the mat is exceeded. These analyses use a very conservative estimate of the residual shear strength of the clayey soils under the building, based on the results of the direct shear tests that were performed on specimens of the soils obtained from approximately the elevation of the potential sliding plane under the building. The results of these analyses are presented in Table 2.6-14, and they demonstrate that the factor of safety against sliding is at least 1.26.

The Canister Transfer Building, founded on clayey soils and with the soil-cement backfill, has an adequate factor of safety against sliding due to the dynamic forces from the design

basis ground motion. As shown in SAR Figures 2.6-21 through 2.6-23, however, some of the soils underlying the building are cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. Simplified analyses were performed to address the possibility that sliding may occur along a deeper slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

These analyses included the passive resistance acting on a plane extending from grade down to the top of the cohesionless layer, plus the frictional resistance available along the top of the cohesionless layer. The weight of the clayey soils existing between the top of the cohesionless soils and the bottom of the mat was included in the normal force used to calculate the frictional resistance acting along the top of the cohesionless layer. The factor of safety against sliding along the top of this layer was found to be >1.1 for all of the dynamic load cases; therefore, there is an adequate factor of safety against sliding along the surface of the cohesionless soils underlying the Canister Transfer Building.

BEARING CAPACITY

STATIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

Table 2.6-9 presents the results of the bearing capacity analyses for the following static load cases. The minimum factor of safety required for static load cases is 3.

Case IA Static using undrained strength parameters ($\phi = 0°$ & c = 3.18 ksf).

Case IB Static using effective-stress strength parameters ($\phi = 30°$ & c = 0).

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 6.5 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and c = 3.18 ksf, the average undrained strength for the soils in the upper layer at the site, to model the end of construction. Using the estimated effective-stress strength of $\phi = 30°$ and c = 0 results in higher allowable bearing pressures. As shown in Table 2.6-9, the gross allowable bearing capacity of the Canister Transfer Building for static loads for these soil strengths is 56.6 ksf.

DYNAMIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

The dynamic bearing capacity was analyzed using the dynamic loads for the building that were developed in Calculation 05996.02-SC-5, (S&W, 2001). The development of these dynamic loads is described in SAR Section 4.7.1.5.3. As in the structural analyses discussed in Section 4.7.1.5.3, the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions.

Table 2.6-10 presents the results of the bearing capacity analyses for the following cases, which include static loads plus dynamic loads due to the earthquake. The minimum factor of safety required for dynamic load cases is 1.1.

CALCULATION IDENTIFICATION NUMBER				PAGE 45
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
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.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction.

Table 2.6-10 indicates the minimum factor of safety against a dynamic bearing capacity failure was obtained for Load Case II, the load combination of full static, 100% of the seismic forces acting in the N-S direction and the E-W direction and 0% in the upward direction. This load case resulted in an actual soil bearing pressure of 2.4 ksf, compared with an ultimate bearing capacity of 13.2 ksf. The resulting factor of safety against a bearing capacity failure for this load case is ~5.5, which is much greater than 1.1, the minimum allowable factor of safety for seismic loading cases. In these analyses, no credit was taken for the fact that strength of cohesive soil increases as the rate of loading increases. Therefore, the Canister Transfer Building has an adequate factor of safety against a dynamic bearing capacity failure

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TABLE 2.6-9
SUMMARY - ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING
Based on Static Loads

Case	F_V	$EQ_{H\,N\text{-}S}$	$EQ_{H\,E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\,E\text{-}W}$	β_L $EQ_{H\,N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
IA - Static Undrained Strength	97,749	0	0	0	0	0.0	0.0	19.63	**6.54**	0.0	0.0	240.0	279.5	**1.46**	**13.47**
IB - Static Effective Strength	97,749	0	0	0	0	0.0	0.0	169.92	**56.64**	0.0	0.0	240.0	279.5	**1.46**	**116.61**

$c =$	**3,180**	Undrained strength (psf) & $\phi = 0$.
$\phi =$	**30.0**	Effective stress friction angle (deg), c =
$B =$	**240.0**	Footing width (ft)
$L =$	**279.5**	Footing length (ft)
$D_f =$	**5.0**	Depth of footing (ft)
$\gamma =$	**90**	Unit weight of soil (pcf)
$\gamma_{surch} =$	**80**	Unit weight of surcharge (pcf)
$FS =$	**3**	Factor of safety for static loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\qquad e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2e_B$ $\qquad L' = L - 2e_L$

$q_{actual} = F_V / (B' \times L')$

[geot]\05996\calc\brng_cap\can_xfr.xls Table 2.6-9

TABLE 2.6-10

SUMMARY - ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Based on Dynamic Loads Due to Design Basis Ground Motion: PSHA 2,000-yr Return Period

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\ E\text{-}W}$	β_L $EQ_{H\ N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
II	97,749	111,108	99,997	2,706,961	2,849,703	45.7	48.7	13.17	**11.97**	27.7	29.2	184.6	221.2	**2.39**	**5.50**
IIIA	17,970	44,443	39,999	1,082,784	1,139,881	65.8	68.0	13.80	**12.54**	60.3	63.4	119.5	152.6	**0.99**	**14.01**
IIIB	65,837	44,443	99,997	2,706,961	1,139,881	56.6	34.0	14.10	**12.82**	41.1	17.3	157.8	244.9	**1.70**	**8.28**
IIIC	65,837	111,108	39,999	1,082,784	2,849,703	31.3	59.4	15.04	**13.67**	16.4	43.3	207.1	192.9	**1.65**	**9.13**
IVA	177,528	44,443	39,999	1,082,784	1,139,881	12.7	14.1	17.90	**16.26**	6.1	6.4	227.8	266.7	**2.92**	**6.12**
IVB	129,661	44,443	99,997	2,706,961	1,139,881	37.6	18.9	15.62	**14.19**	20.9	8.8	198.2	261.9	**2.50**	**6.25**
IVC	129,661	111,108	39,999	1,082,784	2,849,703	17.1	40.6	15.99	**14.53**	8.4	22.0	223.3	235.5	**2.47**	**6.49**

$c =$ **3,180** Undrained strength (psf)

$\phi =$ **0.0** Friction angle (deg)

$B =$ **240.0** Footing width (ft)

$L =$ **279.5** Footing length (ft)

$D_f =$ **5.0** Depth of footing (ft)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

$FS =$ **1.1** Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E\text{-}W}$ or $EQ_{H\ N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\ E\text{-}W}) / F_V]$ = Angle of load inclination from vertical as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\ N\text{-}S}) / F_V]$ = Angle of load inclination from vertical as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\quad e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\,e_B$ $\quad L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

[geotl]\05996\calc\brng_cap\can_xfr.xls Table 2.6-10

Table 2.6-11

Foundation Loadings for the Canister Transfer Building

JOINT	ELEV	MASS X	MASS Y	MASS Z	Ax	Ay	Az	SHEAR X $F_{H\ N\text{-}S}$	SHEAR Y $F_{V\ Dyn}$	SHEAR Z $F_{H\ E\text{-}W}$	$\Sigma M_{Base\ @\ El\ 93.5}$ $M_{@X}=M_{@N\text{-}S}$	$M_{@Z}=M_{@E\text{-}W}$
	ft	k-sec^2 / ft	k-sec^2 / ft	k-sec^2 / ft	g	g	g	k	k	k	ft-k	ft-k
0	94.25	260.1	260.1	260.1	1.047	0.78	0.92	8,761	6,551	7,699	5,774	6,571
1	95	1,908.0	1,908.0	1,908.0	1.047	0.78	0.92	64,265	48,055	56,470	367,055	417,724
2	130	420.4	420.4	420.4	1.111	0.82	0.99	15,023	11,106	13,446	490,773	548,331
3	170	304.3	304.3	170.3	1.778	0.91	1.19	17,402	8,939	6,493	496,728	1,331,291
4	190	144.7	117.1	144.7	1.215	0.93	1.41	5,656	3,495	6,554	632,439	545,787
5	190	1.0	27.6	1.0	0	1.84	0.00	0	1,634	0	0	0
6	170	1.0	1.0	134.0	0	0	2.17	0	0	9,336	714,193	0
							TOTALS	111,108	79,779	99,997	2,706,961	2,849,703

B = 240.0 ft

L = 279.5 ft WEIGHT 97,749 k FS_{UPLIFT} = 1.23

Depth = 5.0 ft + 1.5 ft deep key with base at Elev 93.5 ft

Note: Elevations are referenced to assumed final grade of Elev 100.
Joint 0 equals clayey soils enclosed by perimeter key with γ = 90 pcf and width of key = 6.5 ft.
Based on masses and accelerations from p 37 of Calc 05996.02-SC-5, Rev. 2, which are applicable for "High" Moduli received from Geomatrix Calc 05996.02-G(PO18)-2, Rev. 1.

|geot|\05996\Calc\brng_cap\CTB\Stability.xls "Dyn Loads"

Table 2.6-13

Sliding Stability of Canister Transfer Building Using Shear Strength Along Bottom of Plane Formed by 1.5-ft Deep Perimeter Key and Half of Resistance from Soil Cement Using Peak Strength of Clay

Joint	MASS X	MASS Y	MASS Z	N-S	Vert	E-W	Static	Earthquake		
				a_x	a_y	a_z	F_v	$Shear_{N-S}$	F_v	$Shear_{E-W}$
	$k\text{-}sec^2/ft$	$k\text{-}sec^2/ft$	$k\text{-}sec^2/ft$	g	g	g	k	k	k	k
0	260.1	260.1	260.1	1.047	0.783	0.920	8,368	8,761	6,551	7,699
1	1,908.0	1,908.0	1,908.0	1.047	0.783	0.920	61,380	64,265	48,055	56,470
2	420.4	420.4	420.4	1.111	0.821	0.994	13,524	15,023	11,106	13,446
3	304.3	304.3	170.3	1.778	0.913	1.185	9,789	17,402	8,939	6,493
4	144.7	117.1	144.7	1.215	0.928	1.408	3,767	5,656	3,495	6,554
5	1.0	27.6	1.0	0.000	1.840	0.000	888	0	1,634	0
6	1.0	1.0	134.0	0.000	0.000	2.166	32	0	0	9,336
CTB Mat Dimensions:			B =	240.0	ft (E-W)	*Totals* =	97,749	111,108	79,779	99,997
Depth =	5	ft	L =	279.5	ft (N-S)			*Resisting*	*Driving*	

For ϕ = 0.0 degrees		c = 1.70				N (*k*)	T (*k*)	V (*k*)	FS
Earthquake Vertical Forces Acting Up	IIIA	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	100% $F_{v(Eqk)}$ -79,779	40% $F_{H(EW)}$ 39,999	17,970	135,999	59,792	2.27
	IIIB	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	40% $F_{v(Eqk)}$ -31,912	100% $F_{H(EW)}$ 99,997	65,837	135,999	109,429	1.24
	IIIC	$F_{v(Static)}$ 97,749	100% $F_{H(NS)}$ 111,108	40% $F_{v(Eqk)}$ -31,912	40% $F_{H(EW)}$ 39,999	65,837	135,999	118,088	1.15
Earthquake Vertical Forces Acting Down	IVA	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	100% $F_{v(Eqk)}$ 79,779	40% $F_{H(EW)}$ 39,999	177,529	135,999	59,792	2.27
	IVB	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	40% $F_{v(Eqk)}$ 31,912	100% $F_{H(EW)}$ 99,997	129,661	135,999	109,429	1.24
	IVC	$F_{v(Static)}$ 97,749	100% $F_{H(NS)}$ 111,108	40% $F_{v(Eqk)}$ 31,912	40% $F_{H(EW)}$ 39,999	129,661	135,999	118,088	1.15
Soil Cement ΔF_H for q_u (psi) = 250			21,600	N/A	25,155	for FS_{SC} = 2.0			

Table 2.6-14

Sliding Stability of Canister Transfer Building Using Shear Strength Along Bottom of Plane Formed by 1.5-ft Deep Perimeter Key and Resistance from Soil Cement Using Residual Strength = 80% of Peak Strength of

Joint	MASS X $k\text{-}sec^2/ft$	MASS Y $k\text{-}sec^2/ft$	MASS Z $k\text{-}sec^2/ft$	N-S a_x g	Vert a_y g	E-W a_z g	Static F_v k	Earthquake $Shear_{N\text{-}S}$ k	Earthquake F_v k	Earthquake $Shear_{E\text{-}W}$ k
0	260.1	260.1	260.1	1.047	0.783	0.920	8,368	8,761	6,551	7,699
1	1,908.0	1,908.0	1,908.0	1.047	0.783	0.920	61,380	64,265	48,055	56,470
2	420.4	420.4	420.4	1.111	0.821	0.994	13,524	15,023	11,106	13,446
3	304.3	304.3	170.3	1.778	0.913	1.185	9,789	17,402	8,939	6,493
4	144.7	117.1	144.7	1.215	0.928	1.408	3,767	5,656	3,495	6,554
5	1.0	27.6	1.0	0.000	1.840	0.000	888	0	1,634	0
6	1.0	1.0	134.0	0.000	0.000	2.166	32	0	0	9,336
CTB Mat Dimensions:			**B =**	**240.0**	**ft (E-W)**	***Totals =***	**97,749**	**111,108**	**79,779**	**99,997**
Depth =	**5**	**ft**	**L =**	**279.5**	**ft (N-S)**			***Resisting***	***Driving***	

For ϕ =	**0.0**	**degrees**	**c =**	**1.36**			**N (*k*)**	**T (*k*)**	**V (*k*)**	**FS**
Earthquake Vertical Forces Acting Up	**IIIA**	$F_{v(Static)}$	40% $F_{H(NS)}$	100% $F_{v(Eqk)}$	40% $F_{H(EW)}$					
		97,749	44,443	-79,779	39,999	17,970	148,586	59,792	**2.49**	
	IIIB	$F_{v(Static)}$	40% $F_{H(NS)}$	40% $F_{v(Eqk)}$	100% $F_{H(EW)}$					
		97,749	44,443	-31,912	99,997	65,837	148,586	109,429	**1.36**	
	IIIC	$F_{v(Static)}$	100% $F_{H(NS)}$	40% $F_{v(Eqk)}$	40% $F_{H(EW)}$					
		97,749	111,108	-31,912	39,999	65,837	148,586	118,088	**1.26**	
Earthquake Vertical Forces Acting Down	**IVA**	$F_{v(Static)}$	40% $F_{H(NS)}$	100% $F_{v(Eqk)}$	40% $F_{H(EW)}$					
		97,749	44,443	79,779	39,999	177,529	148,586	59,792	**2.49**	
	IVB	$F_{v(Static)}$	40% $F_{H(NS)}$	40% $F_{v(Eqk)}$	100% $F_{H(EW)}$					
		97,749	44,443	31,912	99,997	129,661	148,586	109,429	**1.36**	
	IVC	$F_{v(Static)}$	100% $F_{H(NS)}$	40% $F_{v(Eqk)}$	40% $F_{H(EW)}$					
		97,749	111,108	31,912	39,999	129,661	148,586	118,088	**1.26**	
Soil Cement ΔF_H for q_u (psi) = 250			43,200	N/A	50,310	**for FS_{SC} = 1.0**				

STONE & WEBSTER, INC.
CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE
05996.02	G(B)	13-6	N/A

5010.65

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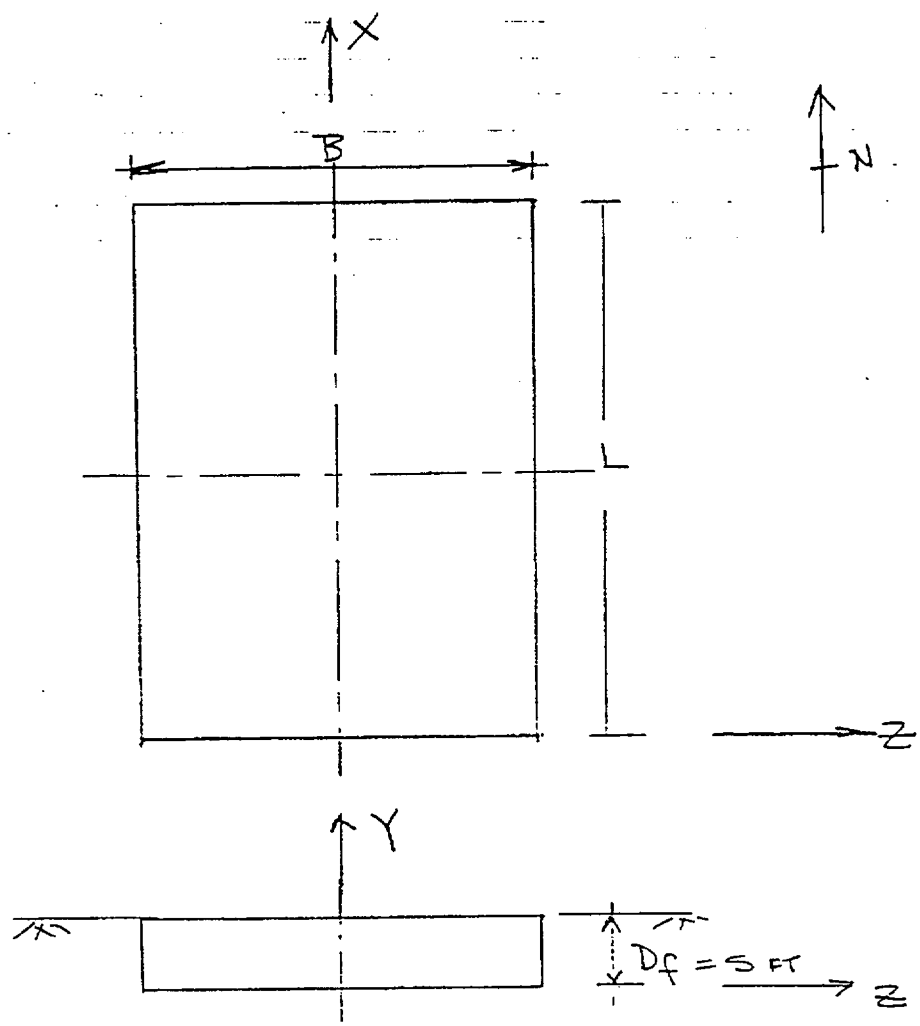
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

FIGURE 1

FOUNDATION SCHEMATIC & COORDINATE SYSTEM



Note: The coordinate system is consistent with that used in Calculation 05996.02-SC-5.

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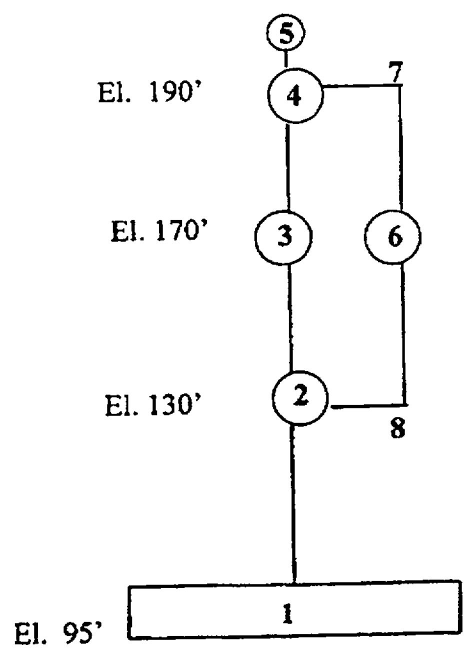
05996.02	G(B)	13-6	N/A	

FIGURE 2

CANISTER TRANSFER BUILDING STICK MODEL



Note: From Calculation 05996.02-SC-5, Rev. 2, Page 8.

STONE & WEBSTER, INC.

5010.65

CALCULATION SHEET

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J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

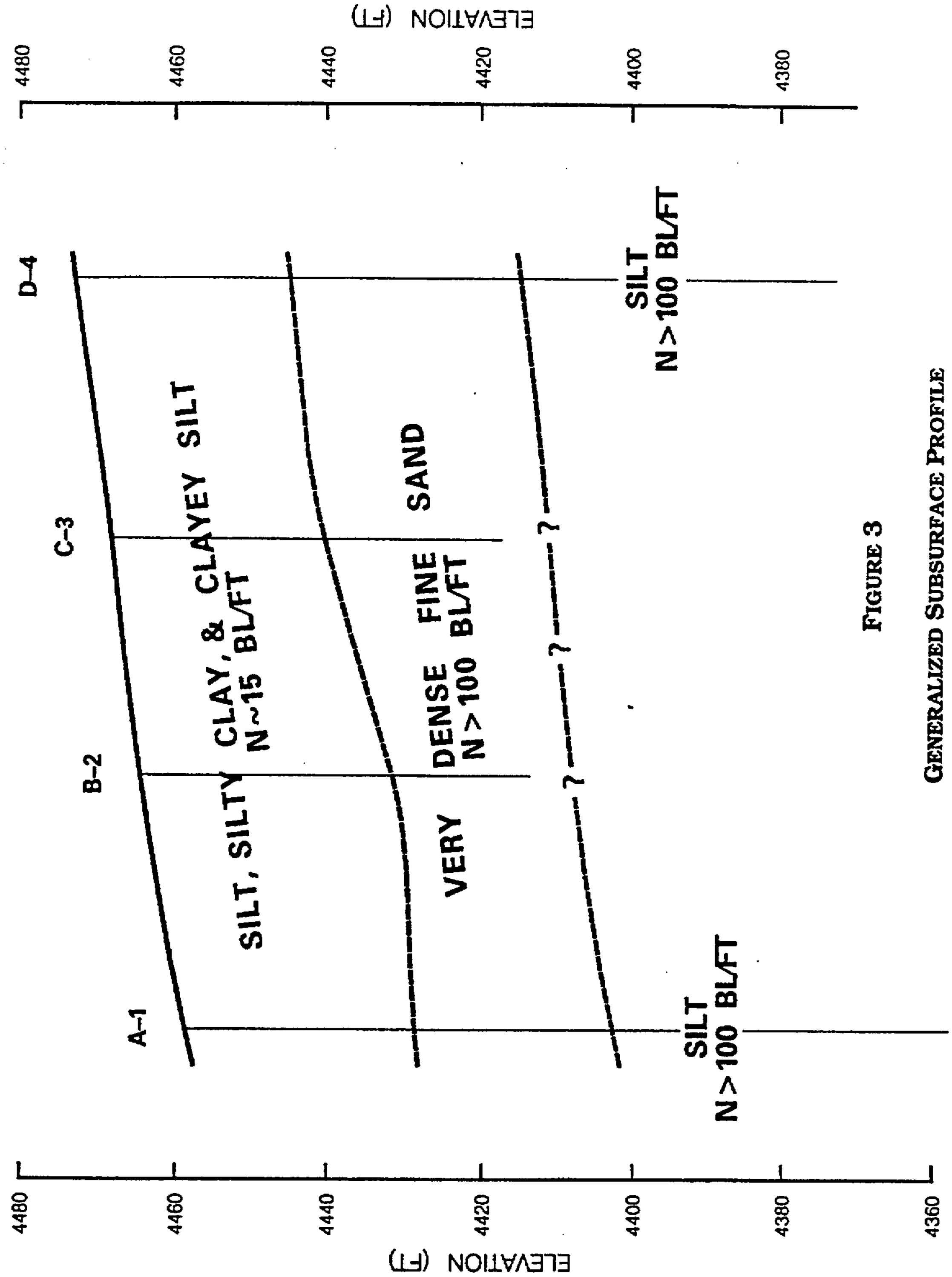

FIGURE 3

GENERALIZED SUBSURFACE PROFILE

Note: From Calculation 05996.01-G(B)-03-1.

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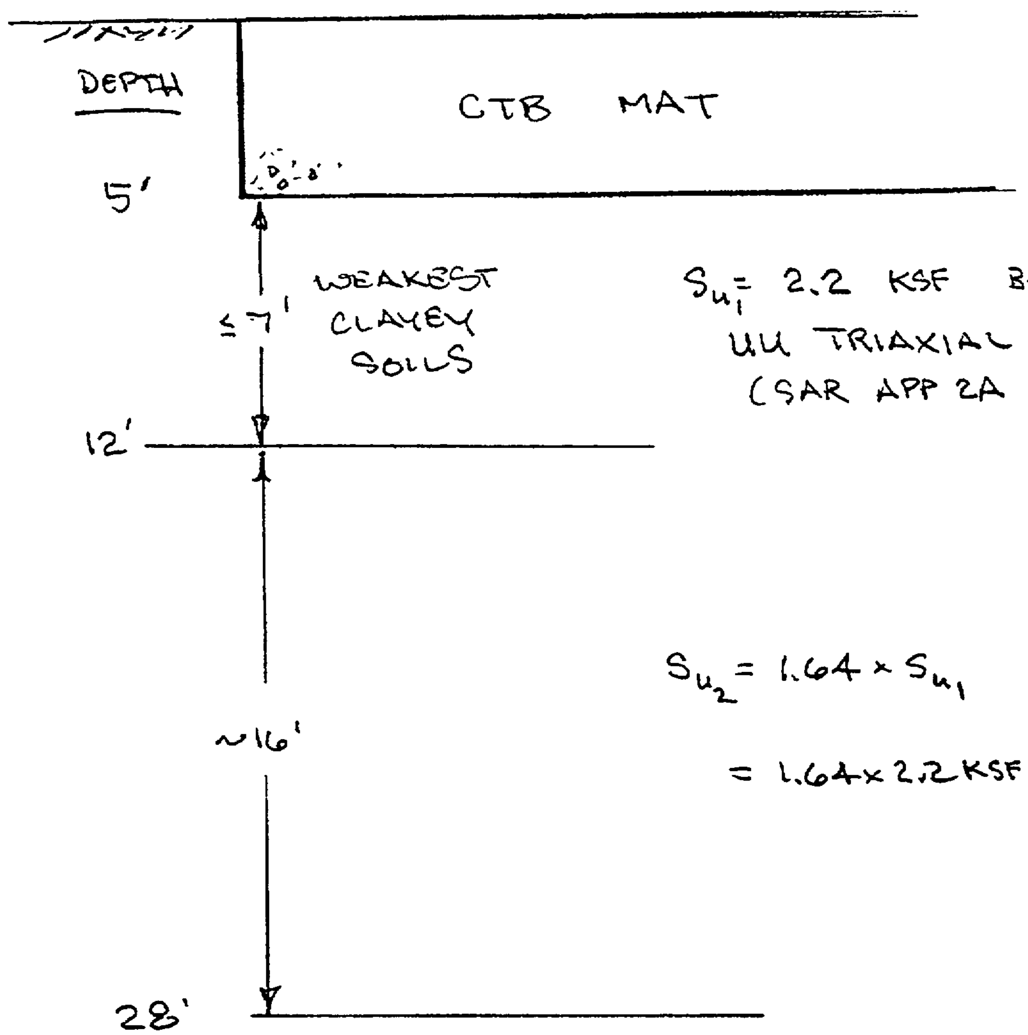
05996.02	G(B)	13-6	N/A	

FIGURE 4

DETERMINATION OF WEIGHTED AVERAGE VALUE OF S_U BASED ON RELATIVE STRENGTH DIFFERENCE OF DEEPER LYING SOILS MEASURED IN CONE PENETRATION TESTS



$S_{u_1} = 2.2$ KSF BASED ON UU TRIAXIAL TESTS (SAR APP 2A ATT 2)

$S_{u_2} = 1.64 \times S_{u_1}$

$= 1.64 \times 2.2\text{ KSF} = 3.6\text{ KSF}$

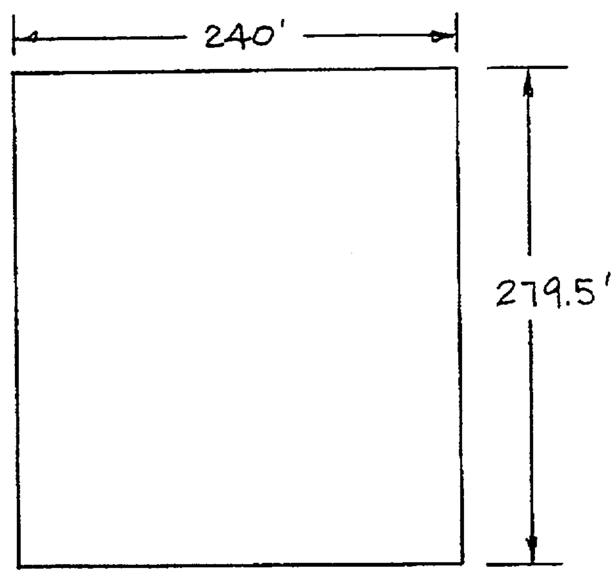
$$S_{u_{AVG}} = \frac{7' \times 2.2\text{ KSF} + 16' \times 1.64 \times 2.2\text{ KSF}}{23\text{ FT}} = 3.18\text{ KSF}$$

FIGURE 5

ESTIMATE STRESSES UNDER THE CANISTER TRANSFER BUILDING AT COMPLETION OF CONSTRUCTION

F_V = Total dead weight = 97,749 K from Table 2.6-11

A = Area of mat = 240ft x 279.5 ft = 67,080 ft^2



PLAN

$$\sigma = \frac{F_V}{A} = \frac{97{,}749\,K}{67{,}080\,K} = 1.46\,ksf$$

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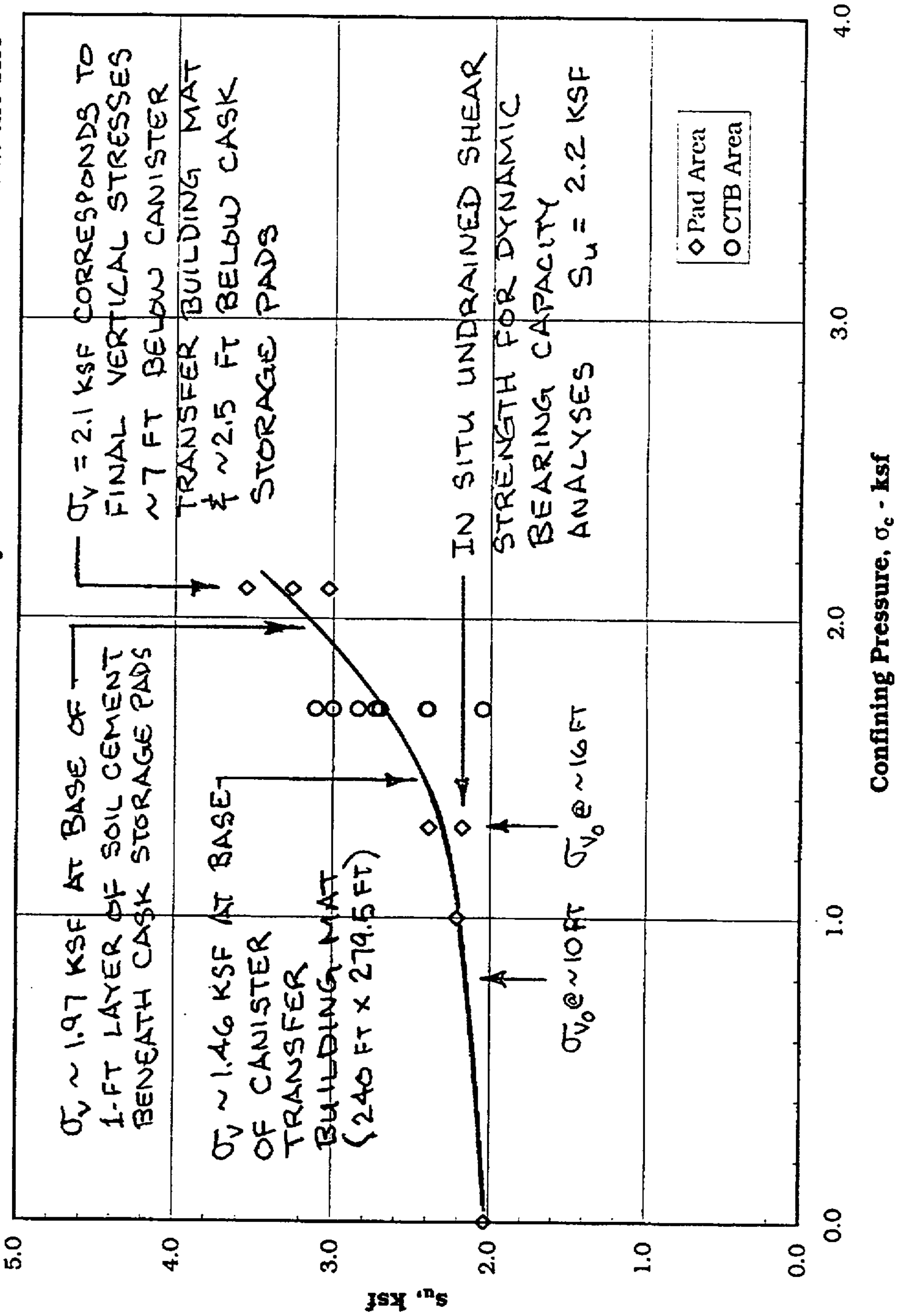
05996.02	G(B)	13-6	N/A	

Figure 6

Summary of Triaxial Test Results for Soils Within Depth of ~10 ft of Ground Surface at the Site



[geot]\05996\lab\triax-1.xls on 5/31/2001

5010.65

STONE & WEBSTER, INC.

CALCULATION SHEET

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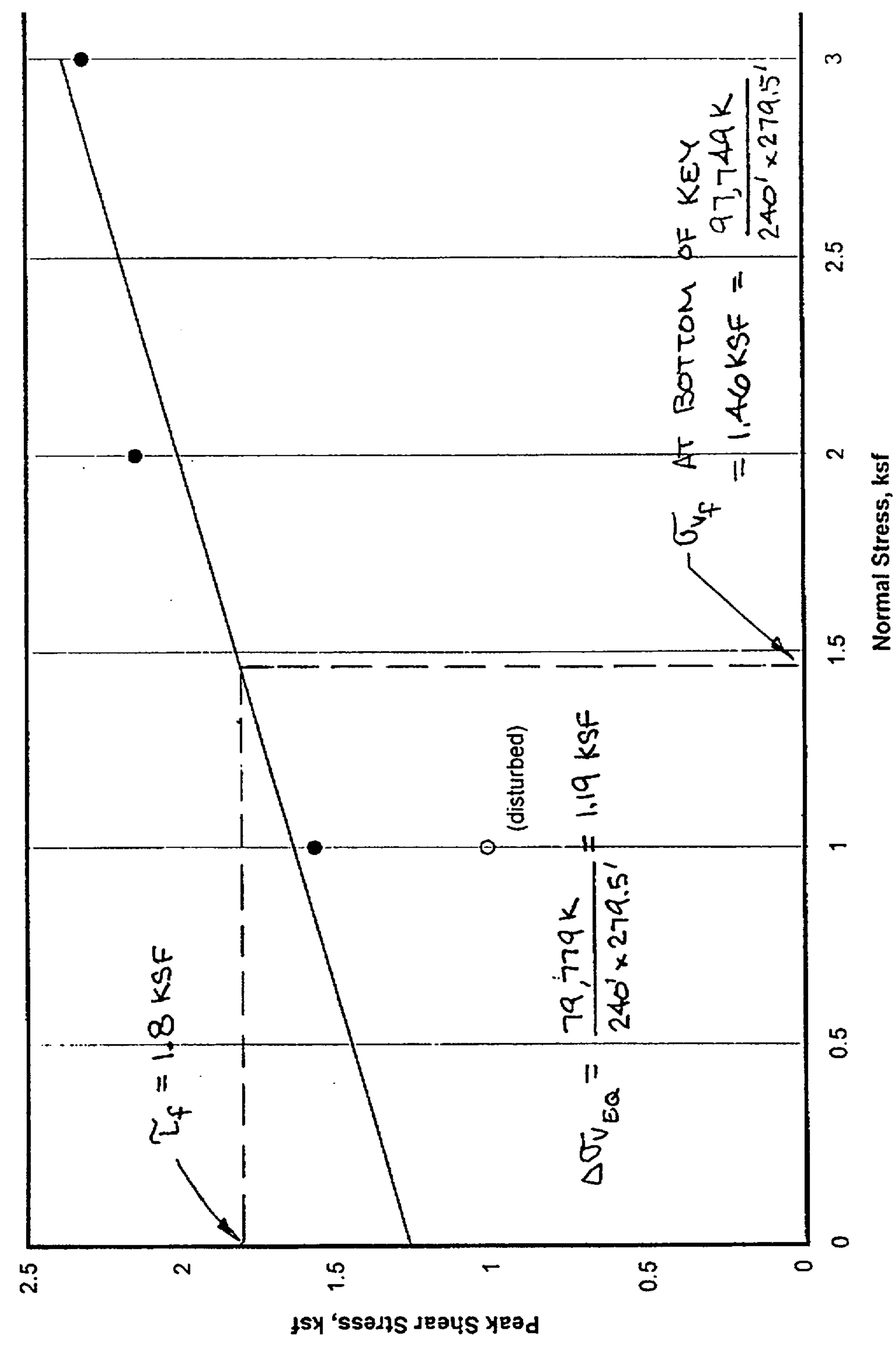
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

FIGURE 7

DIRECT SHEAR TEST

Boring CTB-6, Sample U-3B&C

CANISTER TRANSFER BUILDING AREA



PFSF SKULL VALLEY, UT
SAR APP 2A ATT 7

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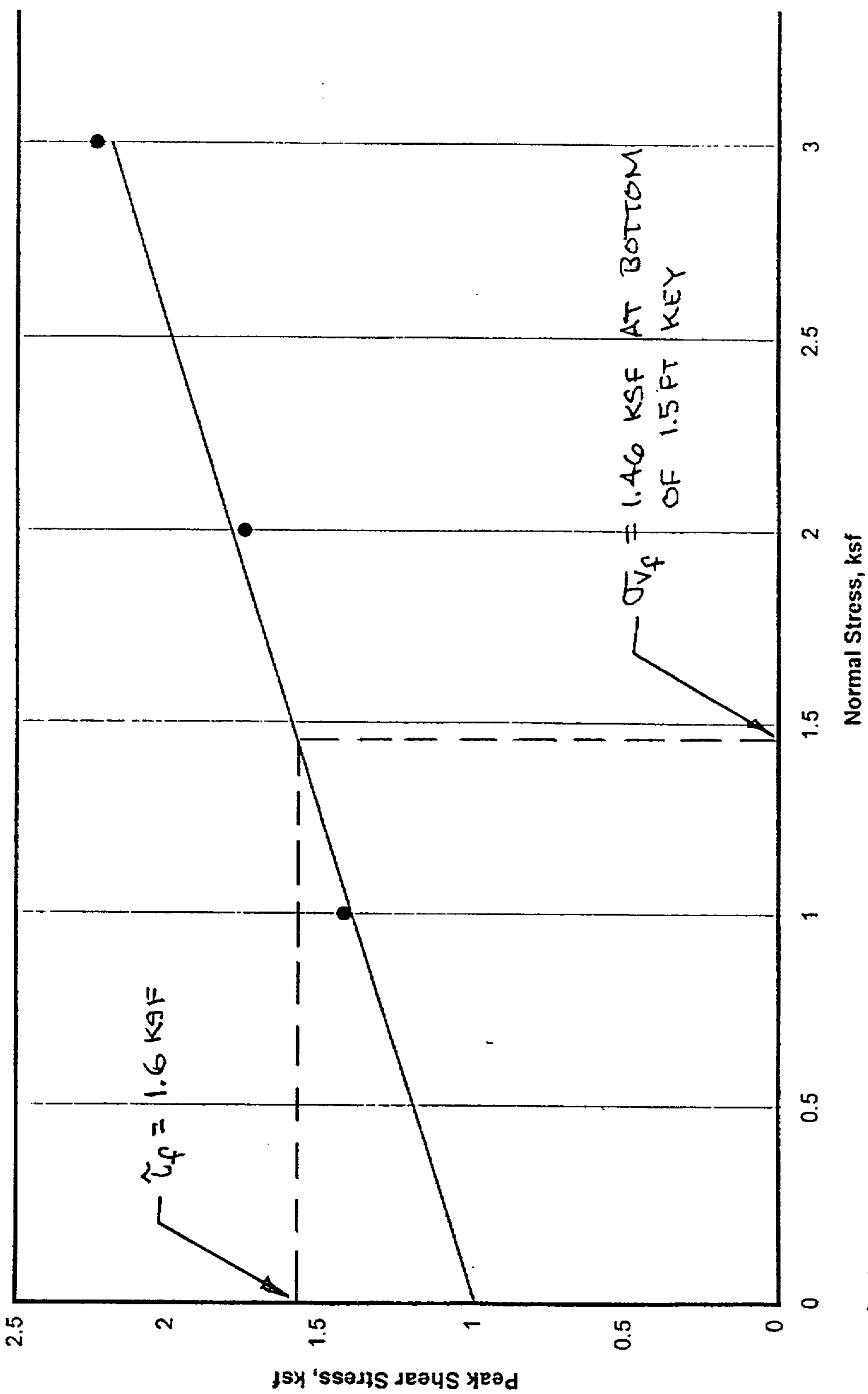
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

FIGURE 8
DIRECT SHEAR TEST
Boring CTB-S, Sample U-1AA
CANISTER TRANSFER BUILDING AREA



PFSF SKULL VALLEY, UT
SAR APP 2A ATT 7

TABLE 6
SUMMARY OF TRIAXIAL TEST RESULTS FOR SOILS WITHIN ~10 FT OF GROUND SURFACE AT THE SITE

Boring	Sample	Depth ft	Elev ft	W %	ATTERBERG LIMITS LL	PL	PI	USC Code	γ_m pcf	γ_d pcf	e_o	σ_c ksf	s_u ksf	ε_a %	Type	Date
B-1	U-2C	5.9	4453.9	47.1	66.1	33.4	32.7	MH	79.3	53.9	2.15	0.0	2.03	1.7	CU	Nov '99
B-1	U-2B	5.3	4454.5	52.9	80.6	40.9	39.7	MH	70.8	46.3	2.67	1.0	2.21	6.0	CU	Nov '99
B-4	U-3D	10.4	4462.1	27.4	42.5	24.7	17.8	CL	85.5	67.1	1.53	1.3	2.18	4.0	UU	Jan '97
C-2	U-2D	11.1	4453.4	35.6	See U-2C & E[1]			CL	78.5	57.9	1.93	1.3	2.39	11.0	UU	Jan '97
CTB-1	U-3D	8.7	4463.7	47.9	See U-3C[2]			CH	91.9	62.1	1.73	1.7	2.84	5.0	CU	June '99
CTB-4	U-2D	9.5	4465.5	45.2	See U-2E[2]			CH	87.7	60.4	1.81	1.7	3.11	6.0	CU	June '99
CTB-6	U-3D	8.3	4467.9	52.7				CH	85.7	56.2	2.02	1.7	2.70	7.0	CU	June '99
CTB-N	U-1B	5.7	4468.4	30.1	41.3	22.5	18.8	CL	100.6	77.3	1.20	1.7	3.00	8.0	CU	Nov '98
CTB-N	U-2B	7.7	4466.4	65.4	See U-2A[2]			MH	74.6	45.1	2.76	1.7	2.41	13.0	CU	June '99
CTB-N	U-3D	10.5	4463.6	52.2	61.1	30.8	30.3	CH	86.3	56.7	1.98	1.7	2.73	7.0	CU	June '99
CTB-S	U-1B	5.8	4468.7	73.6	66.2	40.9	25.3	MH	78.0	44.9	2.78	1.7	2.05	12.0	CU	Nov '98
CTB-S	U-2D	8.4	4466.1	54.6	57.9	28.9	29.0	CH	90.0	58.2	1.92	1.7	2.40	5.0	CU	June '99
B-1	U-2D	6.5	4453.3	45.2	59.8	34.7	25.1	MH	76.7	52.8	2.22	2.1	3.26	15.0	CU	Mar '99
B-3	U-1B	5.2	4463.0	33.5	52.4	25.2	27.2	MH	90.6	67.9	1.50	2.1	3.55	8.0	CU	Mar '99
C-2	U-1D	6.3	4458.2	50.5	70.3	41.3	29.0	MH	74.5	49.5	2.43	2.1	3.03	12.0	CU	Mar '99

NOTES 1 Attachment 2 of SAR Appendix 2A.
2 Attachment 6 of SAR Appendix 2A.

ATTACHMENT A p A1/1
TO CALC 05996.02-G(B)-13-6

[geot]\05996\calc\G(B)\05-2\Table_6.xls

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ATTACHMENT B p B1/2

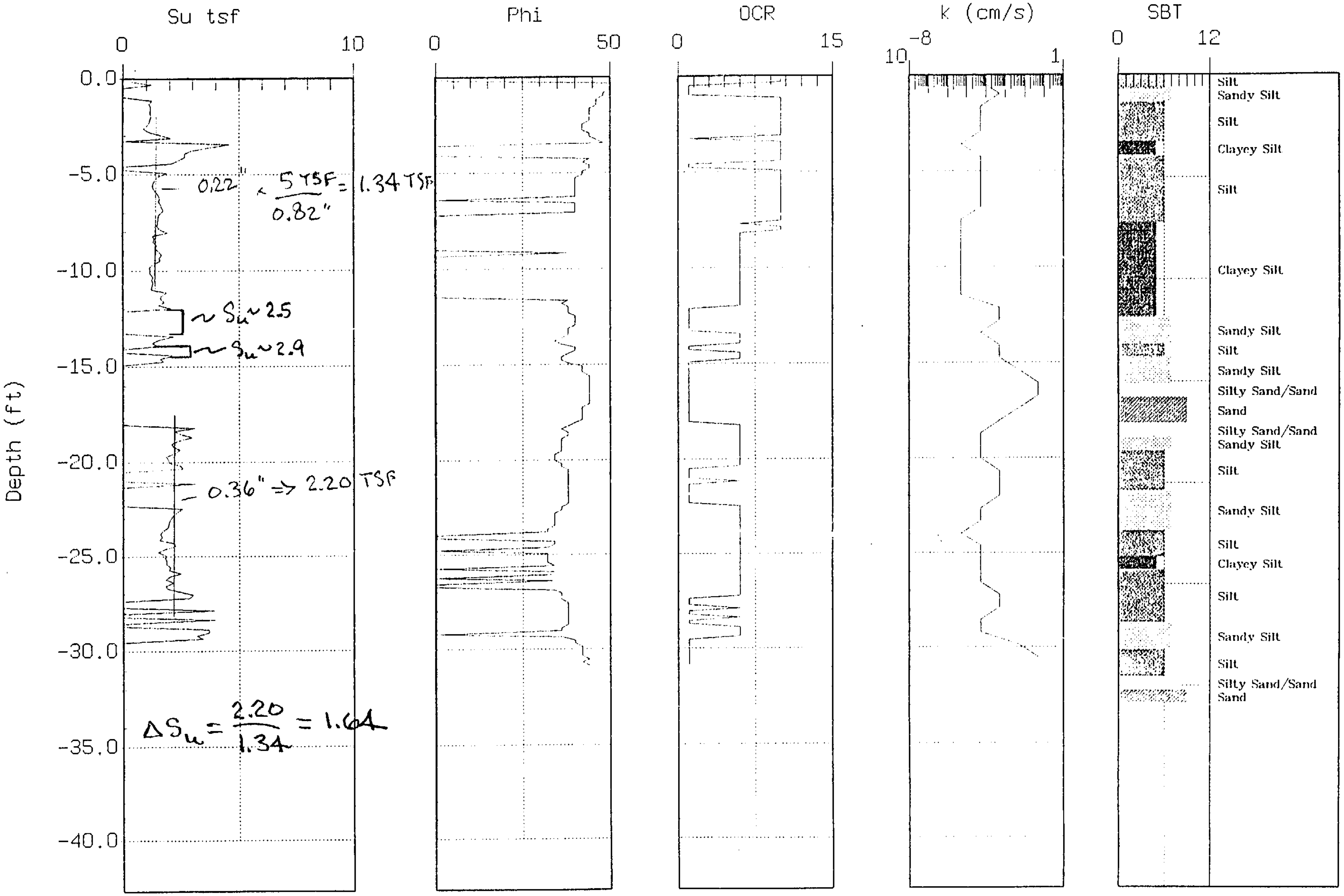
CALC 05996.02-G(B)-13-6

CONETEC

Stone & Webster

Site: CPT-37
Location: PFSF (05996.02)

Cone: 20 TON A 041
Date: 04:23:99 11:36



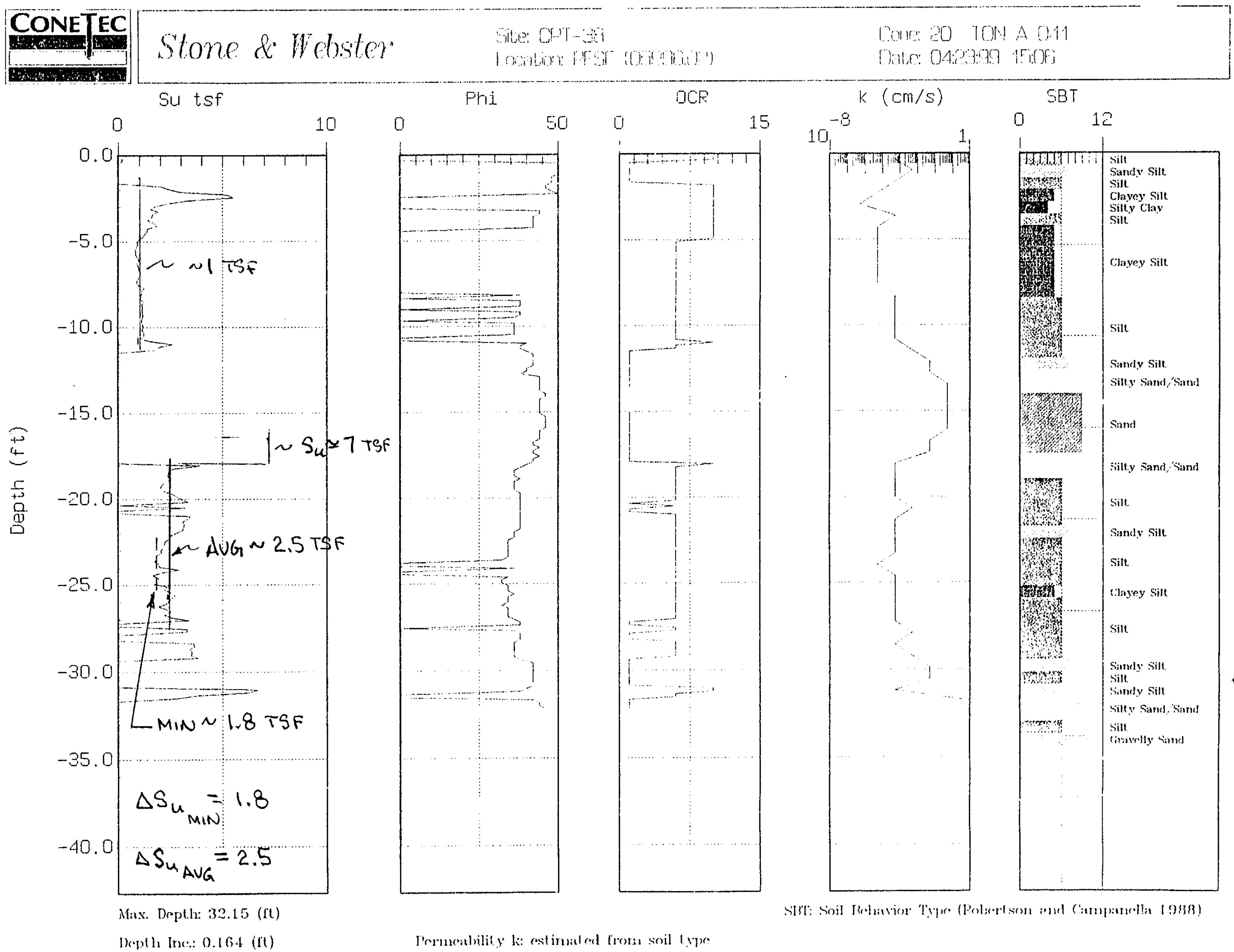
Max. Depth: 30.84 (ft)

Depth Inc.: 0.164 (ft)

Permeability k: estimated from soil type

SBT: Soil Behavior Type (Robertson and Campanella 1988)

App. D-37



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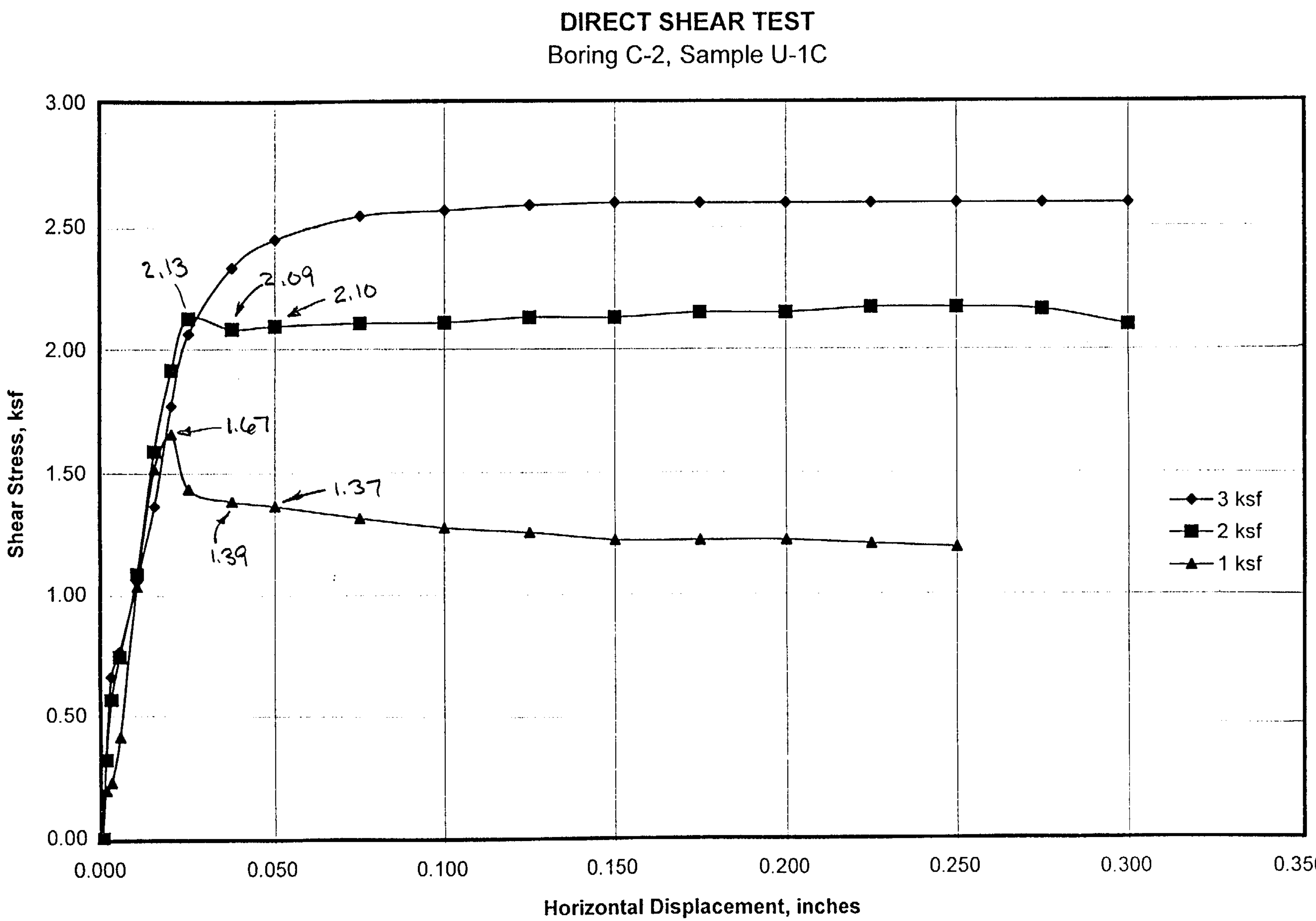
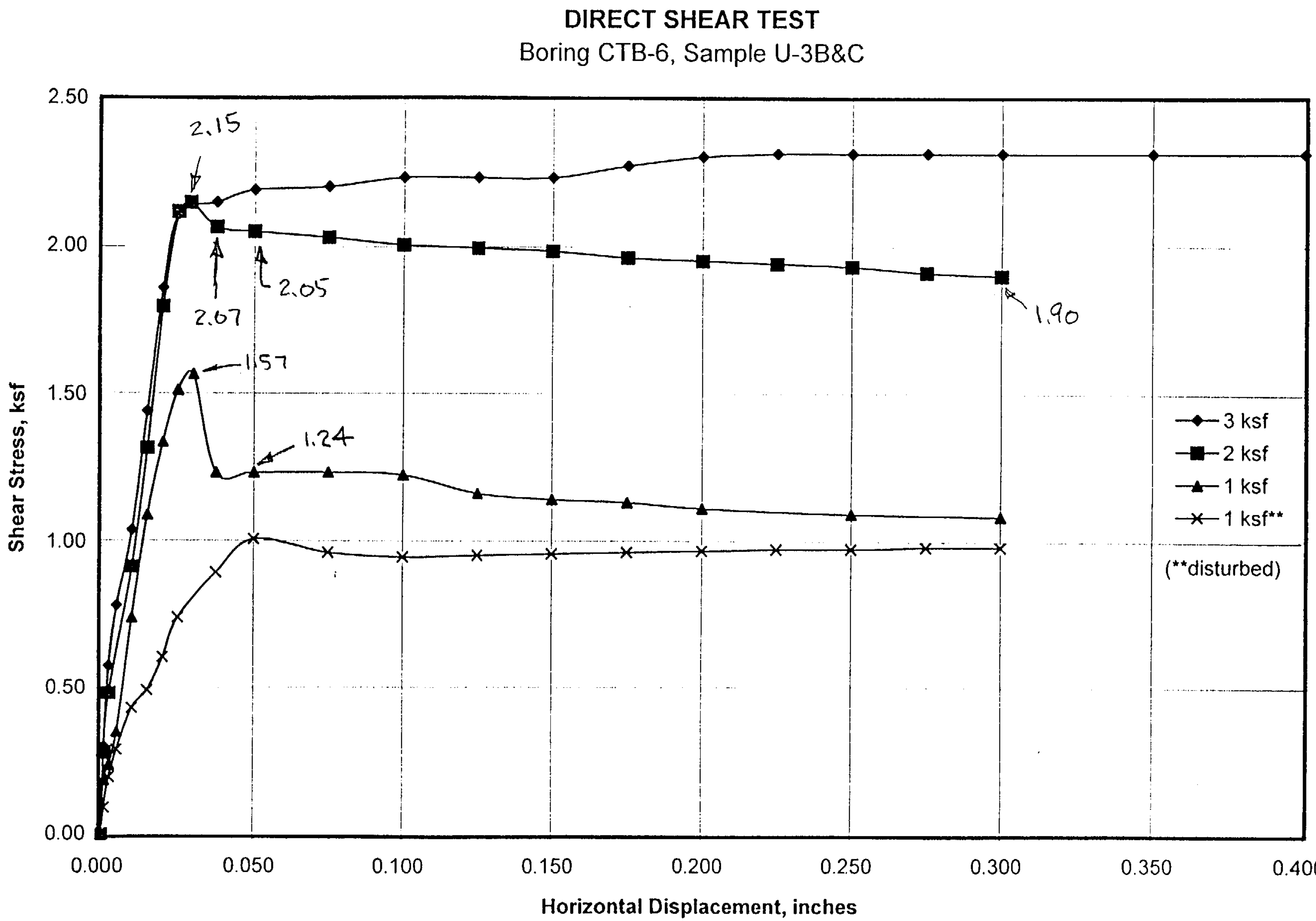
ATTACHMENT B P B2
CALC 05996.02-G(B)-13-6

App. D-38

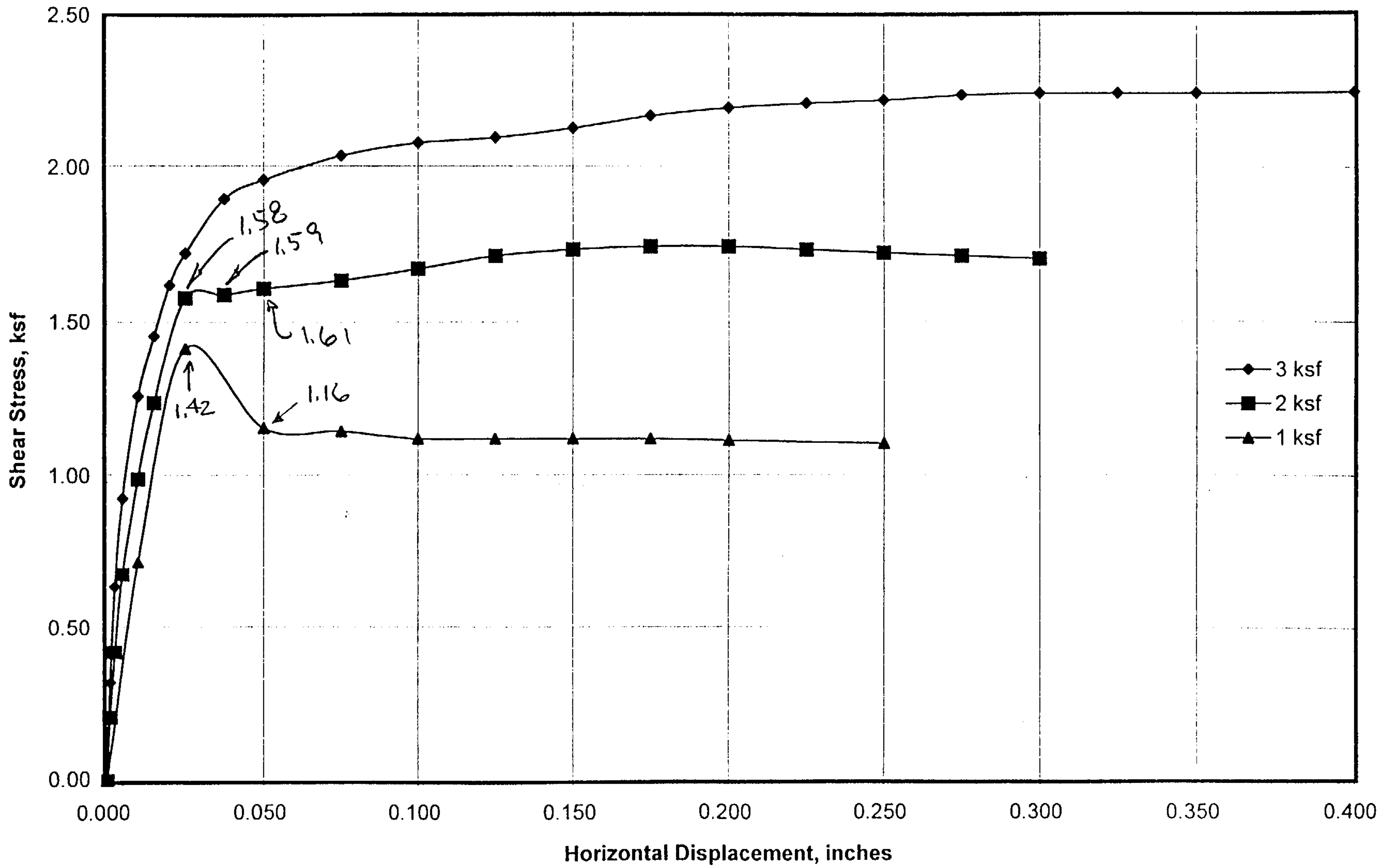


ATTACHMENT C TO CALC 05996.02-G(B)-13-6 p C1/3



ATTACHMENT C TO CALC 05996.02-G(B)-13-6 p C2

DIRECT SHEAR TEST
Boring CTB-S, Sample U-1AA



ATTACHMENT C TO CALC 05996.02-G(B)-13-6 p C3