

RAS 5223

72-22-ISFSI - Applicant Exhibit UU- Rec'd 5/1/02

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 (✓) <input checked="" type="checkbox"/> I NUCLEAR SAFETY RELATED <input type="checkbox"/> II <input type="checkbox"/> III <input type="checkbox"/> (other)	
CALCULATION IDENTIFICATION NUMBER					
JOB ORDER NO.	DISCIPLINE	CURRENT CALC NO	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.	
05996.02	G(B)	04			
APPROVALS - SIGNATURE & DATE			REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED <input checked="" type="checkbox"/> YES <input type="checkbox"/> NO
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)			
Original Signed By: TESponseller / 2-18-97 PJTrudeau / 2-24-97	Original Signed By: PJTrudeau / 2-24-97 TESponseller / 2-24-97	Original Signed By: NTGeorges / 2-27-97	0		✓
Original Signed By: TESponseller / 4-30-97 PJTrudeau / 4-30-97	Original Signed By: PJTrudeau / 4-30-97 TESponseller / 4-30-97	Original Signed By: AFBrown / 5-8-97	1	0	✓
Original Signed By: PJTrudeau / 6-20-97	Original Signed By: NTGeorges / 6-20-97	Original Signed By: AFBrown / 6-20-97	2	1	✓
Original Signed By: PJTrudeau / 6-27-97	Original Signed By: LPSingh / 7-1-97	Original Signed By: LPSingh / 7-1-97	3	2	✓
Original Signed By: DLAloysius / 9-3-99 SYBoakye / 9-3-99	Original Signed By: SYBoakye / 9-3-99 DLAloysius / 9-3-99	Original Signed By: TYChang / 9-3-99	4	3	✓
Original Signed By: PJTrudeau / 1-26-00	Original Signed By: TYC for SYBoakye 1-26-00 Lliu / 1-26-00	Original Signed By: TYChang / 1-26-00	5	4	✓
Original Signed By: PJTrudeau / 6-16-00	Original Signed By: TYChang / 6-16-00	Original Signed By: TYChang / 6-16-00	6	5	✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	7	6	✓
DISTRIBUTION					
GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)
RECORDS MGT. FILES (OR FIRE FILE IF NONE) Geotechnical	JOB BOOK R4.2G FIRE FILE - Denver PJTrudeau - Stoughton/3	ORIG <input checked="" type="checkbox"/> <input checked="" type="checkbox"/>		SECRETED USNRC 2003 JAN 13 AM 10:09 OFFICE OF THE SECRETARY RULEMAKINGS AND ADJUDICATIONS STAFF	

Template = SECY-028

SECY-02

CLEAR REGULATORY COMMISSION

Docket No. _____ Official Exh. No. 44
In the matter of PPS
Staff _____ IDENTIFIED ✓
Applicant ✓ RECEIVED ✓
Intervenor _____ REJECTED _____
Other _____ WITHDRAWN _____
DATE 5-1-02 Witness _____
Clerk pmp

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.84

CLIENT & PROJECT PRIVATE FUEL STORAGE, LLC - PFSF				PAGE 2		
CALCULATION TITLE STABILITY ANALYSES OF CASK STORAGE PADS				QA CATEGORY (✓) <input checked="" type="checkbox"/> I NUCLEAR SAFETY RELATED <input type="checkbox"/> II <input type="checkbox"/> III <input type="checkbox"/> (other)		
CALCULATION IDENTIFICATION NUMBER						
JOB ORDER NO.	DISCIPLINE	CURRENT CALC NO	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.		
05996.02	G(B)	04				
APPROVALS - SIGNATURE & DATE				REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED <input checked="" type="checkbox"/>
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
Original Signed By: PJTrudeau / 5-31-01	Original Signed By: TYChang / 5-31-01	Original Signed By: TYChang / 5-31-01	8	7		✓
PJTrudeau / 7-26-01 <i>PJ Trudeau</i>	TYChang / 7-26-01 <i>Thomas Y. Chang</i>	TYChang / 7-26-01 <i>Thomas Y. Chang</i>	9	8		✓
DISTRIBUTION						
GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)	
RECORDS MGT. FILES (OR FIRE FILE IF NONE)	JOB BOOK R4.2G	ORIG				
Geotechnical	FIRE FILE - Denver	<input checked="" type="checkbox"/>				
	PJTrudeau - Stoughton/3	<input checked="" type="checkbox"/>				

CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 3
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		
TABLE OF CONTENTS				
TABLE OF CONTENTS				3
RECORD OF REVISIONS				5
OBJECTIVE OF CALCULATION				9
ASSUMPTIONS/DATA				9
GEOTECHNICAL PROPERTIES				10
METHOD OF ANALYSIS				12
DESCRIPTION OF LOAD CASES				12
OVERTURNING STABILITY OF THE CASK STORAGE PADS				13
SLIDING STABILITY OF THE CASK STORAGE PADS				15
<i>Design Issues Related to Sliding Stability of the Cask Storage Pads</i>				15
Sliding Stability at Interface Between In Situ Clayey Soils and Bottom of Soil				
Cement Beneath the Pads				18
Active Earth Pressure				19
Dynamic Earth Pressure				19
Weights				21
Earthquake Accelerations - PSHA 2,000-Yr Return Period				21
Cask Earthquake Loadings				21
Foundation Pad Earthquake Loadings				21
Soil Cement Beneath Pad Earthquake Loadings				21
Case III: 0% N-S, -100% Vertical, 100% E-W (Earthquake Forces Act Upward)				22
Case IV: 0% N-S, 100% Vertical, 100% E-W (Earthquake Forces Act				
Downward)				22
Minimum Shear Strength Required at the Base of the Pads to Provide a Factor				
of Safety of 1.1				23
Adhesion between the Base of Pad and Underlying Clayey Soils				24
Limitation of Strength of Soil Cement Beneath the Pads				27
Sliding Along Contact Between the Concrete Pad and the Underlying Soil				
Cement				28
Soil Cement Above the Base of the Pads				29
Sliding Resistance of Entire N-S Column of Pads				31
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				33
Sliding Resistance of Last Pad in Column of Pads ("Edge Effects")				35
Width of Soil Cement Adjacent to Last Pad to Provide Full Passive Resistance				35
Sliding Stability of the Pads Assuming Resistance is Based on Only Frictional				
Resistance Along Base Plus Passive Resistance				36
Summary of Horizontal Displacements Calculated Based on Newmark's Method				
for assumption that Cask Storage Pads Are Founded Directly on				
Cohesionless Soils with $\phi = 17^\circ$ and Passive Pressure Due to Site Soils Acts				
on 5-ft Thick Layer of Soil Cement at End of Row of 20 Pads				45
Evaluation of Sliding on Deep Slip Surface Beneath Pads				46
<i>Estimation of Horizontal Displacement using Newmark's Method</i>				46
Maximum Ground Motions				47
Load Cases				48

STONE & WEBSTER, INC.
CALCULATION SHEET

5810.65

CALCULATION IDENTIFICATION NUMBER				PAGE 4
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		
Ground Motions for Analysis				48
Summary of Horizontal Displacements Calculated Based on Newmark's Method for assumption that Cask Storage Pads Are Founded Directly on Cohesionless Soils with $\phi = 30^\circ$ and No Soil Cement				50
ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS				52
Bearing Capacity Factors				52
Shape Factors (for $L > B$)				53
Depth Factors (for $\frac{D_f}{B} \leq 1$)				53
Inclination Factors				53
Static Bearing Capacity of the Cask Storage Pads				53
Dynamic Bearing Capacity of the Cask Storage Pads				57
Based on Inertial Forces				57
Based on Maximum Cask Dynamic Forces from the SSI Analysis				73
CONCLUSIONS				99
OVERTURNING STABILITY OF THE CASK STORAGE PADS				99
SLIDING STABILITY OF THE CASK STORAGE PADS				99
ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS				101
Static Bearing Capacity of the Cask Storage Pads				101
Dynamic Bearing Capacity of the Cask Storage Pads				101
REFERENCES:				103
TABLES				105
FIGURES				109
ATTACHMENT A	Telcon 6-19-97 SMM to PJT Dynamic Bearing Capacity of Pad			1 page
ATTACHMENT B	Pages from Calc 05996.02-G(PO17)-2, Rev. 3 providing maximum cask dynamic loads.			14 pages
ATTACHMENT C	Pages from Calc 05996.02-G(B)-05-2 providing basis for undrained strength used for dynamic bearing capacity analyses.			3 pages
ATTACHMENT D	Annotated Copies of Direct Shear Test Plots of Horizontal Displacement vs Shear Stress			3 pages
ATTACHMENT E	Triaxial Test Plot from Attachment 8 of Appendix 2A of SAR			1 page

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 5
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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; $\phi = 24.9^\circ$, 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}

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 6
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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.

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 7
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		
<p>REVISION 6</p> <ol style="list-style-type: none"> 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.. <p>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.</p>				
<p>REVISION 7</p> <ol style="list-style-type: none"> 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. 				

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 8
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

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

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 $B/L \sim 0$ & $B=30$ 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 ($N \geq 100$ 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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 10
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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),

$\gamma_{\text{moist}} = 80$ 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 $B' \sim 15$ 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.

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 11
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 $\phi = 30^\circ$. 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).

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 12
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

Case IA Static using undrained strength: $\phi = 0^\circ$ & $c = 2.2$ ksf.

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

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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 13
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04 - 9	OPTIONAL TASK CODE	

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:

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

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/ft³ = 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,

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

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 ($\mu = 0.8$, 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.

$$\Sigma M_{Driving} = a_h \quad W_p \quad EQ_{hc} \\ = 1.5 \text{ ft} \times 0.711 \times 904.5 \text{ K} + 3 \text{ ft} \times 696 \text{ K} = 3,053 \text{ ft-K}$$

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

CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 14
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 15
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

SLIDING STABILITY OF THE CASK STORAGE PADS

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

$$FS = \text{resisting force} + \text{driving force}$$

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

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

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

$$\phi = 0^\circ \text{ (for Silty Clay/Clayey Silt)}$$

$$c = 2.1 \text{ ksf, as indicated on p C-2.}$$

$$B = 30 \text{ feet}$$

$$L = 67 \text{ feet}$$

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 HTS) 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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 16
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		
<p>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.</p> <p>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 HTS.</p> <p>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 HTS) will be at least 40 psi.</p> <p>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.</p> <p>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.</p> <p>11. The pads are being surrounded with soil cement so that PFS can effectively use the colian 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.</p> <p>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.</p> <p>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).</p>				

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 17
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		
<p>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.</p>				

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 18
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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:

$\gamma = 100-110$ 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.

$\phi = 40^\circ$ 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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 19
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

ACTIVE EARTH PRESSURE

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

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

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

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

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:

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

where :

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

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:

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

γ = unit weight of soil,

H = wall height, and

K_{AE} is calculated as shown above.

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 20
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

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,

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

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

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

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

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

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:

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

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 21
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

WEIGHTS

Casks: $W_c = 8 \times 356.5 \text{ K/cask} = 2,852 \text{ K}$

Pad: $W_p = 3 \text{ ft} \times 67 \text{ ft} \times 30 \text{ ft} \times 0.15 \text{ kips/ft}^3 = 904.5 \text{ K}$

Soil Cement Beneath Pad: $W_{sc} = 2 \text{ ft} \times 67 \text{ ft} \times 30 \text{ ft} \times 0.10 \text{ kips/ft}^3 = 402 \text{ K}$

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

$EQ_{vc} = -0.695 \times 2,852 \text{ K} = -1,982 \text{ K}$ (minus sign signifies uplift force)

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

$EQ_{hcN-S} = 2,102 \text{ K}$ (acting in long direction of pad, N-S) $Q_{yd \text{ max}}$ 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. $EQ_{hc \text{ max}}$ is limited to a maximum value of 696 K for Case III, based on the upper-bound value of $\mu = 0.8$, as shown in the following table:

Cask Loads	WT K	EQ_{vc} K	N K	$0.2 \times N$ K	$0.8 \times N$ K	$EQ_{hc \text{ max}}$ 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
$EQ_{vp} = -0.695 \times 904.5 \text{ K} = -629 \text{ K}$	$EQ_{vsc} = -0.695 \times 402 \text{ K} = -279.4 \text{ K}$
$EQ_{hp} = 0.711 \times 904.5 \text{ K} = 643 \text{ K}$	$EQ_{hp} = 0.711 \times 402 \text{ K} = 285.8 \text{ K}$

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 22
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

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:

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

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

The driving force, V, is defined as:

$$V = F_{AE} + EQ_{hp} + E_{qhc} + EQ_{hsc}$$

The factor of safety against sliding is calculated as follows:

$$FS = \begin{matrix} T \\ 4,221 \text{ K} \end{matrix} + \begin{matrix} F_{AEWS} \\ (181.5 \text{ K}) \end{matrix} + \begin{matrix} EQ_{hp} \\ 643 \text{ K} \end{matrix} + \begin{matrix} E_{qhc} \\ 696 \text{ K} \end{matrix} + \begin{matrix} EQ_{hsc} \\ 285.8 \text{ K} \end{matrix} = 2.34$$

(1,806.3 K)

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, $\mu = 0.8$, 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) = $\sum F_v = Wc + Wp + EQvc + Eqvp + EQvsc$

$$N = \begin{matrix} Wc \\ 2,852 \text{ K} \end{matrix} + \begin{matrix} Wp \\ 904.5 \text{ K} \end{matrix} + \begin{matrix} EQvc \\ 1,982 \text{ K} \end{matrix} + \begin{matrix} EQvp \\ 629 \text{ K} \end{matrix} + \begin{matrix} EQvsc \\ 279.4 \text{ K} \end{matrix} = 6,647 \text{ K}$$

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

The driving force, V, is defined as:

$$V = F_{AE} + EQ_{hp} + E_{qhc} + EQ_{hsc}$$

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 23
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

The factor of safety against sliding is calculated as follows:

$$FS_{\text{Soil Cement to Clayey Soil}} = \frac{T}{F_{AE-E-W}} + \frac{EQ_{hp}}{EQ_{hCE-W}} + \frac{EQ_{hsc}}{EQ_{hCE-W}} = \frac{4,221 \text{ K} + (181.5 \text{ K} + 643 \text{ K} + 2,212 \text{ K} + 285.8 \text{ K})}{(3,322.3 \text{ K})} = \underline{1.27 (=Min)}$$

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 EQ_{hc} (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:

$$FS = \frac{T}{F_{AE-E-W}} + \frac{EQ_{hp}}{EQ_{hCE-W}} + \frac{EQ_{hsc}}{EQ_{hCE-W}} \geq 1.1$$

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

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

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

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 24
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

ADHESION BETWEEN THE BASE OF PAD AND UNDERLYING CLAYEY SOILS

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

CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 25
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

ADHESION BETWEEN THE BASE OF PAD AND UNDERLYING CLAYEY SOILS

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

CALCULATION SHEET

8010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 26
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

ADHESION BETWEEN THE BASE OF PAD AND UNDERLYING CLAYEY SOILS

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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 27
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 Profiles, 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.

STONE & WEBSTER, INC.
CALCULATION SHEET

8010.55

CALCULATION IDENTIFICATION NUMBER				PAGE 28
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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:

$$T = \frac{N}{\phi} + \frac{c}{\tan \theta} = 6,368 \text{ K} \times \tan 0^\circ + 2.88 \text{ ksf} \times 30 \text{ ft} \times 67 \text{ ft} = 5,789 \text{ K}$$

As indicated above, the driving force, V, is defined as: $V = F_{AE} + EQ_{hp} + EQ_{hc}$

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:

$$FS_{\text{Pad to Soil Cement}} = \frac{T}{F_{AE} + EQ_{hp} + EQ_{hc}} = \frac{5,789 \text{ K}}{(65.3 \text{ K} + 643 \text{ K} + 2,212 \text{ K})} = 1.98$$

(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 colian 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 $\sim 40/40 = 1\%$. 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.

STONE & WEBSTER, INC.
CALCULATION SHEET

8010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 29
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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, $\sigma_v \sim 2$ ksf, at horizontal displacements of ~ 0.025 " past the peak strength, there is $\sim 1.5\%$ 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 $\sim 5\%$.

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 30
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 $0.95 \times 2.1 \text{ ksf} \times 30 \text{ ft} \times 67 \text{ ft} = 4,010 \text{ K}$.

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:

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

$$\begin{array}{cc} \text{Clay} & \text{Soil Cement} \\ T_{N-S} = 4,010 \text{ K} + 2,516 \text{ K} = 6,526 \text{ K} \end{array}$$

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

$$\text{FS Pad to Clayey Soil N-S w/Passive} = \frac{T_{N-S}}{F_{AE N-S}} = \frac{6,526 \text{ K}}{(29.3 \text{ K} + 643 \text{ K} + 2,102 \text{ K})} = \frac{6,526 \text{ K}}{2,774.3 \text{ K}} = 2.35$$

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:

$$\text{FS Pad to Clayey Soil N-S w/o Passive} = \frac{T_{N-S}}{F_{AE N-S}} = \frac{4,221 \text{ K}}{(29.3 \text{ K} + 643 \text{ K} + 2,102 \text{ K})} = \frac{4,221 \text{ K}}{2,774.3 \text{ K}} = 1.52$$

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:

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

$$\begin{array}{cc} \text{Clay} & \text{Soil Cement} \\ T_{E-W} = 4,010 \text{ K} + 5,620 \text{ K} = 9,630 \text{ K} \end{array}$$

$$\text{FS Pad to Clayey Soil E-W} = \frac{T_{E-W}}{F_{AE E-W}} = \frac{9,630 \text{ K}}{(65.3 \text{ K} + 643 \text{ K} + 2,212 \text{ K})} = \frac{9,630 \text{ K}}{2,920.3 \text{ K}} = 3.30$$

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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 31
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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:

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

Inertial forces due to Pads + Soil Cement:

$$\text{Weight of Pads} = 10 \times 904.5 \text{ K} = 9,045 \text{ K}$$

$$\text{Weight of Soil Cement} = 9 \times 3.33 \text{ ft} \times 30 \text{ ft} \times 5 \text{ ft} \times 0.11 \text{ kips/ft}^3 = 495 \text{ K}$$

$$+ 10 \times 30 \text{ ft} \times 67 \text{ ft} \times 1 \text{ ft} \times 0.11 \text{ kips/ft}^3 = 2,211 \text{ K}$$

$$\text{Total Weight} = 11,751 \text{ K}$$

$$\text{Inertial forces due to Pads + Soil Cement} = 0.711 \times 11,751 \text{ K} = 8,355 \text{ K}$$

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

$$\text{Total driving force in N-S direction} = 21,020 \text{ K} + 8,355 \text{ K} + 81.3 \text{ K} = 29,456 \text{ K}$$

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:

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 32
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

Soil cement

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

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

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

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

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

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:

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

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

Soil cement

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

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

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 33
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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.

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

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

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

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

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 \times 2,212 K = 22,120 K vs 10 pads \times 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) \times 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 =

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

$$= 1.1 \times [21,020 \text{ K} + (11,751 \text{ K} \times 0.711) + 81.3 \text{ K}]$$

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

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.

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 34
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
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

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

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

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

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

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

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

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:

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

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,

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

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

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

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

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

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:

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.55

CALCULATION IDENTIFICATION NUMBER				PAGE 35
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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:

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

The base area required to provide this shear resistance = 30 ft x L_{N-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.

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

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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 36
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 $N \tan \phi$. $N = 1,146 \text{ K}$, as shown on p. 21:

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 37
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

$$N = \begin{matrix} W_c \\ 2,852 \text{ K} \end{matrix} + \begin{matrix} W_p \\ 904.5 \text{ K} \end{matrix} + \begin{matrix} EQ_{vc} \\ (-1,982 \text{ K}) \end{matrix} + \begin{matrix} EQ_{vp} \\ (-629 \text{ K}) \end{matrix} = 1,146 \text{ K}$$

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

The driving force, V, is defined as: $V = F_{AE} + EQ_{hp} + EQ_{hc}$

The factor of safety against sliding is calculated as follows:

$$FS = \begin{matrix} T \\ 350.4 \text{ K} \end{matrix} + \begin{matrix} F_{AE-N-S} \\ (29.3 \text{ K}) \end{matrix} + \begin{matrix} EQ_{hp} \\ 643 \text{ K} \end{matrix} + \begin{matrix} EQ_{hc} \\ 696 \text{ K} \end{matrix} = 0.26$$

(1,368.3 K)

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)

$$N = \begin{matrix} W_c \\ 2,852 \text{ K} \end{matrix} + \begin{matrix} W_p \\ 904.5 \text{ K} \end{matrix} + \begin{matrix} EQ_{vc} \\ (-1,982 \text{ K}) \end{matrix} + \begin{matrix} EQ_{vp} \\ (-629 \text{ K}) \end{matrix} = 1,146 \text{ K}$$

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

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

$$FS = \begin{matrix} T \\ 350.4 \text{ K} \end{matrix} + \begin{matrix} 40\% \text{ of } [F_{AE-N-S} \\ (0.4 \times (29.3 \text{ K} + 643 \text{ K})) \end{matrix} + \begin{matrix} EQ_{hp} \\ 696 \text{ K} \end{matrix} = 0.36$$

(964.9 K)

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

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

$0.8 \times N_{Casks} = 696 \text{ K}$, as shown in the table on p. 20; therefore, EQ_{hc-N-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.

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 38
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 $1.1 \times 965 \text{ K} - 350 \text{ K}$, 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 $2.33' \times 30'$. 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 $5' \times 0.110 \text{ kcf} = 0.55 \text{ ksf}$. Subtracting the uplift forces, the normal stress is reduced to $(1 - 0.695) \times 0.55 \text{ ksf} = 0.168 \text{ ksf}$. The shear resistance available due to friction at the base of the soil cement overlying the clayey soils is calculated as $N \tan \phi$, or $0.168 \text{ ksf} \times \tan 17^\circ = 0.051 \text{ ksf}$.

STONE & WEBSTER, INC.
CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 39
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 \times the peak horizontal acceleration, $0.711g$, which equals $0.711g \times 5' \times 0.110 \text{ kcf/g} \times$ 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 $0.4 \times 0.39 \text{ ksf} = 0.156 \text{ ksf}$.

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

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

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.,

$$FS_{\text{Soil-cement Block Case IIIC}} = \frac{(1 - 0.4 \times 0.695) \times 5 \text{ ft} \times 0.11 \text{ kcf} \times \tan 17^\circ}{100\% \times 0.711 \times 5 \text{ ft} \times 0.11 \text{ kcf}} = \frac{0.121 \text{ ksf}}{0.391 \text{ ksf}} = 0.31$$

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.,

$$FS_{\text{Soil-cement Block Case IIIA}} = \frac{w / \phi = 40^\circ}{40\% \times 0.711 \times 5 \text{ ft} \times 0.11 \text{ kcf}} = \frac{0.141 \text{ ksf}}{0.156 \text{ ksf}} = 0.90$$

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.55

CALCULATION IDENTIFICATION NUMBER				PAGE 40
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

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 $\phi = 21.3^\circ$.

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

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

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

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 = $30 \text{ ft} \times 20.91 \text{ K/LF} = 627.3 \text{ K}$.

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:

$$\text{FS} = \frac{T}{(977.7 \text{ K})} = \frac{P_p}{(627.3 \text{ K})} + \frac{40\% \text{ of } [F_{AE} N - S \quad E_{Qhp} \quad E_{qhc}]}{(964.9 \text{ K})} = 1.01$$

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.**

STONE & WEBSTER, INC.
CALCULATION SHEET

8010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 41
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

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 $N \tan \phi$. $N = 1,146$ K, calculated as follows:

$$N = \begin{matrix} W_c \\ 2,852 \text{ K} \end{matrix} + \begin{matrix} W_p \\ 904.5 \text{ K} \end{matrix} + \begin{matrix} EQ_{vc} \\ (-1,982 \text{ K}) \end{matrix} + \begin{matrix} EQ_{vp} \\ (-629 \text{ K}) \end{matrix} = 1,146 \text{ K}$$

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

Therefore, the total resistance due to friction acting on the base of 20 pads in the row is $20 \times 350.4 \text{ K} = 7,008 \text{ K}$. 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 $20.91 \text{ K/LF} \times (30' + 35')$, which equals $1,359 \text{ K}$.

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*.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.55

CALCULATION IDENTIFICATION NUMBER				PAGE 42
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

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 $V = F_{AE} + EQ_{hp} + EQ_{hc}$. For the entire row of 20 pads, the maximum horizontal driving force is calculated as:

$$V = 29.3 \text{ K} + 20 \text{ pads} \times [643 \text{ K} + 696 \text{ K}] = 26,809 \text{ K}.$$

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-S} is:

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

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

$$FS = \frac{T}{40\% \text{ of } F_{AE \text{ N-S}} + EQ_{hp} + EQ_{hc}} = \frac{7,635.3 \text{ K}}{19,076 \text{ K}} = 0.40$$

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:

$$FS = \frac{T}{40\% \text{ of } F_{AE \text{ N-S}} + EQ_{hp} + EQ_{hc}} = \frac{8,367 \text{ K}}{19,076 \text{ K}} = 0.44$$

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 $\phi = 17^\circ$. 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."

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 43
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

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

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

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

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

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

$$30 \text{ ft} \times 2 \text{ ft} \times \left[20 \text{ pads} \times 67 \frac{\text{ft}}{\text{pad}} + 9 \frac{\text{gaps}}{\text{area}} \times 5 \text{ ft} \frac{\text{length}}{\text{gap}} \times 2 \text{ areas} + 90 \text{ ft between areas} \right] \times 0.100 \text{ kcf} = 9,120 \text{ K}$$

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

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

$$\phi = 17^\circ$$

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

$$N = \frac{F_v - F_{v \text{ Eqk}}}{W} \tan \phi + \frac{P_p}{W} = \frac{[(86,032 - 59,792) \tan 17^\circ + 627.3 \text{ K}]}{86,032} = 0.101$$

Acceleration in N-S direction, $A = 0.284g$

Velocity in N-S direction, $V = 13.7 \text{ in./sec}$

$$\Rightarrow N / A = 0.101 / 0.284 = 0.354$$

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

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

where g is in units of inches/sec².

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

STONE & WEBSTER, INC.
CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 44
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

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

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

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

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, $F_v = W = 86,032 \text{ K}$

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

$$\phi = 17^\circ$$

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

Acceleration in N-S direction, $A = 0.711g$

Velocity in N-S direction, $V = 34.1 \text{ in./sec}$

$$\Rightarrow N/A = 0.228 / 0.711 = 0.321$$

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

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

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 45
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

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

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

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

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 $\phi = 17^\circ$ 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 $\phi = 17^\circ$, 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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 46
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

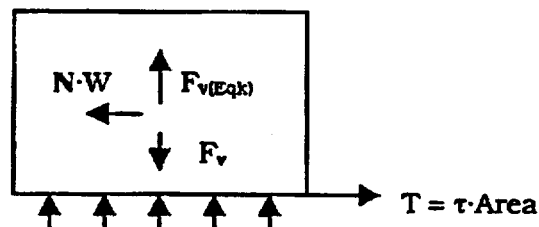
EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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



STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 47
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04 - 9	OPTIONAL TASK CODE	

EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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

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

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

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

where

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

σ_n = Normal Stress

ϕ = Friction angle of cohesionless layer

σ_n = Net Vertical Force/Area

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

$$T = (F_v - F_{v \text{ Eqs}}) \tan \phi$$

$$N W = T$$

$$\Rightarrow N = [(F_v - F_{v \text{ Eqs}}) \tan \phi] / W$$

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

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

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

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

MAXIMUM GROUND MOTIONS

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 48
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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

STONE & WEBSTER, INC.
CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 49
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04 - 9	OPTIONAL TASK CODE	

EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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

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

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

$$\phi = 30^\circ$$

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

$$N = [(3,757 - 2,611) \tan 30^\circ] / 3,757 = 0.176$$

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

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

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

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

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

where g is in units of inches/sec².

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

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

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

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

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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 50
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04 - 9	OPTIONAL TASK CODE	

EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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

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

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

$$\phi = 30^\circ$$

$$N = \frac{F_v - F_{v(Eqk)} \tan \phi}{F_v} = \frac{3,757 - 1,044 \tan 30^\circ}{3,757} = 0.417$$

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

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

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

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

$$u_m = \frac{V^2 (1 - N/A)}{2gN}$$

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

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 CASE STORAGE PADS ARE FOUNDED DIRECTLY ON COHESIONLESS
SOILS WITH $\phi = 30^\circ$ 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

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 51
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with $\phi = 30^\circ$, 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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 52
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 $q_{ult} = c \cdot N_c + q \cdot N_q + \frac{1}{2} \gamma B \cdot N_r$. 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_r . 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:

$$q_{ult} = c N_c s_c d_c i_c + q N_q s_q d_q i_q + \frac{1}{2} \gamma B N_r s_r d_r i_r$$

where

q_{ult} = ultimate bearing capacity

c = cohesion or undrained strength

q = effective surcharge at bottom of foundation, $= \gamma D_f$

γ = unit weight of soil

B = foundation width

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

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

i_c, i_q, i_r = load inclination factors

N_c, N_q, N_r = 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:

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

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

$$N_r = 2 (N_q + 1) \tan \phi$$

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 53
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS

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 $\frac{D_f}{B} \leq 1$)

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

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

$$d_\gamma = 1$$

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 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 ($\phi = 0^\circ$ & $c = 2.2$ ksf).

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 54
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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)

$FS = 3.0$ Factor of Safety required for $q_{allowable}$

$0 g = a_H$

$0 g = a_v$

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

$EQ_{H \text{ E-W}} = 0 \text{ k}$ & $EQ_{H \text{ N-S}} = 0 \text{ k} \rightarrow 0 \text{ k}$ 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)

$$N_c = (N_q - 1) \cot(\phi), \text{ but } = 5.14 \text{ for } \phi = 0 \quad = 5.14 \quad \text{Eq 3.6 \& Table 3.2}$$

$$N_q = e^{\pi \tan \phi} \tan^2(\pi/4 + \phi/2) \quad = 1.00 \quad \text{Eq 3.6}$$

$$N_\gamma = 2 (N_q + 1) \tan(\phi) \quad = 0.00 \quad \text{Eq 3.8}$$

$$s_c = 1 + (B/L)(N_q/N_c) \quad = 1.09 \quad \text{Table 3.2}$$

$$s_q = 1 + (B/L) \tan \phi \quad = 1.00 \quad "$$

$$s_\gamma = 1 - 0.4 (B/L) \quad = 0.82 \quad "$$

$$\text{For } D_f/B \leq 1: d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 D_f/B \quad = 1.00 \quad \text{Eq 3.26}$$

$$d_\gamma = 1 \quad = 1.00 \quad "$$

$$\text{For } \phi > 0: d_c = d_q - (1 - d_q) / (N_q \tan \phi) \quad = \text{N/A}$$

$$\text{For } \phi = 0: d_c = 1 + 0.4 (D_f/B) \quad = 1.04 \quad \text{Eq 3.27}$$

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 =	$(F_v \text{ Static} + EQ_v) / (B' \times L')$				
$FS_{actual} =$	7.00	=	q_{ult} / q_{actual}				
							> 3 Hence OK

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 55
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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)
 $\gamma_{\text{surch}} = 100$ 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)

$FS = 3.0$ Factor of Safety required for $q_{\text{allowable}}$ $0 g = a_H$
 $F_{V \text{ Static}} = 3,757 \text{ k}$ & $EQ_V = 0 \text{ k} \rightarrow 3,757 \text{ k for } F_V$ $0 g = a_V$
 $EQ_{H \text{ E-W}} = 0 \text{ k}$ & $EQ_{H \text{ N-S}} = 0 \text{ k} \rightarrow 0 \text{ k for } F_H$

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

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

$$N_c = (N_q - 1) \cot(\phi), \text{ but } = 5.14 \text{ for } \phi = 0 = 30.14 \text{ Eq 3.6 \& Table 3.2}$$

$$N_q = e^{\pi \tan \phi} \tan^2(\pi/4 + \phi/2) = 18.40 \text{ Eq 3.6}$$

$$N_\gamma = 2 (N_q + 1) \tan(\phi) = 22.40 \text{ Eq 3.8}$$

$$s_c = 1 + (B/L)(N_q/N_c) = 1.27 \text{ Table 3.2}$$

$$s_q = 1 + (B/L) \tan \phi = 1.26 \text{ "}$$

$$s_\gamma = 1 - 0.4 (B/L) = 0.82 \text{ "}$$

$$\text{For } D_f/B \leq 1: d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 D_f/B = 1.03 \text{ Eq 3.26}$$

$$d_\gamma = 1 = 1.00 \text{ "}$$

$$\text{For } \phi > 0: d_c = d_q - (1 - d_q) / (N_q \tan \phi) = 1.03$$

$$\text{For } \phi = 0: d_c = 1 + 0.4 (D_f/B) = \text{N/A} \text{ Eq 3.27}$$

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

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5810.65

CALCULATION IDENTIFICATION NUMBER				PAGE 56
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04 - 9	OPTIONAL TASK CODE	

STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

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 $\phi = 0^\circ$ 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 $\phi = 30^\circ$ 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.

CALCULATION SHEET

6010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 57
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

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 \pm the vertical inertial forces due to the earthquake. The vertical inertial force is calculated as $a_v \times$ [weight of the pad + cask dead loads], multiplied by the appropriate factor ($\pm 40\%$ or $\pm 100\%$) 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 $a_h \times$ [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 ($\mu = 0.8$, as shown in SAR Section 8.2.1.2, Accident Analysis) \times 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 $\mu = 0.2$.

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 ($\phi = 0^\circ$ 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

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 58
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 II: 100% N-S, 0% Vertical, 100% E-W

Determine forces and moments due to earthquake.

$$F_v = \frac{W_c}{2} + \frac{W_p}{2} = 2,852 \text{ K} + 904.5 \text{ K} = 3,757 \text{ K} \text{ and } EQ_v = 0 \text{ for this case.}$$

$$EQ_{H \text{ Pad}} = 0.711 \times 3' \times 30' \times 67' \times 0.15 \text{ kcf} = 643 \text{ K}$$

$$EQ_{hc} = \text{Minimum of } \left[\frac{a_H}{W_c} \times 2,852 \text{ K} \text{ \& } \frac{\mu}{N_c} \times 2,852 \text{ K} \right] \Rightarrow EQ_{hc} = 2,028 \text{ K}$$

2,028 K 2,282 K

Note, $N_c = W_c$ in this case, since $a_v = 0$.

$$EQ_{H \text{ N-S}} = EQ_{hp} + EQ_{hc} = 643 \text{ K} + 2,028 \text{ K} = 2,671 \text{ K}$$

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

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

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb .

$$\Delta b = \frac{9.83' \times EQ_{hc}}{W_c + EQ_{vc}} = \frac{9.83' \times 2,028 \text{ K}}{2,852 \text{ K} + 0} = 6.99 \text{ ft}$$

$$\begin{aligned} \Sigma M_{\Theta \text{ N-S}} &= 1.5' \times 0.711 \times 904.5 \text{ K} + 3' \times 2,028 \text{ K} + 6.99' \times (2,852 \text{ K} + 0) \\ &= 965 \text{ ft-K} + 6,084 \text{ ft-K} + 19,935 \text{ ft-K} = 26,984 \text{ ft-K} \end{aligned}$$

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

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

See Table 2.6-7 for definition and calculation of B' and L' for these forces and moments.Determine $q_{allowable}$ for $FS = 1.1$.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 59
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				
Allowable Bearing Capacity of Cask Storage Pads			Based on Inertial Forces Combined: 100 % N-S, 0 % Vert, 100 % E-W	
PSHA 2,000-Yr Earthquake: Case II				
Soil Properties:	c =	2,200 Cohesion (psf)	Footing Dimensions:	
	φ =	0.0 Friction Angle (degrees)	B = 30.0	Width - ft (E-W)
	γ =	80 Unit weight of soil (pcf)	L = 67.0	Length - ft (N-S)
	γ _{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)		
	FS =	1.1 Factor of Safety required for q _{allowable}	0.711 g = a _H	0.695 g = a _V
	F _{V Static} =	3,757 k & EQ _V =	0 k →	3,757 k for F _V
	EQ _{H-E-W} =	2,671 k & EQ _{H-N-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(φ), but = 5.14 for φ = 0	= 5.14	Eq 3.6 & Table 3.2	
	N _q = e ^{π tan φ} tan ² (π/4 + φ/2)	= 1.00	Eq 3.6	
	N _γ = 2 (N _q + 1) tan(φ)	= 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 φ	= 1.00	"	
	s _γ = 1 - 0.4 (B/L)	= 0.88	"	
	For D _f /B ≤ 1: d _q = 1 + 2 tan φ (1 - sin φ) ² D _f /B	= 1.00	Eq 3.26	
	d _γ = 1	= 1.00	"	
	For φ > 0: d _c = d _q - (1 - d _q) / (N _q tan φ)	= N/A		
	For φ = 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-S} > 0: θ _n = tan ⁻¹ (EQ _{H-E-W} / EQ _{H-N-S})	= 0.79 rad		
	m _n = m _L cos ² θ _n + m _B sin ² θ _n	= 1.50	Eq 3.18c	
	I _q = { 1 - F _H / [(F _V + EQ _V) + B' L' c cot φ] } ^m	= 1.00	Eq 3.14a	
	I _γ = { 1 - F _H / [(F _V + EQ _V) + B' L' c cot φ] } ^{m+1}	= 0.00	Eq 3.17a	
	For φ = 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' x L')	
	FS _{actual} =	1.17	= q _{ult} / q _{actual} > 1.1 Hence OK	

CALCULATION SHEET

8010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 60
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 IIIA: 40% N-S, -100% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$$EQ_v = -100\% \times 0.695 \times \overset{a_v}{0.695} \times \overset{W_p}{904.5 \text{ K}} + \overset{W_c}{2,852 \text{ K}} = -2,611 \text{ K}$$

$$EQ_{hp} = \overset{a_H}{0.711} \times \overset{W_c}{904.5 \text{ K}} = 643 \text{ K}$$

$$\begin{aligned} \text{Normal force at base of the cask} &= \text{Cask DL} = 2,852 \text{ K} \\ \text{— Cask } EQ_{vc} &= -1. \times 0.695 \times 2,852 \text{ K} = -1,982 \text{ K} = a_v \times W_c \\ &\Rightarrow N_c = 870 \text{ K} \end{aligned}$$

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

$$EQ_{hc} = \text{Minimum of } \left[\overset{a_H}{0.711} \times \overset{W_c}{2,852 \text{ K}} \text{ \& } \overset{\mu}{0.8} \times \overset{N_c}{870 \text{ K}} \right]$$

$$\begin{matrix} 2,028 \text{ K} & 696 \text{ K} \end{matrix}$$

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, EQ_{hc} 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 EQ_{hc} . For this case, the N-S and E-W components of EQ_{hc} are the same, and they are calculated as follows:

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

$$\begin{aligned} &\quad 40\% \text{ of } EQ_{hp} \quad EQ_{hc N-S} \\ \Rightarrow EQ_{HN-S} &= 0.4 \times 643 \text{ K} + 492.1 \text{ K} = 749.3 \text{ K} \end{aligned}$$

Since horizontal components are the same for this case, $EQ_{HE-W} = EQ_{HN-S}$

$$\Rightarrow F_H = \sqrt{EQ_{HE-W}^2 + EQ_{HN-S}^2} = \sqrt{749.3^2 + 749.3^2} = 1,060 \text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb . Note: $EQ_{vc} = -1. \times 0.695 \times 2,852 \text{ K} = -1,982 \text{ K}$

$$\Delta b_{E-W} = \frac{9.83' \times EQ_{hc}}{W_c + EQ_{vc}} = \frac{9.83' \times 492.1 \text{ K}}{2,852 \text{ K} - 1,982 \text{ K}} = 5.56 \text{ ft}$$

$$\begin{aligned} \Sigma M_{EN-S} &= \overset{40\% a_H}{1.5'} \times \overset{W_p}{0.4} \times \overset{W_p}{0.711} \times \overset{W_p}{904.5 \text{ K}} + \overset{E_{qhc E-W}}{3'} \times \overset{\Delta b}{492.1 \text{ K}} + \overset{W_c}{5.56'} \times \overset{EQ_{vc}}{(2,852 \text{ K} - 1,982 \text{ K})} \\ &= 386 \text{ ft-K} + 1,476 \text{ ft-K} + 4,837 \text{ ft-K} = 6,699 \text{ ft-K} \end{aligned}$$

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

$$\Sigma M_{EE-W} = \Sigma M_{EN-S} = 6,699 \text{ ft-K}$$

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 61
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				
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) $D_f =$ 3.0 Depth of Footing (ft)	$L' = 55.3$ Length - ft (N-S)		
			$0.711 \text{ g} = a_H$ $0.695 \text{ g} = a_V$	
			$FS =$ 1.1 Factor of Safety required for $q_{allowable}$	
			$F_{V \text{ Static}} =$ 3,757 k & $EQ_V =$ -2,611 k \rightarrow 1,146 k for F_V $EQ_{H \text{ E-W}} =$ 749 k & $EQ_{H \text{ N-S}} =$ 749 k \rightarrow 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 $D_f/B \leq 1$: $d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 D_f/B$			= 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 $\phi = 0$: $d_c = 1 + 0.4 (D_f/B)$			= 1.07	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 $EQ_{H \text{ N-S}} > 0$: $\theta_n = \tan^{-1}(EQ_{H \text{ E-W}} / EQ_{H \text{ N-S}})$			= 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
Gross $q_{ult} =$ 11,344 psf			= 11,044	+ 300 + 0
$q_{all} =$ 10,310 psf			= q_{ult} / FS	
$q_{actual} =$ 1,132 psf			= $(F_{V \text{ Static}} + EQ_V) / (B' \times L')$	
$FS_{actual} =$ 10.02			= q_{ult} / q_{actual} > 1.1 Hence OK	

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

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.

$$EQ_v = -40\% \times 0.695 \times (904.5 \text{ K} + 2,852 \text{ K}) = -1,044 \text{ K}$$

$$\text{Normal force at base of the cask} = \text{Cask DL} = 2,852 \text{ K}$$

$$\begin{aligned} -40\% \text{ of Cask } EQ_{vc} &= -0.4 \times 0.695 \times 2,852 \text{ K} = -793 \text{ K} = 40\% \text{ of } a_v \times W_c \\ \Rightarrow N_c &= 2,059 \text{ K} \end{aligned}$$

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

$$EQ_{hc} = \text{Min of } \left[\begin{array}{cc} a_H & W_c \\ 0.711 & 2,852 \text{ K} \end{array} \text{ \& } \begin{array}{cc} \mu & N_c \\ 0.8 & 2,059 \text{ K} \end{array} \right] \Rightarrow EQ_{hc} = 1,647 \text{ K};$$

2,028 K 1,647 K

i.e., EQ_{hc} 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 EQ_{hc} . For this case, the N-S component of $EQ_{hc} = 0.4 \times 2,028 \text{ K} = 811 \text{ K}$, and the E-W component is calculated as follows:

$$EQ_{hcE-W}^2 + EQ_{hcN-S}^2 = EQ_{hc}^2 = 1,647^2 \Rightarrow EQ_{hcE-W} = \sqrt{1,647^2 - 811^2} = 1,433.5 \text{ K}$$

$$\begin{aligned} \text{Using 40\% of N-S:} \quad & 40\% \text{ of } EQ_{hp} \quad EQ_{hcN-S} \\ \Rightarrow EQ_{HN-S} &= 0.4 \times 643 \text{ K} + 811 \text{ K} = 1,068 \text{ K} \end{aligned}$$

$$\begin{aligned} \text{Using 100\% of E-W:} \quad & 100\% \text{ of } EQ_{hp} \quad EQ_{hcE-W} \\ \Rightarrow EQ_{HE-W} &= 1.0 \times 643 \text{ K} + 1,433.5 \text{ K} = 2,076.5 \text{ K} \end{aligned}$$

$$\Rightarrow F_H = \sqrt{EQ_{HE-W}^2 + EQ_{HN-S}^2} = \sqrt{2,076.5^2 + 1,068^2} = 2,335 \text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb . Note: $EQ_{vc} = -0.4 \times 0.695 \times 2,852 \text{ K} = -793 \text{ K}$

$$\Delta b_{E-W} = \frac{9.83' \times EQ_{hcE-W}}{W_c + EQ_{vc}} = \frac{9.83' \times 1,433.5 \text{ K}}{2,852 \text{ K} - 793 \text{ K}} = 6.84 \text{ ft}$$

$$\begin{aligned} \Sigma M_{ON-S} &= 1.5' \times 0.711 \times 904.5 \text{ K} + 3' \times 1,433.5 \text{ K} + 6.84' \times (2,852 \text{ K} - 793 \text{ K}) \\ &= 965 \text{ ft-K} + 4,300 \text{ ft-K} + 14,084 \text{ ft-K} = 19,349 \text{ ft-K} \end{aligned}$$

$$\Delta b_{N-S} = \frac{9.83' \times EQ_{hcN-S}}{W_c + EQ_{vc}} = \frac{9.83' \times 811 \text{ K}}{2,852 \text{ K} - 793 \text{ K}} = 3.87 \text{ ft}$$

$$\begin{aligned} \Sigma M_{OE-W} &= 1.5' \times 0.4 \times 0.711 \times 904.5 \text{ K} + 3' \times 811 \text{ K} + 3.87' \times (2,852 \text{ K} - 793 \text{ K}) \\ &= 386 \text{ ft-K} + 2,434 \text{ ft-K} + 7,969 \text{ ft-K} = 10,787 \text{ ft-K} \end{aligned}$$

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

STONE & WEBSTER, INC.
CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 63
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				
Allowable Bearing Capacity of Cask Storage Pads			Based on Inertial Forces Combined:	
PSHA 2,000-Yr Earthquake: Case IIIB			40 % N-S, -40 % Vert, 100 % E-W	
Soil Properties:	c =	2,200 Cohesion (psf)	Footing Dimensions:	
	ϕ =	0.0 Friction Angle (degrees)	B = 30.0	Width - ft (E-W)
	γ =	80 Unit weight of soil (pcf)	L = 67.0	Length - ft (N-S)
	γ_{surch} =	100 Unit weight of surcharge (pcf)		
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-W} =	2,077 k & EQ _{H N-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)	
	$N_c = (N_q - 1) \cot(\phi)$, 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 $EQ_{H N-S} > 0$: $\theta_n = \tan^{-1}(EQ_{H E-W} / EQ_{H N-S})$	= 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		

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 64
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 IIIc: 100% N-S, -40% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$$EQ_v = -40\% \times 0.695 \times (904.5 \text{ K} + 2,852 \text{ K}) = -1,044 \text{ K}$$

$$\text{Normal force at base of the cask} = \text{Cask DL} = 2,852 \text{ K}$$

$$- 40\% \text{ of Cask } EQ_{vc} = -0.4 \times 0.695 \times 2,852 \text{ K} = -793 \text{ K} = 40\% \text{ of } a_v \times W_c$$

$$\Rightarrow N_c = 2,059 \text{ K}$$

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

$$EQ_{hc} = \text{Min of } \left[\begin{matrix} a_H & W_c & \mu & N_c \\ 0.711 \times 2,852 \text{ K} & & 0.8 \times 2,059 \text{ K} \end{matrix} \right] \Rightarrow EQ_{hc} = 1,647 \text{ K};$$

$$2,028 \text{ K} \quad 1,647 \text{ K}$$

i.e., EQ_{hc} 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 EQ_{hc} . For this case, the E-W component of $EQ_{hc} = 0.4 \times 2,028 \text{ K} = 811 \text{ K}$, and the N-S component is calculated as follows:

$$EQ_{hcN-S}^2 + EQ_{hcE-W}^2 = EQ_{hc}^2 = 1,647^2 \Rightarrow EQ_{hcN-S} = \sqrt{1,647^2 - 811^2} = 1,433.5 \text{ K}$$

Using 100% of N-S:

$$\Rightarrow EQ_{HN-S} = 1.0 \times 643 \text{ K} + 1,433.5 \text{ K} = 2,076 \text{ K}$$

Using 40% of E-W:

$$\Rightarrow EQ_{HE-W} = 0.4 \times 643 \text{ K} + 811 \text{ K} = 1,068 \text{ K}$$

$$\Rightarrow F_H = \sqrt{EQ_{HE-W}^2 + EQ_{HN-S}^2} = \sqrt{1,068^2 + 2,076^2} = 2,335 \text{ K}$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb . Note: $EQ_{vc} = -0.4 \times 0.695 \times 2,852 \text{ K} = -793 \text{ K}$

$$\Delta b_{E-W} = \frac{9.83' \times EQ_{hcE-W}}{W_c + EQ_{vc}} = \frac{9.83' \times 811 \text{ K}}{2,852 \text{ K} - 793 \text{ K}} = 3.87 \text{ ft}$$

$$\Sigma M_{ON-S} = 1.5' \times 0.4 \times 0.711 \times 904.5 \text{ K} + 3' \times 811 \text{ K} + 3.87' \times (2,852 \text{ K} - 793 \text{ K})$$

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

$$\Delta b_{N-S} = \frac{9.83' \times EQ_{hcN-S}}{W_c + EQ_{vc}} = \frac{9.83' \times 1,433.5 \text{ K}}{2,852 \text{ K} - 793 \text{ K}} = 6.84 \text{ ft}$$

$$\Sigma M_{OE-W} = 1.5' \times 0.711 \times 904.5 \text{ K} + 3' \times 1,433.5 \text{ K} + 6.84' \times (2,852 \text{ K} - 793 \text{ K})$$

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

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 65																								
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04 - 9	OPTIONAL TASK CODE																									
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: Foundation Properties:	<table style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 30%;">c =</td> <td style="width: 30%;">2,200 Cohesion (psf)</td> <td style="width: 30%;">Footing Dimensions:</td> <td style="width: 10%;"></td> </tr> <tr> <td>φ =</td> <td>0.0 Friction Angle (degrees)</td> <td>B =</td> <td>30.0 Width - ft (E-W)</td> </tr> <tr> <td>γ =</td> <td>80 Unit weight of soil (pcf)</td> <td>L =</td> <td>67.0 Length - ft (N-S)</td> </tr> <tr> <td>γ_{surch} =</td> <td>100 Unit weight of surcharge (pcf)</td> <td></td> <td></td> </tr> <tr> <td>B' =</td> <td>22.0 Effective Ftg Width - ft (E-W)</td> <td>L' =</td> <td>52.7 Length - ft (N-S)</td> </tr> <tr> <td>D_f =</td> <td>3.0 Depth of Footing (ft)</td> <td></td> <td></td> </tr> </table>				c =	2,200 Cohesion (psf)	Footing Dimensions:		φ =	0.0 Friction Angle (degrees)	B =	30.0 Width - ft (E-W)	γ =	80 Unit weight of soil (pcf)	L =	67.0 Length - ft (N-S)	γ_{surch} =	100 Unit weight of surcharge (pcf)			B' =	22.0 Effective Ftg Width - ft (E-W)	L' =	52.7 Length - ft (N-S)	D_f =	3.0 Depth of Footing (ft)		
c =	2,200 Cohesion (psf)	Footing Dimensions:																										
φ =	0.0 Friction Angle (degrees)	B =	30.0 Width - ft (E-W)																									
γ =	80 Unit weight of soil (pcf)	L =	67.0 Length - ft (N-S)																									
γ_{surch} =	100 Unit weight of surcharge (pcf)																											
B' =	22.0 Effective Ftg Width - ft (E-W)	L' =	52.7 Length - ft (N-S)																									
D_f =	3.0 Depth of Footing (ft)																											
	<table style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 30%;">FS =</td> <td style="width: 30%;">1.1 Factor of Safety required for q_{allowable}</td> <td style="width: 30%;"></td> <td style="width: 10%;">0.711 g = a_H</td> </tr> <tr> <td>F_{V Static} =</td> <td>3,757 k & EQ_V = -1,044 k → 2,712 k for F_V</td> <td></td> <td>0.695 g = a_V</td> </tr> <tr> <td>EQ_{H E-W} =</td> <td>1,068 k & EQ_{H N-S} = 2,077 k → 2,336 k for F_H</td> <td></td> <td></td> </tr> </table>				FS =	1.1 Factor of Safety required for q _{allowable}		0.711 g = a_H	F_{V Static} =	3,757 k & EQ _V = -1,044 k → 2,712 k for F _V		0.695 g = a_V	EQ_{H E-W} =	1,068 k & EQ _{H N-S} = 2,077 k → 2,336 k for F _H														
FS =	1.1 Factor of Safety required for q _{allowable}		0.711 g = a_H																									
F_{V Static} =	3,757 k & EQ _V = -1,044 k → 2,712 k for F _V		0.695 g = a_V																									
EQ_{H E-W} =	1,068 k & EQ _{H N-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)																										
$N_c = (N_q - 1) \cot(\phi), \text{ but } = 5.14 \text{ for } \phi = 0$		= 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 $EQ_{H N-S} > 0$: $\theta_n = \tan^{-1}(EQ_{H E-W} / EQ_{H N-S})$		= 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' x L')																											
FS_{actual} =	4.29 = q _{ult} / q _{actual}	> 1.1 Hence OK																										

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 66
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 IVA: 40% N-S, 100% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$$EQ_v = 100\% \times 0.695 \times (904.5 \text{ K} + 2,852 \text{ K}) = 2,611 \text{ K}$$

$$EQ_{hp} = 0.711 \times 904.5 \text{ K} = 643 \text{ K}$$

$$\begin{aligned} \text{Normal force at base of the cask} &= \text{Cask DL} = 2,852 \text{ K} \\ &+ \text{Cask } EQ_{vc} = 1. \times 0.695 \times 2,852 \text{ K} = + 1,982 \text{ K} = a_v \times W_c \\ \Rightarrow N_c &= 4,834 \text{ K} \end{aligned}$$

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

$$EQ_{hc} = \text{Min of } [0.711 \times 2,852 \text{ K} \ \& \ 0.8 \times 4,834 \text{ K}]$$

$$2,028 \text{ K} \qquad 3,867 \text{ K}$$

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, $EQ_{hc} = 811 \text{ K}$ in both the N-S and E-W directions for this case.

$$\Rightarrow EQ_{HNS} = 0.4 \times 643 \text{ K} + 811 \text{ K} = 1,068 \text{ K}$$

Since horizontal components are the same for this case, $EQ_{HEW} = EQ_{HNS}$

$$\Rightarrow F_H = \sqrt{EQ_{HEW}^2 + EQ_{HNS}^2} = \sqrt{1,068^2 + 1,068^2} = 1,510 \text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb . Note: $EQ_{vc} = 1.0 \times 0.695 \times 2,852 \text{ K} = 1,982 \text{ K}$

$$\Delta b_{E-W} = \frac{9.83' \times EQ_{hcE-W}}{W_c + EQ_{vc}} = \frac{9.83' \times 811 \text{ K}}{2,852 \text{ K} + 1,982 \text{ K}} = 1.65 \text{ ft}$$

$$\begin{aligned} \Sigma M_{\Theta NS} &= 1.5' \times 0.4 \times 0.711 \times 904.5 \text{ K} + 3' \times 811 \text{ K} + 1.65' \times (2,852 \text{ K} + 1,982 \text{ K}) \\ &= 386 \text{ ft-K} + 2,433 \text{ ft-K} + 7,976 \text{ ft-K} = 10,795 \text{ ft-K} \end{aligned}$$

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

$$\Sigma M_{\Theta EW} = \Sigma M_{\Theta NS} = 10,795 \text{ ft-K}$$

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

STONE & WEBSTER, INC.
CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 67
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

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)

Foundation Properties:

$B' = 26.6$ Effective Ftg Width - ft (E-W)

$D_f = 3.0$ Depth of Footing (ft)

Footing Dimensions:

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

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

$L' = 63.6$ Length - ft (N-S)

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

$F_{V Static} = 3,767$ k & $EQ_V = 2,611$ k \rightarrow 6,368 k for F_V

$EQ_{H E-W} = 1,068$ k & $EQ_{H N-S} = 1,068$ k \rightarrow 1,511 k for F_H

$0.711 g = a_H$

$0.695 g = a_V$

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

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

$$N_c = (N_q - 1) \cot(\phi), \text{ but } = 5.14 \text{ for } \phi = 0 = 5.14 \text{ Eq 3.6 \& Table 3.2}$$

$$N_q = e^{\pi \tan \phi} \tan^2(\pi/4 + \phi/2) = 1.00 \text{ Eq 3.6}$$

$$N_\gamma = 2 (N_q + 1) \tan(\phi) = 0.00 \text{ Eq 3.8}$$

$$s_c = 1 + (B/L)(N_q/N_c) = 1.08 \text{ Table 3.2}$$

$$s_q = 1 + (B/L) \tan \phi = 1.00 \text{ "}$$

$$s_\gamma = 1 - 0.4 (B/L) = 0.83 \text{ "}$$

$$\text{For } D_f/B \leq 1: d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 D_f/B = 1.00 \text{ Eq 3.26}$$

$$d_\gamma = 1 = 1.00 \text{ "}$$

$$\text{For } \phi > 0: d_c = d_q - (1 - d_q) / (N_q \tan \phi) = N/A$$

$$\text{For } \phi = 0: d_c = 1 + 0.4 (D_f/B) = 1.05 \text{ Eq 3.27}$$

$$m_B = (2 + B/L) / (1 + B/L) = 1.69 \text{ Eq 3.18a}$$

$$m_L = (2 + L/B) / (1 + L/B) = 1.31 \text{ Eq 3.18b}$$

$$\text{If } EQ_{H N-S} > 0: \theta_n = \tan^{-1}(EQ_{H E-W} / EQ_{H N-S}) = 0.79 \text{ rad}$$

$$m_n = m_L \cos^2 \theta_n + m_B \sin^2 \theta_n = 1.50 \text{ Eq 3.18c}$$

$$i_q = \{ 1 - F_H / [(F_V + EQ_V) + B' L' c \cot \phi] \}^m = 1.00 \text{ Eq 3.14a}$$

$$i_\gamma = \{ 1 - F_H / [(F_V + EQ_V) + B' L' c \cot \phi] \}^{m+1} = 0.00 \text{ Eq 3.17a}$$

$$\text{For } \phi = 0: i_c = 1 - (m F_H / B' L' c N_c) = 0.88 \text{ Eq 3.16a}$$

	N_c term	N_q term	N_γ term
Gross $q_{ult} =$	11,567 psf	11,267	+ 300 + 0

$$q_{all} = 10,510 \text{ psf} = q_{ult} / FS$$

$$q_{actual} = 3,762 \text{ psf} = (F_{V Static} + EQ_V) / (B' \times L')$$

$$FS_{actual} = 3.07 = q_{ult} / q_{actual} > 1.1 \text{ Hence OK}$$

CALCULATION SHEET

8010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 68
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 IVB: 40% N-S, 40% Vertical, 100% E-W*Determine forces and moments due to earthquake.*

$$EQ_v = 0.4 \times 0.695 \times (904.5 \text{ K} + 2,852 \text{ K}) = 1,044 \text{ K}$$

$$\text{Normal force at base of the cask} = \text{Cask DL} = 2,852 \text{ K}$$

$$+ 40\% \text{ of Cask } EQ_{vc} = +0.4 \times 0.695 \times 2,852 \text{ K} = +793 \text{ K} = 40\% \text{ of } a_v \times W_c$$

$$\Rightarrow N_c = 3,645 \text{ K}$$

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

$$EQ_{hc} = \text{Min of } [0.711 \times 2,852 \text{ K} \text{ \& } 0.8 \times 3,645 \text{ K}] \Rightarrow EQ_{hc} = 2,028 \text{ K, since it is } < F_{EQ \mu=0.8}$$

$$2,028 \text{ K} \quad 2,916 \text{ K}$$

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, $Eq_{hcN-S} = 0.4 \times 2,028 \text{ K} = 811 \text{ K}$ and 100% in the E-W direction, $Eq_{hcE-W} = 2,028 \text{ K}$ for this case.

Using 40% of N-S:

$$\Rightarrow EQ_{HN-S} = 0.4 \times 643 \text{ K} + 811 \text{ K} = 1,068 \text{ K}$$

Using 100% of E-W:

$$\Rightarrow EQ_{HE-W} = 1.0 \times 643 \text{ K} + 2,028 \text{ K} = 2,671 \text{ K}$$

$$\Rightarrow F_H = \sqrt{EQ_{HE-W}^2 + EQ_{HN-S}^2} = \sqrt{2,671^2 + 1,068^2} = 2,877 \text{ K}$$

*Determine moments acting on pad due to casks*See Figure 6 for identification of Δb . Note: $EQ_{vc} = 0.4 \times 0.695 \times 2,852 \text{ K} = 793 \text{ K}$

$$\Delta b_{E-W} = \frac{9.83' \times EQ_{hcE-W}}{W_c + EQ_{vc}} = \frac{9.83' \times 2,028 \text{ K}}{2,852 \text{ K} + 793 \text{ K}} = 5.47 \text{ ft}$$

$$\Sigma M_{EN-S} = 1.5' \times 0.711 \times 904.5 \text{ K} + 3' \times 2,028 \text{ K} + 5.47' \times (2,852 \text{ K} + 793 \text{ K})$$

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

$$\Delta b_{N-S} = \frac{9.83' \times EQ_{hcN-S}}{W_c + EQ_{vc}} = \frac{9.83' \times 811 \text{ K}}{2,852 \text{ K} + 793 \text{ K}} = 2.19 \text{ ft}$$

$$\Sigma M_{EE-W} = 1.5' \times 0.4 \times 0.711 \times 904.5 \text{ K} + 3' \times 811 \text{ K} + 2.19' \times (2,852 \text{ K} + 793 \text{ K})$$

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

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 69
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				
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) $\phi =$ 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:	$B' =$ 18.8 Effective Ftg Width - ft (E-W) $D_f =$ 3.0 Depth of Footing (ft)	$L' = 62.5$ Length - ft (N-S)		
		$0.711 g = a_H$ $0.695 g = a_V$		
		FS = 1.1 Factor of Safety required for $q_{allowable}$		
		$F_{V Static} = 3,757 \text{ k}$ & $EQ_V = 1,044 \text{ k} \rightarrow 4,801 \text{ k}$ for F_V		
		$EQ_{H E-W} = 2,671 \text{ k}$ & $EQ_{H N-S} = 1,068 \text{ k} \rightarrow 2,877 \text{ k}$ 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 $\phi = 0$: $d_c = 1 + 0.4 (D_f/B)$		= 1.06	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 $EQ_{H N-S} > 0$: $\theta_n = \tan^{-1}(EQ_{H E-W} / EQ_{H N-S})$		= 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	

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 70
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 IVC: 100% N-S, 40% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$$EQ_v = 0.4 \times 0.695 \times (904.5 \text{ K} + 2,852 \text{ K}) = 1,044 \text{ K}$$

$$\text{Normal force at base of the cask} = \text{Cask DL} = 2,852 \text{ K}$$

$$+ 40\% \text{ of Cask } EQ_{vc} = 0.4 \times 0.695 \times 2,852 \text{ K} = + 793 \text{ K} = 40\% \text{ of } a_v \times W_c$$

$$\Rightarrow N_c = 3,645 \text{ K}$$

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

$$EQ_{hc} = \text{Min of } [0.711 \times 2,852 \text{ K} \text{ \& } 0.8 \times 3,645 \text{ K}] \Rightarrow EQ_{hc} = 2,028 \text{ K, since it is } < F_{EQ \mu=0.8}$$

$$2,028 \text{ K} \quad 2,916 \text{ K}$$

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, $EQ_{hcN-S} = 2,028 \text{ K}$ and 40% in the E-W direction, $EQ_{hcE-W} = 0.4 \times 2,028 \text{ K} = 811 \text{ K}$ for this case.

Using 100% of N-S:

$$\Rightarrow EQ_{HN-S} = 1.0 \times 643 \text{ K} + 2,028 \text{ K} = 2,671 \text{ K}$$

Using 40% of E-W:

$$\Rightarrow EQ_{HE-W} = 0.4 \times 643 \text{ K} + 811 \text{ K} = 1,068 \text{ K}$$

$$\Rightarrow F_H = \sqrt{EQ_{HE-W}^2 + EQ_{HN-S}^2} = \sqrt{1,068^2 + 2,671^2} = 2,877 \text{ K}$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb . Note: $EQ_{vc} = 0.4 \times 0.695 \times 2,852 \text{ K} = 793 \text{ K}$

$$\Delta b_{E-W} = \frac{9.83' \times EQ_{hcE-W}}{W_c + EQ_{vc}} = \frac{9.83' \times 811 \text{ K}}{2,852 \text{ K} + 793 \text{ K}} = 2.19 \text{ ft}$$

$$\Sigma M_{EN-S} = 1.5' \times 0.4 \times 0.711 \times 904.5 \text{ K} + 3' \times 811 \text{ K} + 2.19' \times (2,852 \text{ K} + 793 \text{ K})$$

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

$$\Delta b_{N-S} = \frac{9.83' \times EQ_{hcN-S}}{W_c + EQ_{vc}} = \frac{9.83' \times 2,028 \text{ K}}{2,852 \text{ K} + 793 \text{ K}} = 5.47 \text{ ft}$$

$$\Sigma M_{EE-W} = 1.5' \times 0.711 \times 904.5 \text{ K} + 3' \times 2,028 \text{ K} + 5.47' \times (2,852 \text{ K} + 793 \text{ K})$$

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

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

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 71
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04 - 9	OPTIONAL TASK CODE	
DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES				
Allowable Bearing Capacity of Cask Storage Pads			Based on Inertial Forces Combined: 100 % N-S, 40 % Vert, 40 % E-W	
PSHA 2,000-Yr Earthquake: Case IVC				
Soil Properties:	c =	2,200 Cohesion (psf)	Footing Dimensions:	
	φ =	0.0 Friction Angle (degrees)	B = 30.0	Width - ft (E-W)
	γ =	80 Unit weight of soil (pcf)	L = 67.0	Length - ft (N-S)
	γ _{surch} =	100 Unit weight of surcharge (pcf)		
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)		
	FS =	1.1 Factor of Safety required for q _{allowable}	0.711 g = a _H	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-W} =	1,068 k & EQ _{H N-S} = 2,671 k → 2,877 k for F _H		
q_{ult} = c N_c s_c d_c i_c + γ_{surch} D_f N_q s_q d_q i_q + 1/2 γ B N_γ s_γ d_γ i_γ			General Bearing Capacity Equation, based on Winterkorn & Fang (1975)	
	N _c = (N _q - 1) cot(φ), but = 5.14 for φ = 0	= 5.14	Eq 3.6 & Table 3.2	
	N _q = e ^{π tan φ} tan ² (π/4 + φ/2)	= 1.00	Eq 3.6	
	N _γ = 2 (N _q + 1) tan(φ)	= 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 φ	= 1.00	"	
	s _γ = 1 - 0.4 (B/L)	= 0.82	"	
	For D _f /B ≤ 1: d _q = 1 + 2 tan φ (1 - sin φ) ² D _f /B	= 1.00	Eq 3.26	
	d _γ = 1	= 1.00	"	
	For φ > 0: d _c = d _q - (1 - d _q) / (N _q tan φ)	= N/A		
	For φ = 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-S} > 0: θ _n = tan ⁻¹ (EQ _{H E-W} / EQ _{H N-S})	= 0.38 rad		
	m _n = m _L cos ² θ _n + m _B sin ² θ _n	= 1.36	Eq 3.18c	
	i _q = { 1 - F _H / [(F _V + EQ _V) + B' L' c cot φ] } ^m	= 1.00	Eq 3.14a	
	i _γ = { 1 - F _H / [(F _V + EQ _V) + B' L' c cot φ] } ^{m+1}	= 0.00	Eq 3.17a	
	For φ = 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' x L')		
	FS _{actual} = 2.98	= q _{ult} / q _{actual} > 1.1 Hence OK		

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 72
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

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 ($FS \geq 1.1$). 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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 73
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 ($\mu = 0.8$, 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.

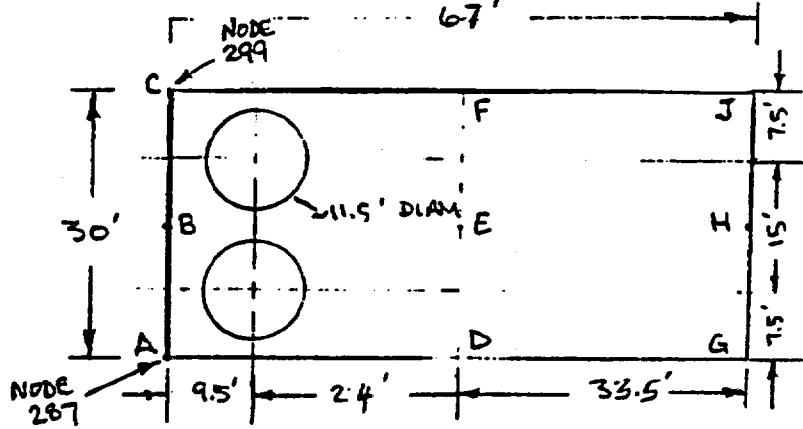
NOTED JUN 19 1997 P. I.
1-26-00

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.65

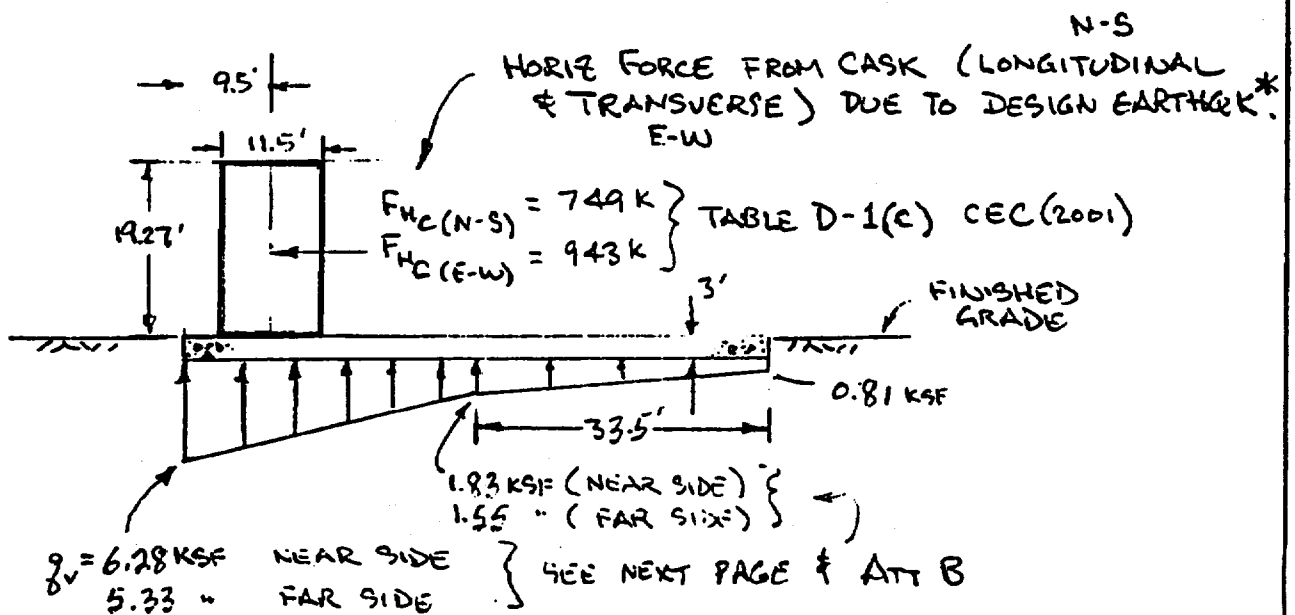
CALCULATION IDENTIFICATION NUMBER				PAGE 74
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

DYN BEARING CAPACITY OF PADS 2-CASK CASE



PLAN

NORTH



ELEV

* STRESSES AT PAD/SOIL INTERFACE OBTAINED FROM
CEC(2001) CALC 05996.02-G(P017)-2, REV.3 - COPIES OF
PERTINENT PAGES ARE INCLUDED IN ATT B OF THIS CALC

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.85

7 CALCULATION IDENTIFICATION NUMBER				PAGE <u>75</u>
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(8)	04-9		

DYN BEARING CAPACITY OF PAD: 2-CASK CASE

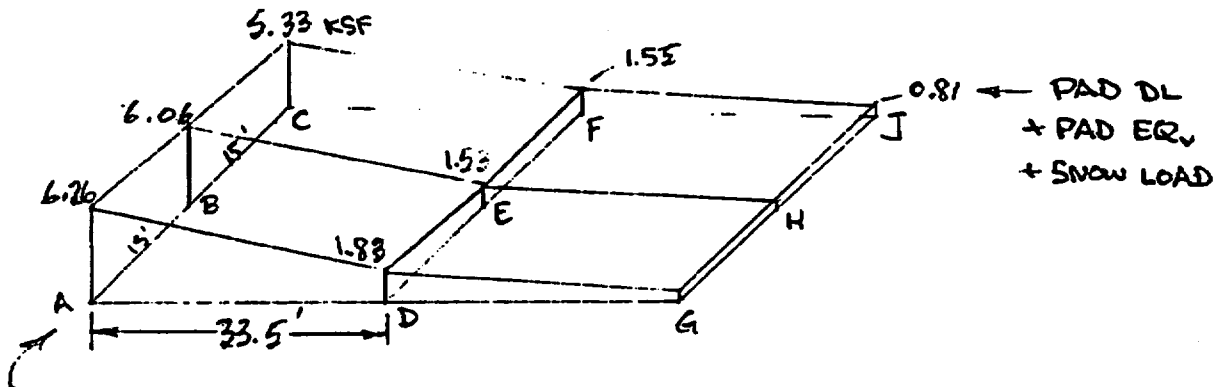
SOIL BEARING PRESSURES ARE BASED ON INFO FROM CEC(2001) INCLUDED IN ATT B AND ARE SUMMARIZED IN TABLE 1.

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:

$$F_v = \left[\frac{15'}{2} \times (6.26 + 2 \times 6.06 + 5.33) + \frac{15'}{2} \times (1.88 + 2 \times 1.53 + 1.55) \right] \frac{33.5}{2}$$

$$F_v \approx 3790 \text{ K FOR LOADED } 1/2 \text{ OF PAD}$$

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

$$A_{DF} \sim 48.30 \frac{\text{K}}{\text{FT}} = 30' \times q_{\text{AUG}_{DF}} \Rightarrow q_{\text{AUG}_{DF}} = 1.61 \text{ KSF}$$

~~4086~~

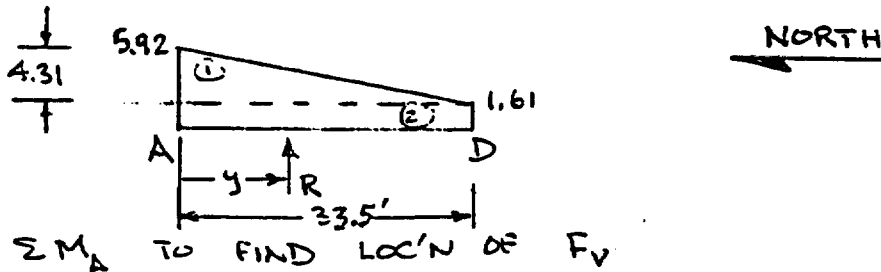
STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.65

2 CALCULATION IDENTIFICATION NUMBER				PAGE 76
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

DYN BEARING CAPACITY OF PAD. 2-CASE CASE

DETERMINE ~ ECCENTRICITY OF F_v IN L DIRECTION (N-S)
USING $q_{AVG AC}$ & $q_{AVG DE}$

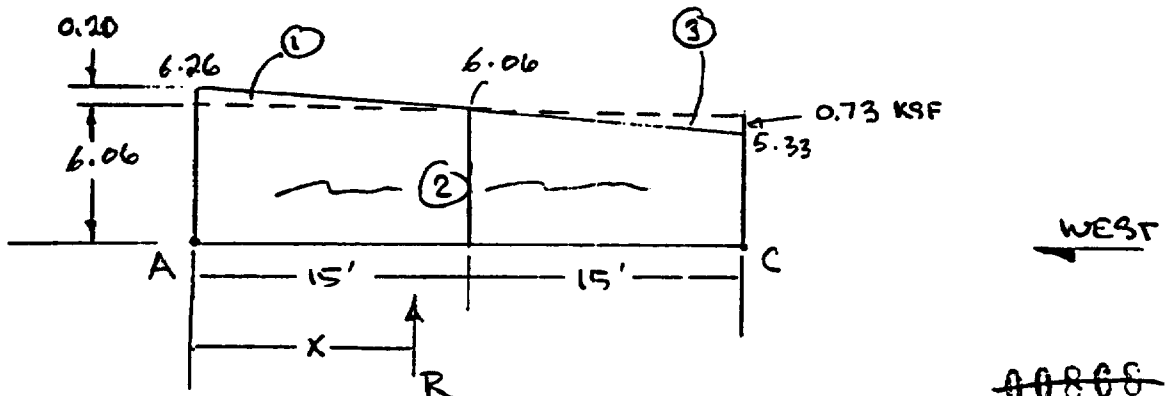


$$R_y = \frac{1}{2} 4.31 \times 33.5 \times \frac{1}{3} 33.5 + 1.61 \times 33.5 \times \frac{33.5}{2}$$

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

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

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



~~00808~~

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 8070.65

9 CALCULATION IDENTIFICATION NUMBER				PAGE <u>77</u>
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

DYN BEARING CAPACITY OF PAD : 2-CASK CASE

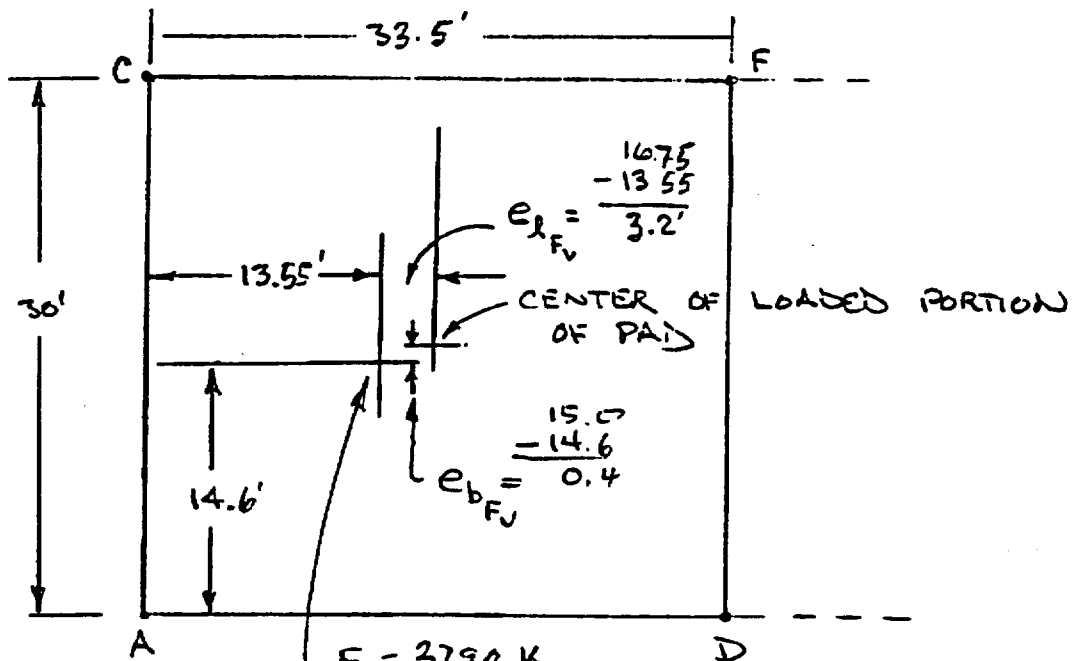
ΣM_A

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

$$\Sigma F_V = R = 177.8 \text{ K/FT}$$

$$2597.6$$

$$\therefore X = \frac{\Sigma M_A}{\Sigma F_V} = \frac{2597.6 \text{ K-FT/FT}}{177.8 \text{ K/FT}} = 14.61'$$



$$F_v = 3790 \text{ K}$$

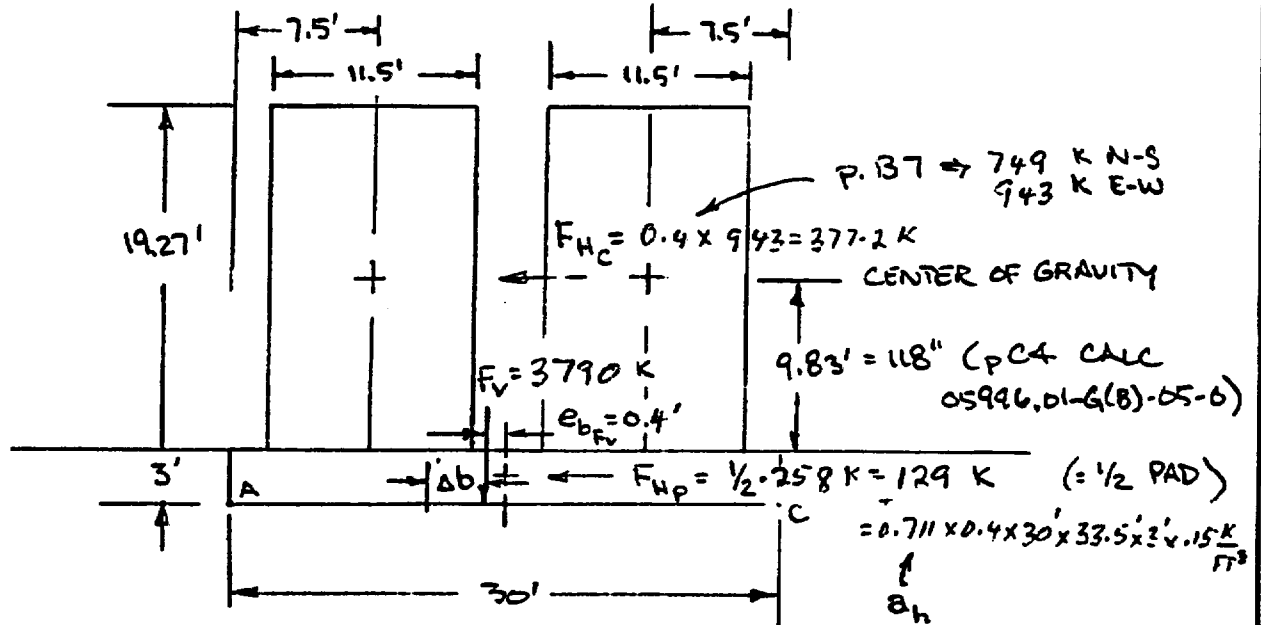
POINT OF APPLICATION OF F_v DUE TO PAD (DL+EQ) & CASKS (LL+EQ) FOR 2-CASK CASE

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 6010.65

10 CALCULATION IDENTIFICATION NUMBER				PAGE <u>78</u>
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

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

$$R_{\Delta b} = 15' \times \frac{129 \text{ K}}{2} + (3' + 9.83') 377.2 \text{ K} \quad \text{2-CASKS TRANSVERSE LOADING (E-W)}$$

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

$$\therefore \Delta b = \frac{193.5 + 4837.5}{3790 \text{ K}} \text{ K-FT} = 1.30 \text{ FT}$$

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

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

* NOTE: HORIZ INERTIA OF OTHER $\frac{1}{2}$ OF PAD $\left(\frac{322 \text{ K}}{30' \times 33.5'} = 0.33 \text{ Ksf} \right)$
IS RESISTED BY $C = 1.4 \text{ Ksf} \ \& \ N \tan \phi$ w/ $\phi = 21.3^\circ$
ALONG BASE OF THAT $\frac{1}{2}$ OF PAD

~~60870~~

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER			
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-9	OPTIONAL TASK CODE
			PAGE <u>79</u>

DYN BEARING CAPACITY OF PAD: 2-CASK CASE

CALCULATE L' SIMILARLY FOR LONGITUDINAL DIRECTION

$$F_{HCL} = 300 \text{ K} (= 40\% Q_{yd \text{ max}} \text{ FROM P B7 FOR 2 CASKS})$$

ΣM_{FV} 1/2 PAD 2-CASKS LONGITUDINAL LOADS

$$R \Delta l = 1.5' \times \frac{129}{2} + (3' + 9.83') (300 \text{ K})$$

$$\uparrow \hat{L} = F_v = 3790 \text{ K}$$

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

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

$$L' = L - 2e_l = 33.5 - 2 \times 4.27' = 24.96' \quad \leftarrow < 26.6' \right. \\ \left. \therefore \text{THIS} = B' \right. \\ \left. \text{? } L' = 26.6' \right.$$

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

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

$$F_{HEW} = 377.2 \text{ K} + 129 \text{ K} = 506.2 \text{ K} \quad \left| \quad F_{HN-S} = 300 \text{ K N-S} \right. \\ \uparrow \quad \quad \quad \uparrow \text{1/2 PAD } EQ_{Hh} \quad \quad \quad + \frac{129 \text{ K PAD}}{429 \text{ K N-S}} \\ \text{2-CASK } EQ_{Hh} (\text{P B7}) \quad \quad \quad FS = 1.1$$

$$F_v = 3790 \text{ K FOR 2-CASK (STATIC + DYN)}$$

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

$$D_f = 3' \quad (\text{TOP OF PAD FLUSH WITH GRADE})$$

$$\text{FOR DYN LOADS, } \phi = 0^\circ \quad C = 2.2 \text{ KSF}$$

~~00871~~

[6]

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

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)	Footing Dimensions:	
φ =	0.0 Friction Angle (degrees)	B = 30.0	Width - ft (E-W)
γ =	80 Unit weight of soil (pcf)	L = 67.0	Length - ft (N-S)
γ _{surch} =	100 Unit weight of surcharge (pcf)		

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 = 3,780 k (Includes EQ_v)

EQ_{H E-W} = 506 k & EQ_{H N-S} = 429 k → 664 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 φ = 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/B ≤ 1: $d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 D/B$	= 1.00	Eq 3.26
$d_\gamma = 1$	= 1.00	"
For φ > 0: $d_c = d_q - (1 - d_q) / (N_q \tan \phi)$	= N/A	
For φ = 0: $d_c = 1 + 0.4 (D/B)$	= 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-S} > 0: $\theta_n = \tan^{-1}(EQ_{H E-W} / EQ_{H N-S})$	= 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 φ = 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' x L')	
FS _{actual} =	2.18	= q _{ult} / q _{actual} > 1.1 Hence OK	

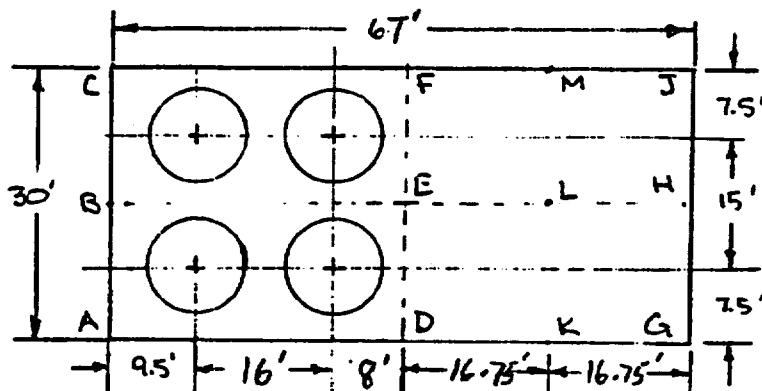
[geot]05996\calc\bmng_cap\PadWint_Fang-8.xls Sheet 2-Cask

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

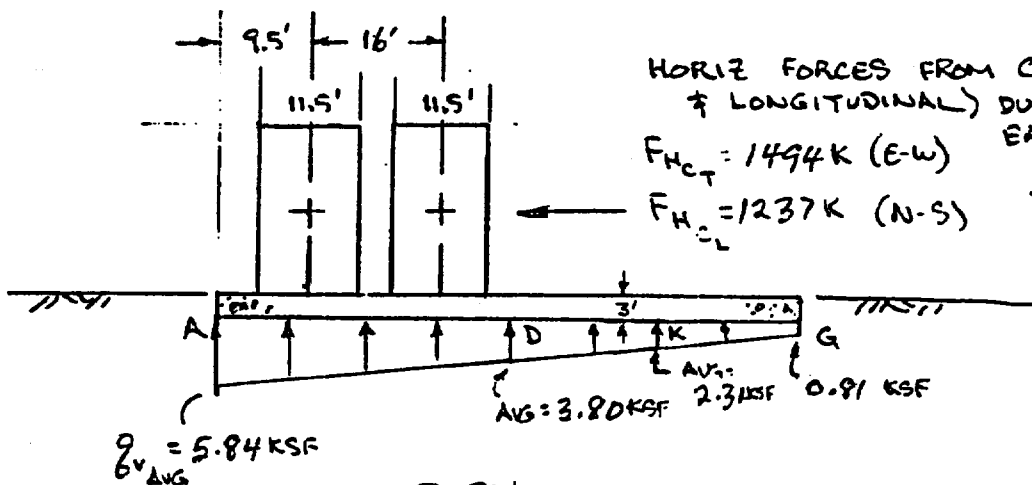
▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE <u>81</u>
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



PLAN



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				PAGE <u>82</u>
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-9	OPTIONAL TASK CODE	

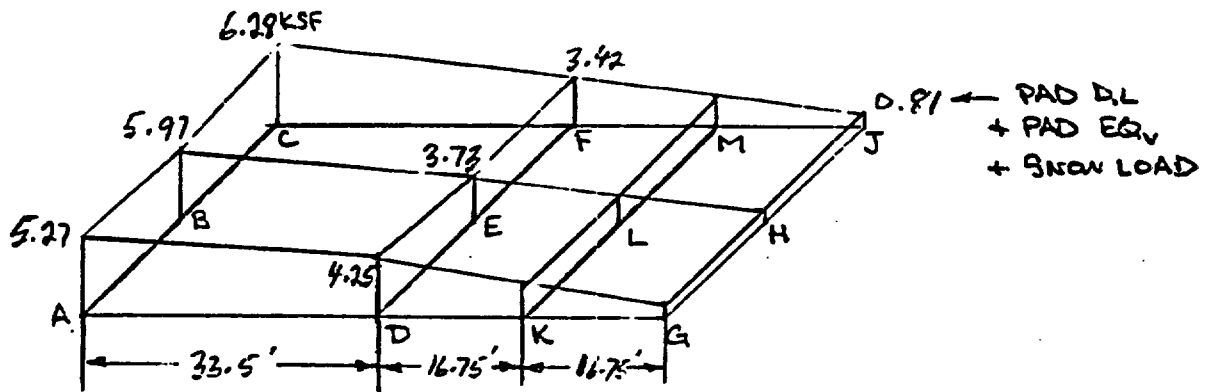
DYN BEARING CAPACITY OF PAD: 4-CASK CASE

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 ECR = 0.31 KSF
SNOW LOAD = 0.045 KSF

LL OF CASKS = 171 KSF ALONG LINE AC & IS ASSUMED TO DECREASE LINEARLY TO 0 ALONG LINE GJ.

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



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

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

LINEARLY DISTRIBUTE STATIC + DYN LOADING FROM LINE DF TO 50.25' AWAY FROM LINE AC & DETERMINE \bar{F}_V

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~~

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.65

B CALCULATION IDENTIFICATION NUMBER				PAGE <u>63</u>
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 64-9	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

CALCULATE F_v

ALONG LINE AREA = K/ft

$$AC \quad \frac{15'}{2} (5.27 + 2 \times 5.97 + 6.28) \text{ KSF} = 176.18 \text{ K/FT}$$

$$DF \quad \frac{15'}{2} (4.25 + 2 \times 3.73 + 3.42) = 113.48$$

$$KM \quad \frac{15'}{2} (2.53 + 2 \times 2.27 + 2.12) = 68.93$$

$$F_v \sim \frac{33.5}{2} (176.18 + 113.48) \text{ K/} + \frac{16.25}{2} (113.48 + 68.93) \text{ K/}$$

$$F_v = 4851.8 \text{ K} + 1527.7 \text{ K} = \underline{6379.5 \text{ K}}$$

ESTIMATE LOCATION WHERE F_v ACTS ON 30' X 50.25 PORTION OF PAD.

NOTE AUG VERT STRESS ALONG LINES.

LINE

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

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

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

~~00876~~

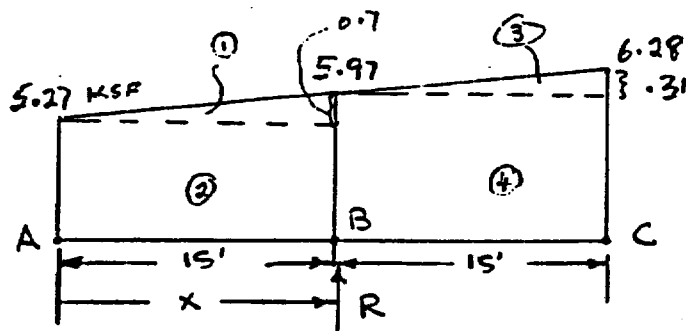
STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE <u>84</u>
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	34-9		

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 Δ	K/FT	MOMENT ARM (FT)	MOMENT
①	$\frac{1}{2} \times 0.7 \frac{\text{K}}{\text{FT}^2} \times 15' = 5.25$		$\frac{2}{3} \times 15' = 10'$	52.5
②	$5.27 \frac{\text{K}}{\text{FT}^2} \times 15' = 79.05$		$\frac{1}{2} \times 15' = 7.5'$	592.88
③	$\frac{1}{2} \times 0.31 \frac{\text{K}}{\text{FT}^2} \times 15' = 2.325$		$15 + \frac{2}{3} \times 15' = 25'$	58.125
④	$5.97 \frac{\text{K}}{\text{FT}^2} \times 15' = 89.55$		$15 + \frac{1}{2} \times 15' = 22.5'$	2014.875

$$F_v = \Sigma = 176.175$$

$$R_x = \Sigma = 2718.38$$

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

$$e_{B F_v} = \frac{B}{2} - x = 15' - 15.4' = 0.4'$$

00857

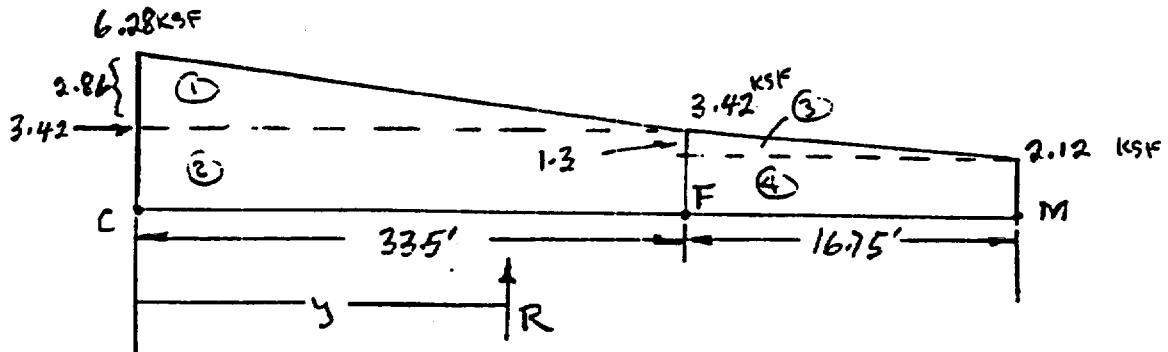
STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.85

C CALCULATION IDENTIFICATION NUMBER				PAGE <u>85</u>
J.O. OR W.O. NO.	D VISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05946.02	G(B)	04-9		

DYN BEARING CAPACITY OF PAD: 4-CASE CASE

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



ΣM_A TO FIND y
FORCE (K) MOMENT ARM (FT) MOMENT
K-FT/L.F.

① $\frac{1}{2} \times 2.86 \times 33.5 = 47.905$

$\frac{1}{3} \times 33.5 = 11.16'$

534.9

② $3.42 \times 33.5 = 114.67$

$\frac{1}{2} \times 33.5 = 16.75'$

1919.0

③ $\frac{1}{2} \times 1.30 \times 16.75 = 10.89$

$33.5 + \frac{1}{3} \times 16.75 = 39.08$

425.6

④ $2.12 \times 16.75 = 35.51$

$33.5 + \frac{1}{2} \times 16.75 = 41.875$

1487.0

$R = \Sigma F_v = 208.88 \text{ K}$

$R_y = \Sigma M = 4366.5$

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

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

AS SHOWN
ON NEXT PAGE

~~00270~~

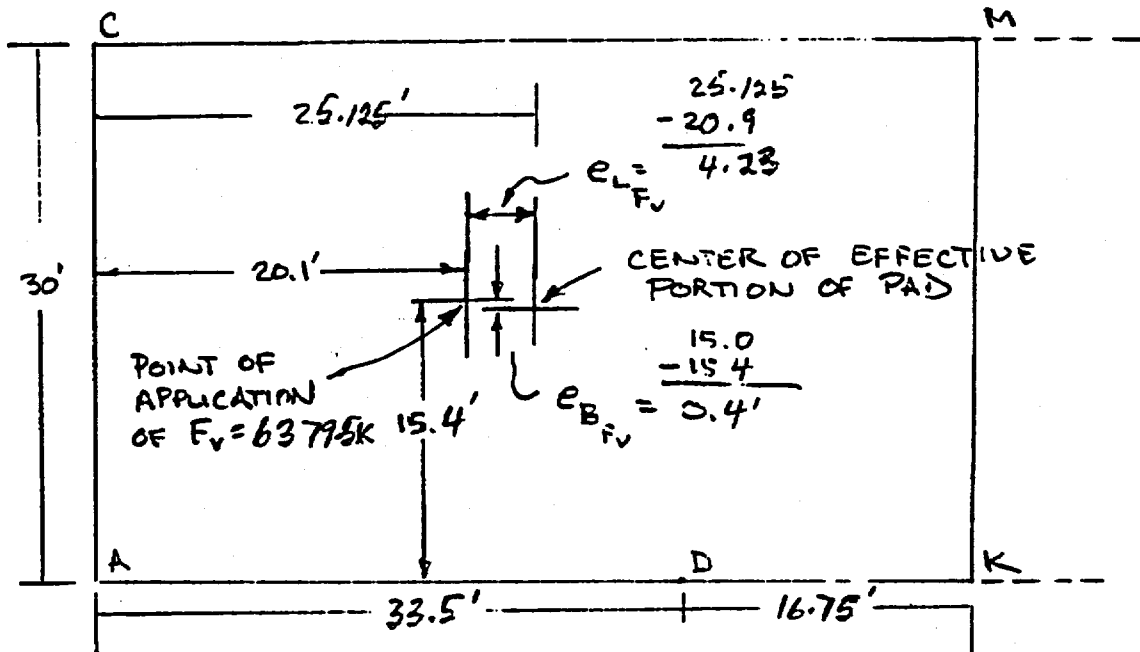
STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5310.85

D CALCULATION IDENTIFICATION NUMBER				PAGE <u>86</u>
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

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



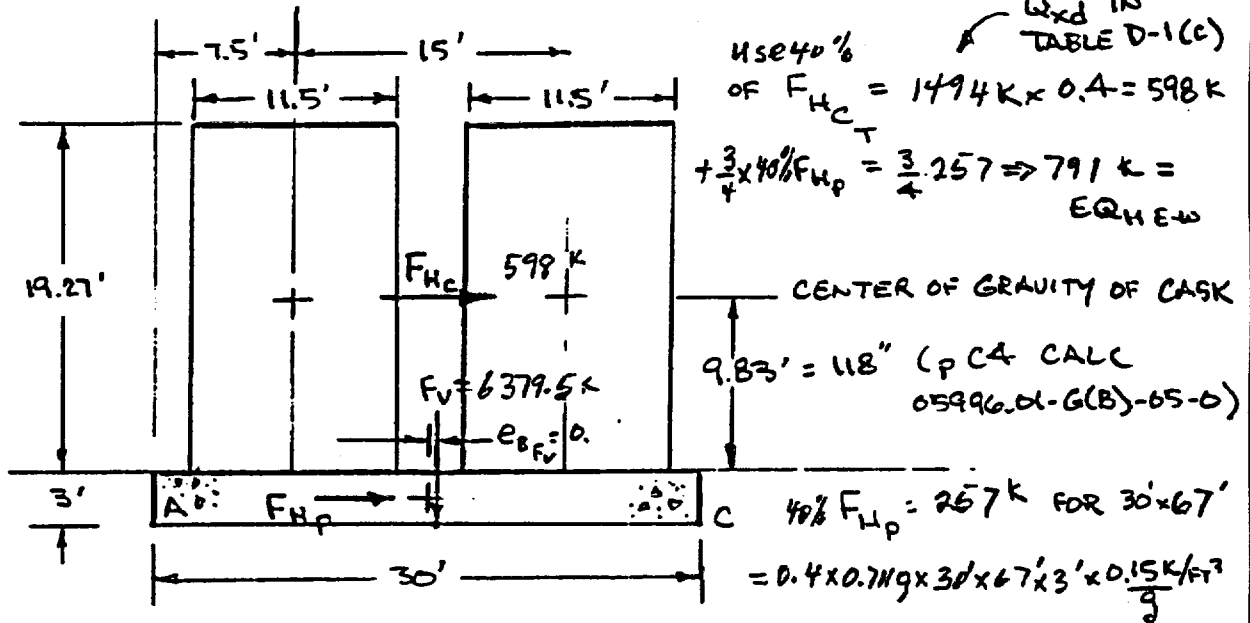
~~00870~~

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.85

E CALCULATION IDENTIFICATION NUMBER				PAGE <u>87</u>
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



ΣM_{F_v} TO FIND LOC'N OF RESULTANT TO RESIST F_H 'S

3/4 PAD \downarrow 40% 4-CASK TRANSVERSE F_H

$$R \Delta b = 15' \times \frac{3}{4} \times 257 K + (3' + 9.83') 598 K$$

$L = F_v = 6379.5 K$ ON 30×50.25 PORTION OF PAD

$$\therefore \Delta b = \frac{289 + 7.672}{6379.5 K} K-FT = 1.25' = e_{B_{F_H}}$$

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

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

~~00000~~

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

AS 5010.65

F CALCULATION IDENTIFICATION NUMBER				PAGE <u>88</u>
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

CALCULATE L' SIMILARLY FOR LONGITUDINAL F_H

$$\begin{aligned}
 & 40\% \text{ of } 1237 \text{ K} \\
 & F_{HCL} = 495 \text{ K} = Q_{yd \max} \text{ 4 CASKS} \quad \text{TABLE D-1(G) PBT} \\
 & + \frac{3}{4} \times 40\% F_{HP} = \frac{3}{4} 257 \Rightarrow EQH \text{ N-S} = 698 \text{ K} \quad \begin{matrix} F_{HCL} \\ \downarrow \text{ 4 CASKS} \end{matrix} \\
 & \sum M_{FV} \quad R \Delta l = 1.5' \times \frac{3}{4} \times 257 \text{ K} + (3' + 9.83') 495 \text{ K} \\
 & \uparrow \\
 & L = F_v = 6379.5 \text{ K ON EFFECTIVE PORTION OF PAD (30' x 50.25')
 \end{aligned}$$

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

$$e_L = e_{LFV} + e_{LFH} = 4.23' + 1.04 = 5.27'$$

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

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

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

$F_v = 6379.5 \text{ K}$ FOR 4-CASK CASE (STATIC + DYN)

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

$$EQH \text{ N-S} = \quad + 495 = 688 \text{ N-S}$$

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

$$\phi = 0^\circ \quad C = 2.2 \text{ KSF}$$

~~0.0051~~ [6]

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 89
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04 - 9	OPTIONAL TASK CODE	

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

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:

$B' = 26.7$ 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)

$EQ_{H\ E-W} = 791$ k & $EQ_{H\ N-S} = 688$ k \rightarrow 1,048 k for F_H

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

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

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

$$= 5.14 \quad \text{Eq 3.6 \& Table 3.2}$$

$$N_q = e^{x \tan \phi} \tan^2(\pi/4 + \phi/2)$$

$$= 1.00 \quad \text{Eq 3.6}$$

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

$$= 0.00 \quad \text{Eq 3.8}$$

$$\epsilon_c = 1 + (B/L)(N_q/N_c)$$

$$= 1.13 \quad \text{Table 3.2}$$

$$\epsilon_q = 1 + (B/L) \tan \phi$$

$$= 1.00 \quad "$$

$$\epsilon_\gamma = 1 - 0.4 (B/L)$$

$$= 0.73 \quad "$$

$$\text{For } D_f/B \leq 1: d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 D_f/B$$

$$= 1.00 \quad \text{Eq 3.26}$$

$$d_\gamma = 1$$

$$= 1.00 \quad "$$

$$\text{For } \phi > 0: d_c = d_q - (1 - d_q) / (N_q \tan \phi)$$

$$= \text{N/A}$$

$$\text{For } \phi = 0: d_c = 1 + 0.4 (D_f/B)$$

$$= 1.04 \quad \text{Eq 3.27}$$

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

$$= 1.69 \quad \text{Eq 3.18a}$$

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

$$= 1.31 \quad \text{Eq 3.18b}$$

$$\text{If } EQ_{H\ N-S} > 0: \theta_n = \tan^{-1}(EQ_{H\ E-W} / EQ_{H\ N-S})$$

$$= 0.85 \quad \text{rad}$$

$$m_n = m_L \cos^2 \theta_n + m_B \sin^2 \theta_n$$

$$= 1.53 \quad \text{Eq 3.18c}$$

$$i_q = \{ 1 - F_H / [(F_v + EQ_v) + B' L' c \cot \phi] \}^m$$

$$= 1.00 \quad \text{Eq 3.14a}$$

$$i_\gamma = \{ 1 - F_H / [(F_v + EQ_v) + B' L' c \cot \phi] \}^{m+1}$$

$$= 0.00 \quad \text{Eq 3.17a}$$

$$\text{For } \phi = 0: i_c = 1 - (m F_H / B' L' c N_c)$$

$$= 0.87 \quad \text{Eq 3.16a}$$

N_c term

N_q term

N_γ term

$$\text{Gross } q_{ult} = 11,879 \text{ psf} = 11,579 + 300 + 0$$

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

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

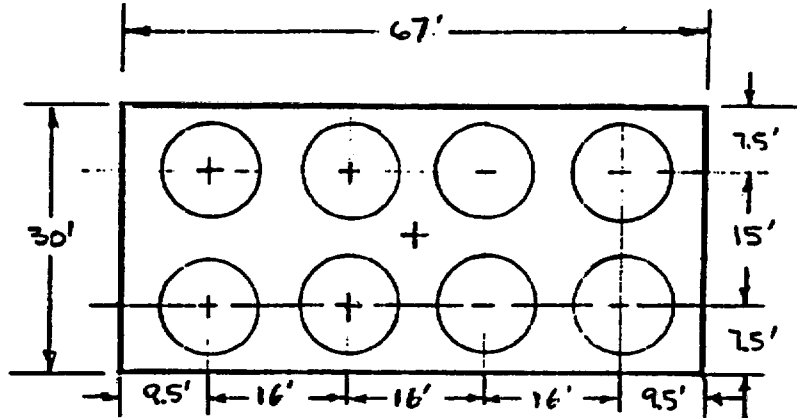
$$FS_{actual} = 1.97 = q_{ult} / q_{actual} > 1.1 \text{ Hence OK}$$

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

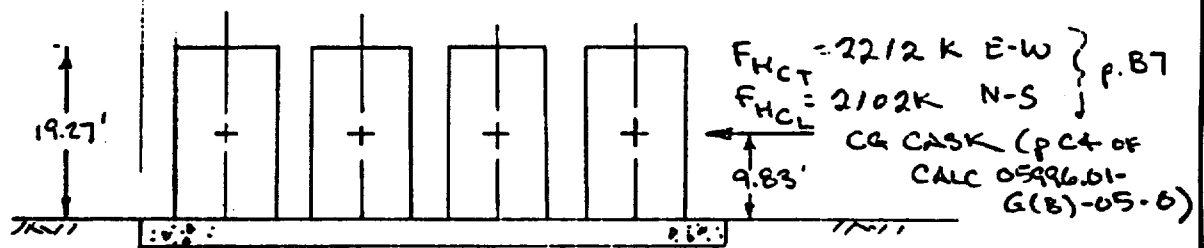
▲ 8010.85

CALCULATION IDENTIFICATION NUMBER				PAGE <u>90</u>
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-9	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 8-CASK CASE



PLAN



~~00884~~

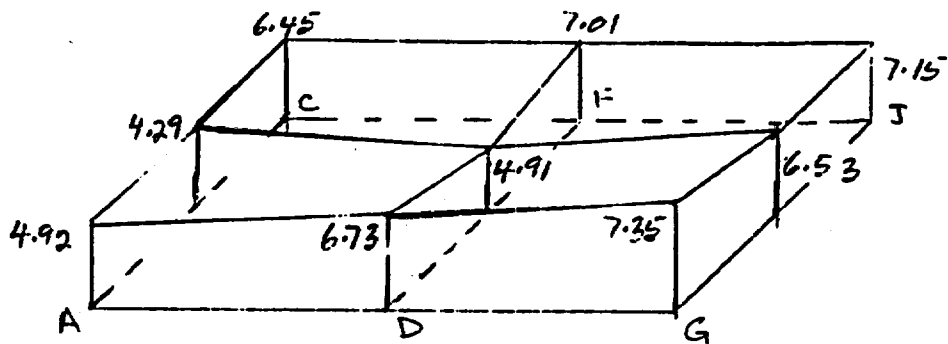
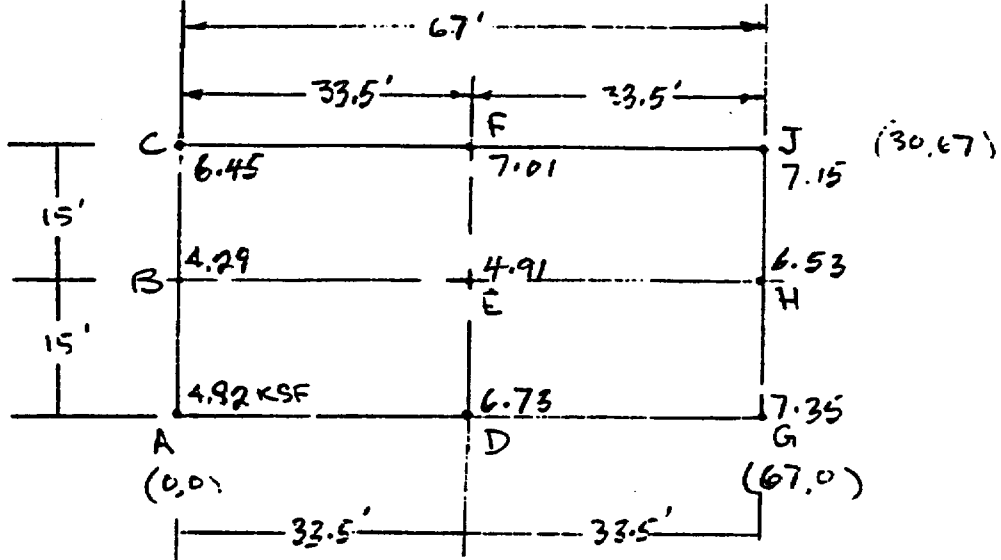
STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE <u>91</u>
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(8)	04-9		

DYN BEARING CAPACITY OF PAD: B-CASE CASE

SOIL BEARING PRESSURES FROM TABLE 1



~~00885~~

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE <u>92</u>
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

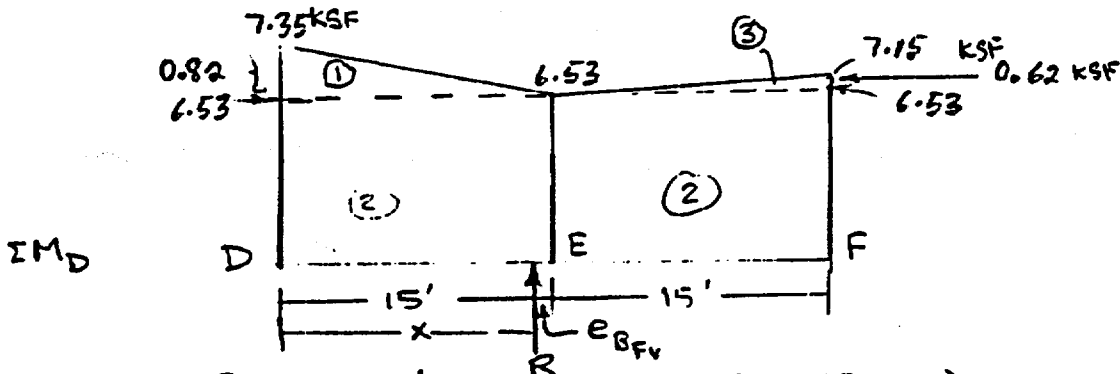
DUN BEARING CAPACITY OF PAIS: 8-CASK CASE

CALCULATE F_v :

ALONG LINE	AREA = K/FT	F_v (K/FT)	$\frac{1}{2}$ 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

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

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



AREA	K/FT	MOMENT ARM (FT)	MOMENT K-FT/FT
① $\frac{1}{2} 0.92 \times 15'$	$= 6.15$	$\frac{1}{3} \times 15' = 5'$	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	
$\therefore x = \frac{Rx}{R} = \frac{3085.5 \text{ K-FT/FT}}{206.7 \text{ K/FT}} = 14.92'$		$e_{BF_v} = \frac{30}{2} - 14.92 = 0.08'$	

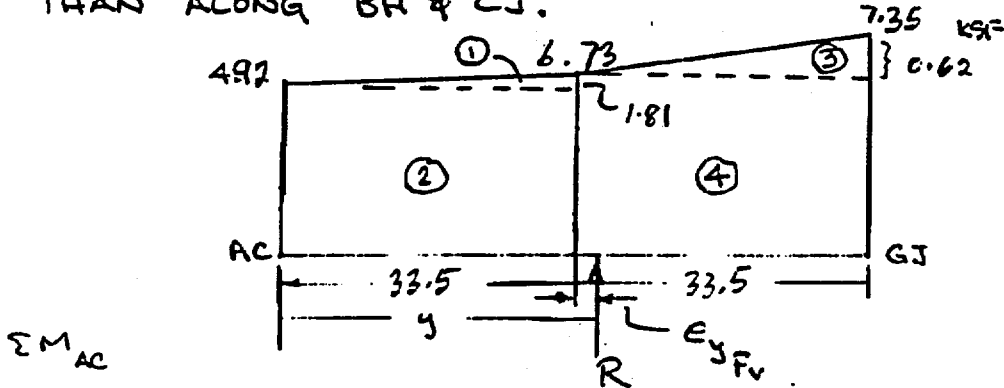
STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE <u>93</u>
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

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 $\frac{K \cdot FT}{FT}$
① $\frac{1}{2} (1.81 \text{ ksf}) \times 33.5 = 30.32$		$\frac{2}{3} \times 33.5 = 22.33'$	677.69
② $4.92 \text{ ksf} \times 33.5 = 164.82$		$\frac{1}{2} \times 33.5 = 16.75'$	2760.74
③ $\frac{1}{2} (7.35 - 6.73) \text{ ksf} \times 33.5 = 10.38$		$33.5' + \frac{2}{3} 33.5 = 55.83'$	580.07
④ $6.73 \text{ ksf} \times 33.5 = 225.46$		$33.5' + \frac{1}{2} 33.5 = 50.25'$	11,329.37
$R = 430.98 \text{ K/FT}$			$R_y = 15,347.3 \frac{K \cdot FT}{FT}$

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

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

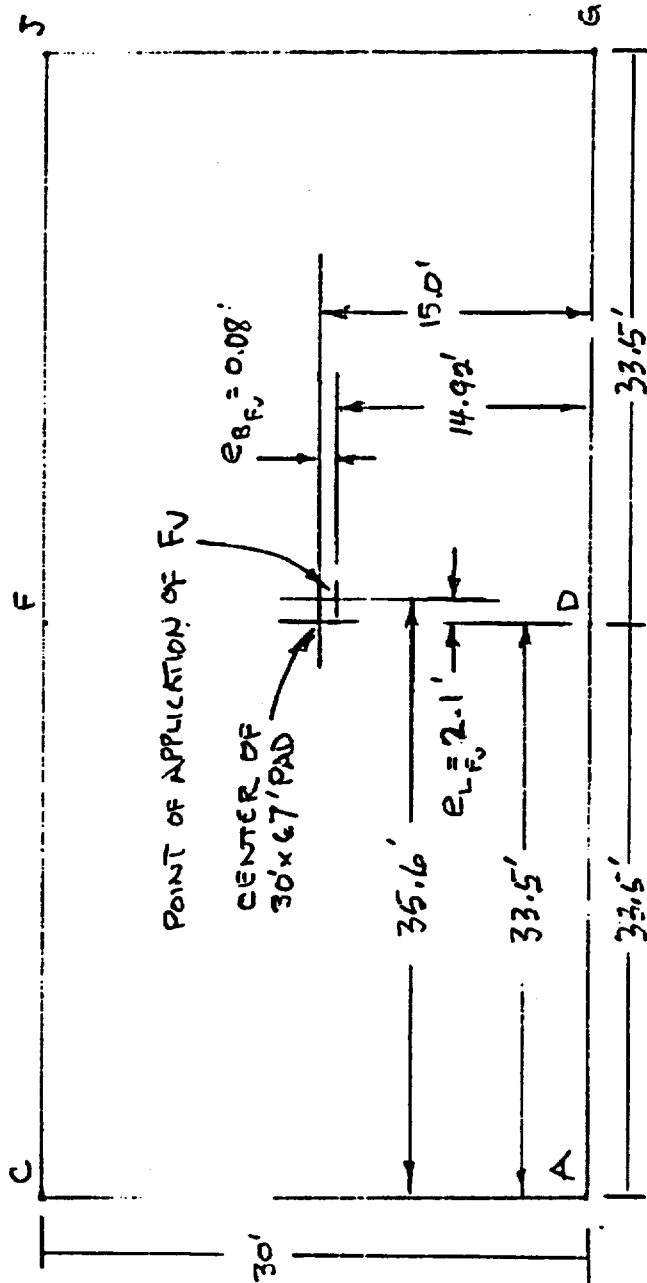
~~00887~~

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE <u>94</u>
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

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



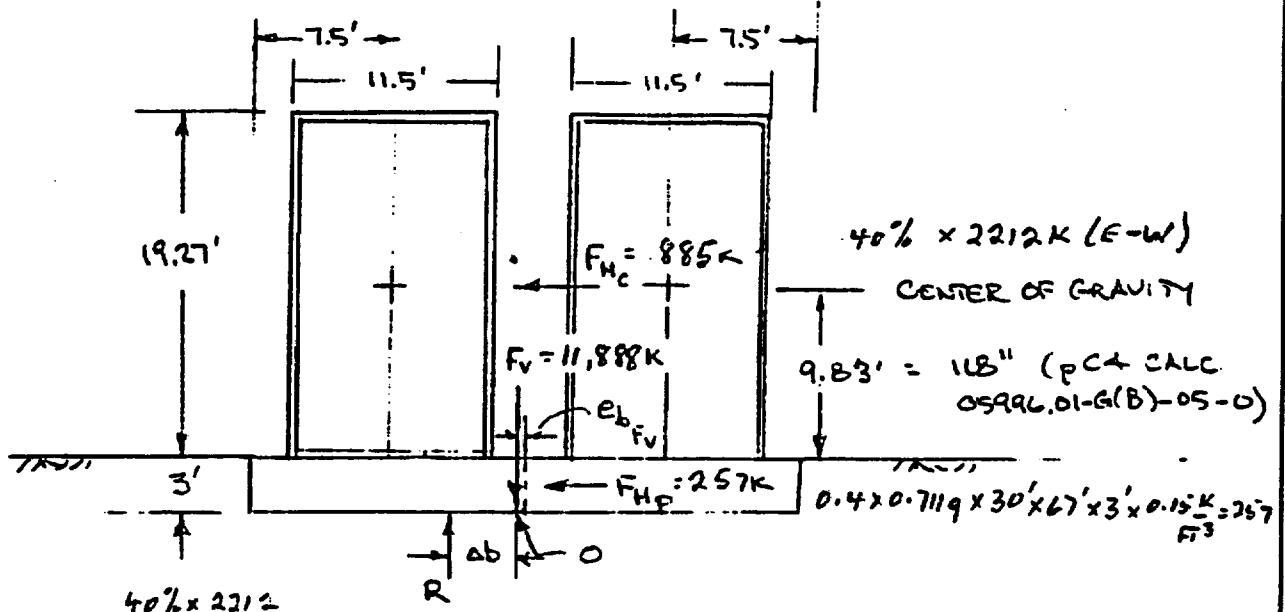
~~00000~~

CALCULATION SHEET

A 5010 65

2 CALCULATION IDENTIFICATION NUMBER				PAGE <u>95</u>
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



$$F_{Hc} = 885 \text{ K} \quad \text{FOR 2-CASK CASE (F B7)} \\ \text{TRANVERSE DIRECTION E-W}$$

$$F_{Hp} = 0.4 \times 711 \text{ g} \times 30' \times 67' \times 3' \times \frac{0.15 \text{ KCF}}{g} = 257 \text{ K}$$

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

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

$$R_{\Delta b} = 1.5' \times 257 \text{ K} + (3' + 9.83') 885 \text{ K}$$

$$\Delta b = \frac{386 + 11,355 \text{ K-FT}}{11,988 \text{ K}} = 0.99 \text{ FT}$$

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

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

~~00888~~

CALCULATION SHEET

A 5010.85

3 CALCULATION IDENTIFICATION NUMBER				PAGE <u>96</u>
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

DYN BEARING CAPACITY OF PAD: 8-CASK CASE

SIMILARLY FOR LONGITUDINAL DIRECTION

40% OF 2102K

$$F_{Hc} = 841 \text{ K} \quad \text{ADD } F_{HP} = 257 \text{ K} \Rightarrow EQ_{HN-S} = 1098 \text{ K}$$

\uparrow PBT

$$\Sigma M_o \quad R = F_v \quad \text{PAD} \quad \text{CASKS}$$

$$11,888 \text{ K } \Delta l = 1.5' \times 257 \text{ K} + (3' + 9.83') (841 \text{ K})$$

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

$$\text{ADD } e_{l_{F_v}} = 2.1 \text{ FT} \Rightarrow e_l = 0.94' + 2.1' = 3.04'$$

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

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

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

$$F_v = 11,888 \text{ K} \quad (\text{STATIC} + \text{DYN } 8 \text{ CASKS})$$

$$EQ_{HE-W} = 257 \text{ K} + 885 \text{ K} = 1142 \text{ K}$$

F_{HP} F_{Hc}

$$EQ_{HN-S} = 257 \text{ K} + 841 \text{ K} = 1098 \text{ K}$$

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

$$\phi = 0^\circ \quad C = 2.2 \text{ KSF}$$

6

~~00880~~

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.85

CALCULATION IDENTIFICATION NUMBER				PAGE 97
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 8 CASKS

PSHA 2,000-Yr Earthquake: Case IVA

40 % N-S, 100 % Vert, 40 % 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:

$B' = 27.9$ Effective Fig 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)

$EQ_{H\ E-W} = 1,142$ k & $EQ_{H\ N-S} = 1,098$ k \rightarrow 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)

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

$$= 5.14 \quad \text{Eq 3.6 \& Table 3.2}$$

$$N_q = e^{\pi \tan \phi} \tan^2(\pi/4 + \phi/2)$$

$$= 1.00 \quad \text{Eq 3.6}$$

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

$$= 0.00 \quad \text{Eq 3.8}$$

$$s_c = 1 + (B/L)(N_q/N_c)$$

$$= 1.09 \quad \text{Table 3.2}$$

$$s_q = 1 + (B/L) \tan \phi$$

$$= 1.00$$

$$s_\gamma = 1 - 0.4 (B/L)$$

$$= 0.82$$

$$\text{For } D_f/B \leq 1: d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 D_f/B$$

$$= 1.00 \quad \text{Eq 3.26}$$

$$d_\gamma = 1$$

$$= 1.00$$

$$\text{For } \phi > 0: d_c = d_q - (1 - d_q) / (N_q \tan \phi)$$

$$= \text{N/A}$$

$$\text{For } \phi = 0: d_c = 1 + 0.4 (D_f/B)$$

$$= 1.04 \quad \text{Eq 3.27}$$

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

$$= 1.69 \quad \text{Eq 3.18a}$$

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

$$= 1.31 \quad \text{Eq 3.18b}$$

$$\text{If } EQ_{H\ N-S} > 0: \theta_n = \tan^{-1}(EQ_{H\ E-W} / EQ_{H\ N-S})$$

$$= 0.81 \quad \text{rad}$$

$$m_n = m_L \cos^2 \theta_n + m_B \sin^2 \theta_n$$

$$= 1.51 \quad \text{Eq 3.18c}$$

$$i_q = \{ 1 - F_H / [(F_v + EQ_v) + B' L' c \cot \phi] \}^m$$

$$= 1.00 \quad \text{Eq 3.14a}$$

$$i_\gamma = \{ 1 - F_H / [(F_v + EQ_v) + B' L' c \cot \phi] \}^{m+1}$$

$$= 0.00 \quad \text{Eq 3.17a}$$

$$\text{For } \phi = 0: i_c = 1 - (m F_H / B' L' c N_c)$$

$$= 0.88 \quad \text{Eq 3.16a}$$

N_c term

N_q term

N_γ term

$$\text{Gross } q_{ult} = 11,546 \text{ psf} = 11,246 + 300 + 0$$

$$q_{all} = 10,490 \text{ psf} = q_{ult} / FS$$

$$q_{actual} = 7,004 \text{ psf} = (F_v + EQ_v) / (B' \times L')$$

$$FS_{actual} = 1.65 = q_{ult} / q_{actual}$$

> 1.1 Hence OK

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 98
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

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 ($FS \geq 1.1$).

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 99
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05896.02	G(B)	04 - 9		

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

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 100
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 $\phi = 17^\circ$, 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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 101
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 $\phi = 30^\circ$ 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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 102
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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 ($FS \geq 1.1$). 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 ($FS \geq 1.1$).

STONE & WEBSTER, INC.
CALCULATION SHEET

6010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 103
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

REFERENCES:

ASCE 4-86, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety-Related Nuclear Structures*, American Society of Civil Engineers, 1986.

ACI, 1998, "State-of-the Art Report on Soil Cement," reported by ACI Committee 230, ACI 230-1R-90 (Reapproved 1997), American Concrete Institute, Detroit, MI.

Bowles, J.E., 1996, Foundation Analysis and Design, 5th edition, The McGraw Hill Companies, New York, New York.

CEC, 2001, PFSF Calculation 05996.02-G(PO17)-2, Rev. 3, "Storage Pad Analysis and Design," prepared by International Civil Engineering Consultants, Inc. for Stone and Webster Engineering Corp, Denver, CO.

ConeTec, 1999, "Cone Penetration Testing Report, Private Fuel Storage Facility," prepared for Stone and Webster Engineering Corp, Denver, CO, 2 volumes.

DeGroot, G., 1976, "Bonding Study on Layered Soil Cement", REC-ERC-76-16, U.S. Bureau of Reclamation, Denver, CO, September 1976.

GTG 6.15-1, "Geotechnical Division Technical Guideline on Determination of Lateral Pressures on Buried Structures in Granular Soils," Stone and Webster Engineering Corp, Boston, MA, 1982.

Geomatrix Consultants, Inc, 2001, "Development of Design Ground Motions for the Private Fuel Storage Facility, Skull Valley, UT," prepared for Stone and Webster Engineering Corp, Denver, CO, March.

Lambe, T.W., and R. V. Whitman, 1969, Soil Mechanics, John Wiley & Sons, Inc., N.Y., 553 pp.

NAVFAC (1986), DM 7.2, "Foundations and Earth Structures," Dept of the Navy, Naval Facilities Eng'g, Command, Alexandria, VA.

Newmark, N. M., 1965, "Effects of Earthquakes on Dams and Embankments," *Fifth Rankine Lecture*, Geotechnique, Institution of Civil Engineers, London, 15(2), pp139-60.

Nussbaum, P. J., and Colley, B. E., 1971, "Dam Construction and Facing with Soil-Cement," Portland Cement Association, Publication RD010.01W.

Seed, H. B., and Whitman, R. V., 1970, "Design of Earth Retaining Structures for Dynamic Loads," ASCE Specialty Conference on Lateral Stresses in the Ground and the Design of Earth Retaining Structures, pp 103-147.

Stone & Webster, 2000, "Storage Facility Design Criteria," Rev. 3, Stone & Webster, Inc., Denver, CO.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 104
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		
<p>SWEC, 1982, Geotechnical Technical Guideline 6.15-1, "Determination of Lateral Pressures on Buried Structures in Granular Soils," Stone & Webster Engineering Corporation, Boston, MA.</p> <p>SWEC, 2000a, Calculation No. 05996.02-G(B)-5, Revision 2, "Document Bases for Geotechnical Parameters Provided in Geotechnical Design Criteria," Stone & Webster Engineering Corporation, Boston, MA.</p> <p>SWEC, 2001b, Calculation No. 05996.02-G(B)-13, Revision 4, "Stability Analyses of the Canister Transfer Building Supported on a Mat Foundation," Stone & Webster, Inc., Stoughton, MA.</p> <p>Terzaghi, K., and Peck, R. B., 1967, <u>Soil Mechanics in Engineering Practice</u>, John Wiley & Sons, New York, NY, p 112.</p> <p>Vesic, A. S., 1973, "Analysis of Ultimate Loads on Shallow Foundations," <i>Journal of the Soil Mechanics and Foundations Division</i>, ASCE, Vol 99, No. SM1, pp 45-73.</p> <p>Winterkorn, H. F., and Fang, H., 1975, <u>Foundation Engineering Handbook</u>, Van Nostrand Reinhold Co., New York, NY.,</p>				

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

OPTIONAL TASK CODE

PAGE 105

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 EG	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EG	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 EG	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EG	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 EG	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EG	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

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

OPTIONAL TASK CODE

PAGE 106

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

Case	F_v k	EQ_{HNS} k	EQ_{HEW} k	$\Sigma M_{\theta NS}$ ft-k	$\Sigma M_{\theta EW}$ ft-k	β_B EQ_{HEW} deg	β_L EQ_{HNS} deg	GROSS		e_B ft	e_L ft	EFFECTIVE			FS_{actual}
								q_{ult} ksf	q_{all} ksf			B' ft	L' ft	q_{actual} ksf	
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), $\phi=0$. $\gamma = 80$ 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. $F_H = EQ_{HEW}$ or EQ_{HNS} $\beta_B = \tan^{-1} [(EQ_{HEW}) / F_v]$ = Angle of load inclination from vertical (deg) as f($\beta_L = \tan^{-1} [(EQ_{HNS}) / F_v]$ = Angle of load inclination from vertical (deg) as f(l $e_B = \Sigma M_{\theta NS} / F_v$ $e_L = \Sigma M_{\theta EW} / F_v$ $B' = B - 2 e_B$ $L' = L - 2 e_L$ $q_{actual} = F_v / (B' \times L')$

CALCULATION SHEET

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

OPTIONAL TASK CODE

PAGE 107

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 k	EQ _{HNS} k	EQ _{HEW} k	ΣM _{ONS} ft-k	ΣM _{OE-W} ft-k	β _B EQ _{HEW} deg	β _L EQ _{HNS} deg	GROSS		e _B ft	e _L ft	EFFECTIVE			FS _{actual}
								q _{ult} ksf	q _{all} ksf			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)

F_V = Vertical load (F_{V Static} + EQ_V)0.711 g = a_H

φ = 0.0 Friction angle (deg)

EQ_H = Earthquake: Horizontal force. F_H = SQRT[EQ_{H E-W}² + EQ_{H N-S}²]0.695 g = a_V

B = 30 Footing width (ft)

β_B = tan⁻¹ [(EQ_{HEW}) / F_V] = Angle of load inclination from vertical (deg) as f(width).

L = 67 Footing length (ft)

β_L = tan⁻¹ [(EQ_{HNS}) / F_V] = Angle of load inclination from vertical (deg) as f(length).D_f = 3.0 Depth of footing (ft)e_B = ΣM_{ONS} / F_Ve_L = ΣM_{OE-W} / F_V

γ = 80 Unit weight of soil (pcf)

B' = B - 2 e_BL' = L - 2 e_Lγ_{surch} = 100 Unit weight of surcharge (pcf)q_{actual} = F_V / (B' x L')

FS = 1.1 Factor of safety for dynamic loads.

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

OPTIONAL TASK CODE

PAGE 108

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_{HNS} k	EQ_{HE-W} k	$\Sigma M_{\theta NS}$ ft-k	$\Sigma M_{\theta E-W}$ ft-k	β_B EQ_{HE-W} deg	β_L EQ_{HNS} deg	GROSS		e_B ft	e_L ft	EFFECTIVE			FS_{actual}
								q_{vr} ksf	q_{ml} ksf			B' ft	L' ft	q_{actual} ksf	
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) $L = \text{Varies}$ 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. $F_H = EQ_{HE-W}$ or EQ_{HNS} $\beta_B = \tan^{-1} [(EQ_{HE-W}) / F_v] =$ Angle of load inclination from vertical (deg) as $f(\text{width})$. $\beta_L = \tan^{-1} [(EQ_{HNS}) / F_v] =$ Angle of load inclination from vertical (deg) as $f(\text{length})$. $\Sigma M_{\theta NS} = \theta_B \times F_v$ $\Sigma M_{\theta E-W} = \theta_L \times F_v$ $B' = B - 2 \theta_B$ $L' = L - 2 \theta_L$ $q_{actual} = F_v / (B' \times L')$

FIGURE 1
FOUNDATION PLAN & PROFILE

PROFILE A-A

19.27'

30'

SOIL CEMENT

CRUSHED STONE

LAYER 1
SILT, SILTY CLAY,
& CLAYEY SILT

LAYER 2
V. DENSE
FINE SAND

LAYER 3
NON-PLASTIC SILT

DEPTH, FT.

0

10

20

30

40

50

60

70

80

90

100

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

PLAN

30'

67'

SEE DETAIL
ON NEXT
PAGE
ON FIGURE 3

Y-DIRECTION
IS VERTICAL

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

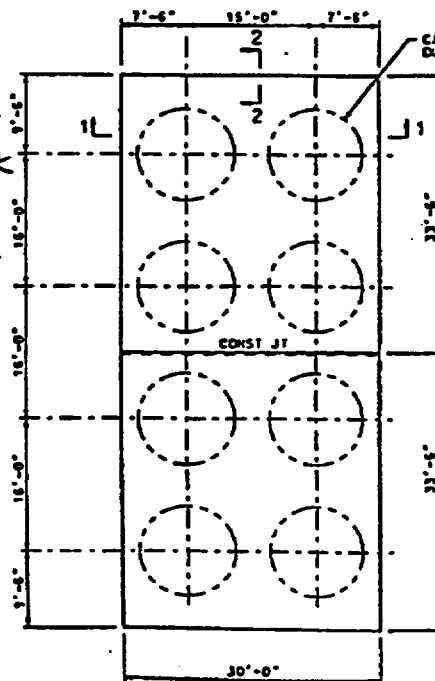
STONE & WEBSTER, INC.
CALCULATION SHEET

5010.55

CALCULATION IDENTIFICATION NUMBER				PAGE 110
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

FIGURE 2
STATIC FOUNDATION LOAD / PRESSURE

Total Load
 $8 \text{ casks} @ 356.5^k = 2852^k$
 $30' \times 67' \times 3' \times 0.15 \frac{k}{ft^3} = 904.5^k$
 $\therefore \text{Total Load} = 3756.5^k$



Bearing Pressure =
 $P_{actual} = \text{Load} / \text{Area}$
 $P_{actual} = 3756.5 / 30' \times 67'$
 $P_{actual} = 1.87 \text{ KSF}$

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.

STONE & WEBSTER, INC.
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 111
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

FIGURE 3

DETAIL OF SOIL CEMENT UNDER &
ADJACENT TO CASK STORAGE PADS

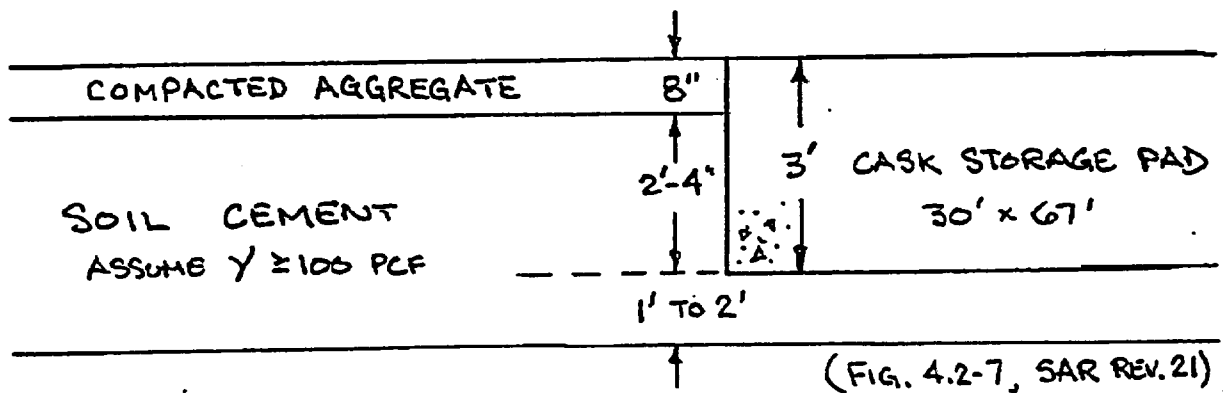
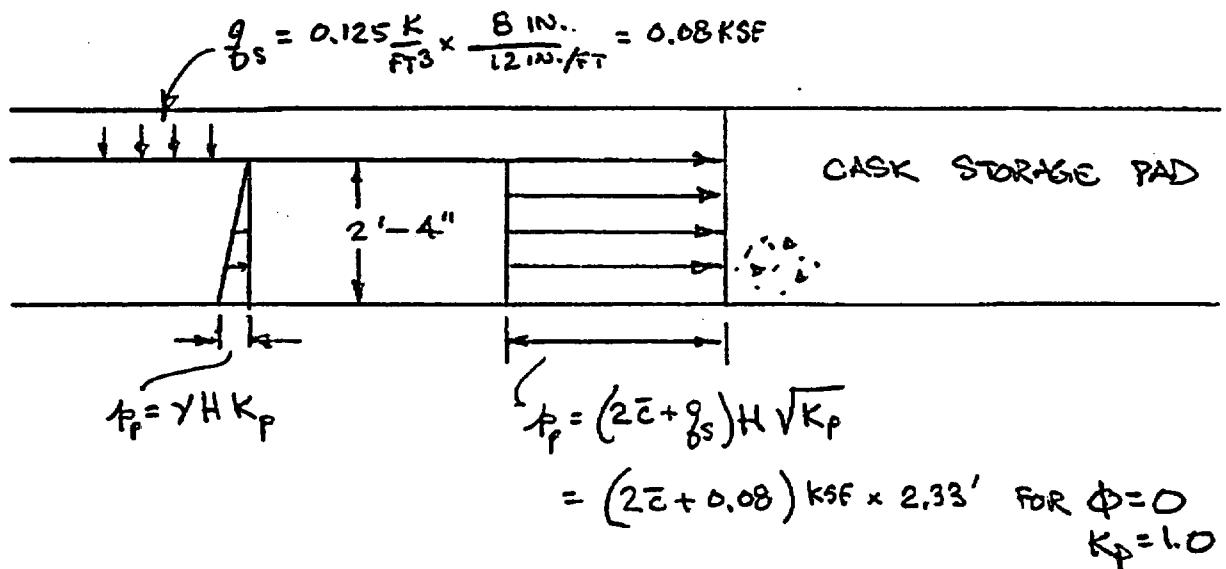


FIGURE 4

PASSIVE PRESSURE ACTING ON CASK STORAGE PADS

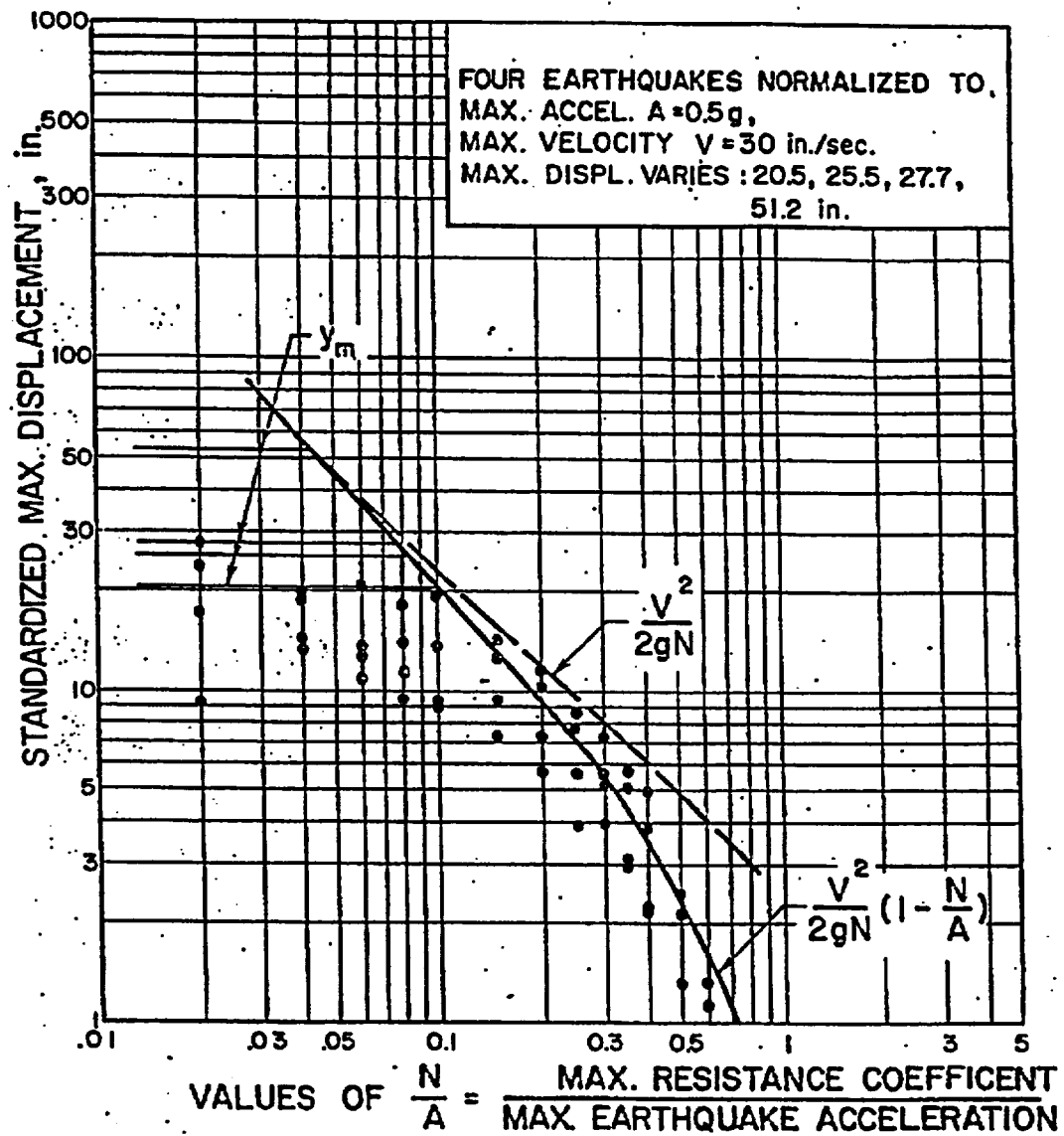


STONE & WEBSTER, INC.
CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 112
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

FIGURE 5

STANDARDIZED DISPLACEMENT FOR NORMALIZED EARTHQUAKES
(SYMMETRICAL RESISTANCE)



From Newmark (1965)

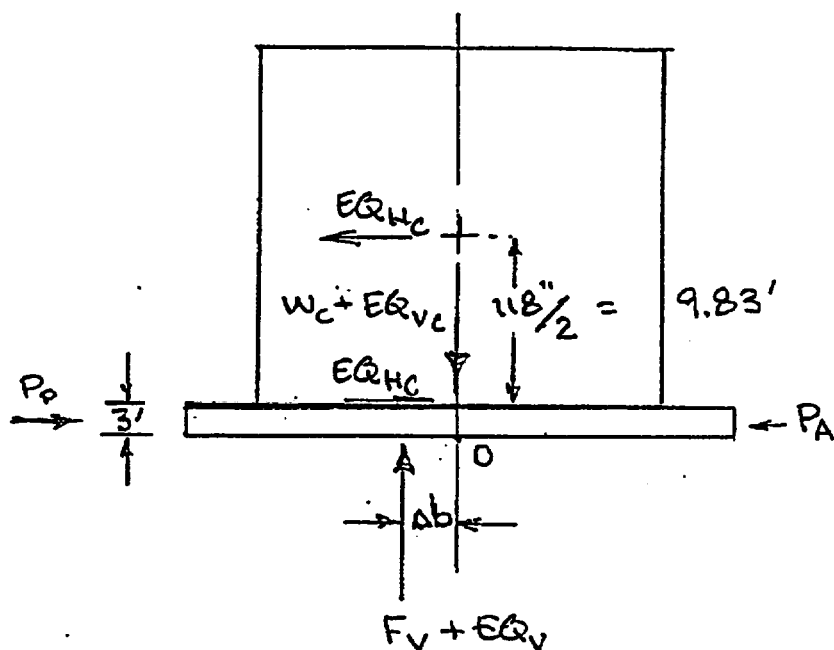
CALCULATION SHEET

8010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 113
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

FIGURE 6

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



$P_A \ll P_P$; 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.

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

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

$$\sum M_{\text{c.o.}} \text{ to find } \sum M_{\text{c.N-S}}$$

$$\sum M_{\text{c.N-S}} = \underset{\text{pad}}{1.5 \text{ ft} \times EQ_{HP}} + \underset{\text{cask horiz}}{3 \text{ ft} \times EQ_{Hc}} + \underset{\text{cask vert}}{\Delta b \times (W_c + EQ_{Vc})}.$$

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.

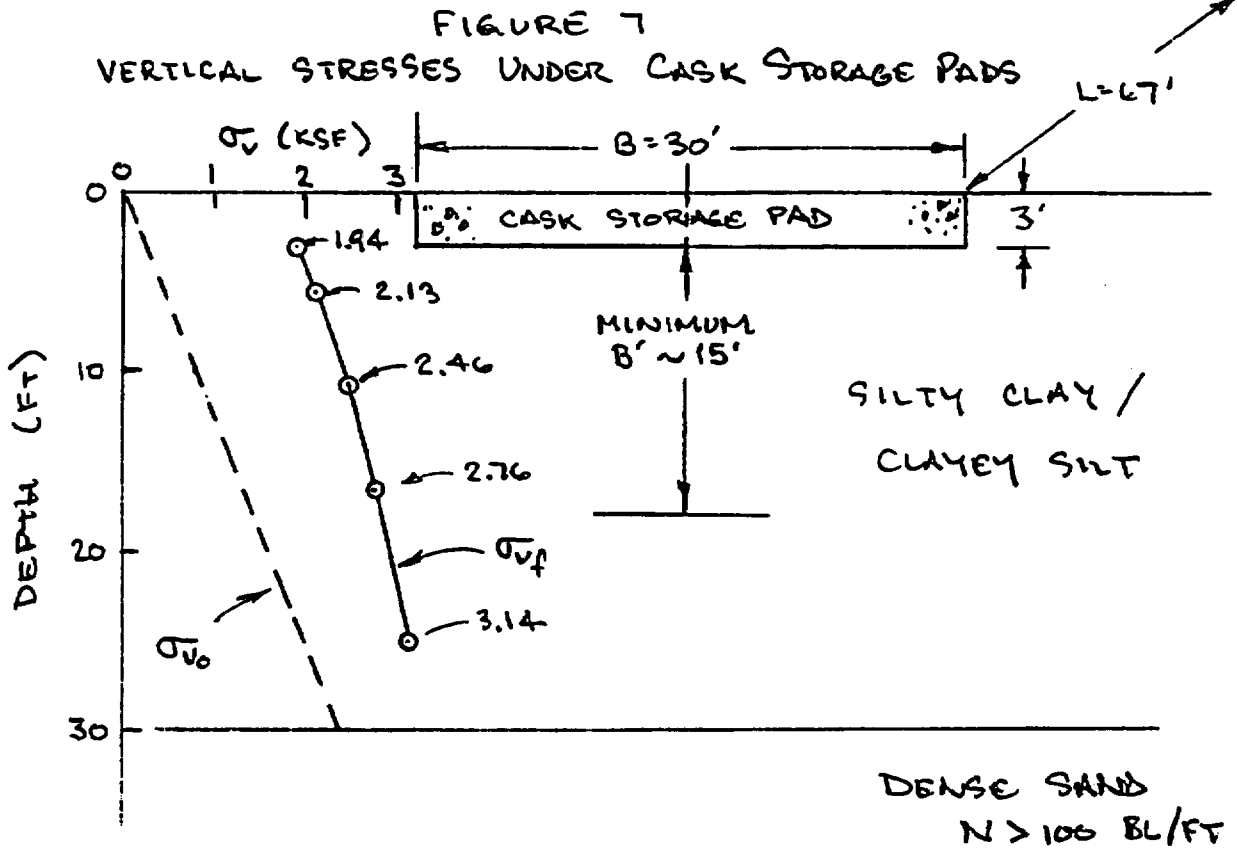
1ED JUN 12 2000

P.J.
Tucker

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 114
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		



NOTE: AVG $\gamma_m \sim 80$ 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 $\Rightarrow \sigma_{vf}$ AS $f(z)$

FIG 1 $\Rightarrow \sigma_v$ AT BOTTOM OF 3' PAD = 1.935 KSF

FIG 7 \Rightarrow SUBLAYERS USED IN DET'G σ_v AS $f(z)$

STONE & WEBSTER, INC.
CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER			
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-9	OPTIONAL TASK CODE
			PAGE <u>115</u>

FIGURE 8

ELEVATION VIEW OF COLUMN OF CASK STORAGE PADS - LOOKING EAST

NOTES OF TELEPHONE CONVERSATION

JO No. 05996.01

**PRIVATE FUEL STORAGE, LLC
PRIVATE FUEL STORAGE FACILITY**Date: 06-19-97
Time: 2:45 PM EDTFROM: Stan M. Macie SWEC-Denver 1E
Wen Tseng (ICEC)Tie Line 321-7305
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.

ACTION ITEMS:SUPERSEDED
BY ATT B 

PJTrudeau to determine the dynamic allowable bearing pressure for the 2-cask loading case.

COPY TO: NTGeorges Boston 245/03
SMMacie Denver 1E~~00001~~



CALCULATION SHEET

ORIGINATOR
PROJECT
SUBJECT

W
Private Fuel Storage Facility
Storage Pad Analysis and Design

DATE 3/27/01

CALC. NO. G(PO17)-2
CHECKED mas

REV. NO. 3
DATE 4-5-01
JOB NO. 1101-000
SHEET 227

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.



CALCULATION SHEET

ORIGINATOR
PROJECT
SUBJECT

W DATE 3/27/01
Private Fuel Storage Facility
Storage Pad Analysis and Design

CALC. NO. G(PO17)-2
CHECKED 22617

REV. NO. 3
DATE 4-5-01
JOB NO. 1101-000
SHEET 228

Table S-1
Maximum Vertical Displacements and Soil Bearing Pressures
Dead Load

	$k_s = 2.75 \text{ kcf}$	$k_s = 26.2 \text{ kcf}$
$Z_w(\text{ft}) =$	0.164	0.017
$q_{zw}(\text{ksf}) =$	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 = $k_s \times Z_w$, where k_s = subgrade modulus = 2.75 and 26.2 kcf for lower-bound and upper-bound soils, respectively.



CALCULATION SHEET

ORIGINATOR

KJ

DATE 3/27/01

CALC. NO. G(PO17)-2

REV. NO. 3

PROJECT

Private Fuel Storage Facility

CHECKED

[Signature]

DATE 4-5-01

SUBJECT

Storage Pad Analysis and Design

JOB NO. 1101-000

SHEET 229

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 _z ⁽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	-1.678	-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).



CALCULATION SHEET

ORIGINATOR	<u>W</u>	DATE	<u>3/28/01</u>	CALC. NO.	<u>G(PO17)-2</u>	REV. NO.	<u>3</u>
PROJECT	<u>Private Fuel Storage Facility</u>			CHECKED	<u>MSJ</u>	DATE	<u>4-5-01</u>
SUBJECT	<u>Storage Pad Analysis and Design</u>					JOB NO.	<u>1101-000</u>
						SHEET	<u>230</u>

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.



CALCULATION SHEET

ORIGINATOR
PROJECT
SUBJECT

DATE 3/28/01
Private Fuel Storage Facility
Storage Pad Analysis and Design

CALC. NO. G(PO17)-2
CHECKED 212

REV. NO. 3
DATE 4-5-01
JOB NO. 1101-000
SHEET 231

Table D-1(a)
Averaged Maximum Horizontal Soil Reactions in the X Direction
Dynamic Load

Node No.	Maximum Displacement X_d ($\times 10^{-3}$ 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
K _{xd} =	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
Q _{xd} =	789	1277	1982	764	1159	2105	943	1494	2212

Notes:

1. Avg = $\{\sum (X_d)\}/N$; X_d = max. x-displ.; i = nodes 1, 7, 13, 144, 150, 156, 287, 293, 299; and $N = 9$.
2. $Q_{xd} = K_{xd} \times \text{Avg}$ = averaged maximum horizontal-x soil reaction in Kips due to dynamic loading.
3. K_{xd} for LB, BE, and UB soils are dynamic horizontal-x soil spring stiffnesses given below:

$$\begin{array}{lll}
 (K_{xd})_{LB} = & 9.51E+06 \text{ lb/in} & (K_{xd})_{BE} = 1.94E+07 \text{ lb/in} & (K_{xd})_{UB} = 4.57E+07 \text{ lb/in} \\
 & 1.14E+05 \text{ Kips/ft} & 2.33E+05 \text{ Kips/ft} & 5.48E+05 \text{ Kips/ft}
 \end{array}$$

4. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.
5. X_d are obtained from CECSAP analysis results given in Att. A.



CALCULATION SHEET

ORIGINATOR
PROJECT
SUBJECT

Private Fuel Storage Facility
Storage Pad Analysis and Design

DATE 3/28/01

CALC. NO. G(PO17)-2
CHECKED msf

REV. NO. 3
DATE 4-5-01
JOB NO. 1101-000
SHEET 232

Table D-1(b)
Averaged Maximum Horizontal Soil Reactions in the Y Direction
Dynamic Load

Node No.	Max. Displacement Yd ($\times 10^{-3}$ 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:

$$\begin{array}{lll}
 (Kyd)_{LB} = 9.04E+06 \text{ lb/in} & (Kyd)_{BE} = 1.84E+07 \text{ lb/in} & (Kyd)_{UB} = 4.34E+07 \text{ lb/in} \\
 1.08E+05 \text{ Kips/ft} & 2.21E+05 \text{ Kips/ft} & 5.21E+05 \text{ Kips/ft}
 \end{array}$$

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.



CALCULATION SHEET

ORIGINATOR
PROJECT
SUBJECT

W DATE 3/28/01
Private Fuel Storage Facility
Storage Pad Analysis and Design

CALC. NO. G(PO17)-2

CHECKED mg

REV. NO. 3

DATE 4-5-01

JOB NO. 1101-000

SHEET 273

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	845	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.



CALCULATION SHEET

ORIGINATOR
PROJECT
SUBJECT

Private Fuel Storage Facility
Storage Pad Analysis and Design

DATE 3/28/01

CALC. NO. G(PO17)-2
CHECKED mm

REV. NO. 3
DATE 4-5-01
JOB NO. 1101-000
SHEET 234

Table D-1(d)
Maximum Vertical Soil Bearing Pressures
Dynamic Load

Node No.	Maximum Displacement Z_d ($\times 10^{-3}$ 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	8.396	-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	-22.56	-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
Node No.	Maximum Soil Bearing Pressure q_{zd} (Kips/ft ²)								
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
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:

- q_{zd} = maximum soil bearing pressure = $(K_{zd} \times Z_d)/A$, where $A = 67' \times 30' = 2010 \text{ ft}^2$.
- K_{zd} for LB, BE, and UB soils are vertical-z dynamic soil spring stiffnesses given below:

$$\begin{array}{lll}
 (K_{zd})_{LB} = 1.20\text{E}+07 \text{ lb/in} & (K_{zd})_{BE} = 2.37\text{E}+07 \text{ lb/in} & (K_{zd})_{UB} = 5.41\text{E}+07 \text{ lb/in} \\
 1.44\text{E}+05 \text{ Kips/ft} & 2.84\text{E}+05 \text{ Kips/ft} & 6.49\text{E}+05 \text{ Kips/ft}
 \end{array}$$

- LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.
- Z_d are obtained from CECSAP analysis results given in Att. A.
- Negative displacements imply downward movements.
- The maximum values of Z_d shown may not be concurrent. However, they are assumed to be concurrent values and concurrent signs are assigned to them.
- Node numbers are shown in Figure 5.1-1.



ATTACHMENT B TO CALC 05996.02-G(B)-04-09 PAGE 39
CALCULATION SHEET

ORIGINATOR
PROJECT
SUBJECT

W
Private Fuel Storage Facility
Storage Pad Analysis and Design

DATE 3/27/01

CALC. NO. G(PO17)-2
CHECKED W

REV. NO. 3
DATE 4-5-01
JOB NO. 1101-000
SHEET 225

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).



CALCULATION SHEET

ORIGINATOR
PROJECT
SUBJECT

DATE 3/27/01
Private Fuel Storage Facility
Storage Pad Analysis and Design

CALC. NO. G(PO17)-2
CHECKED [Signature]

REV. NO. 3
DATE 4-05-01
JOB NO. 1101-000
SHEET 276

Table 4
Summary of Vertical Soil Bearing Pressures (ksf)

		A	B	C	D	E	F	G	H	J
Loading	Point	287	293	299	144	150	156	1	7	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.) α_v , where pad wt.=904.5 kips, and α_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.



CALCULATION SHEET

ORIGINATOR

✓

DATE 3/14/01

CALC. NO. G(PO17)-2

REV. NO. 3

PROJECT

Private Fuel Storage Facility

CHECKED *Alm*

DATE 3/30/01

SUBJECT

Storage Pad Analysis and Design

JOB NO. 1101-000

SHEET NO. 20

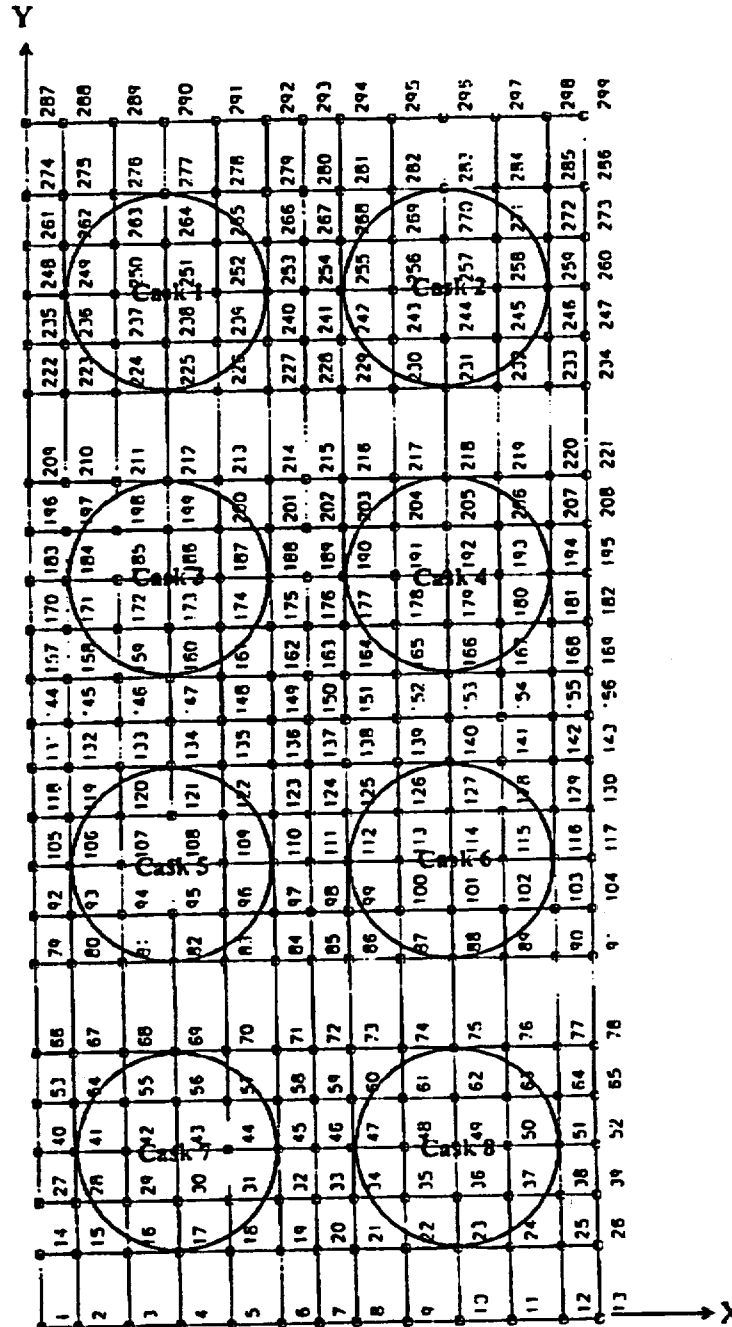


Figure 5.1-1 CECSAP Finite-Element Model with Node Numbers



ORIGINATOR ✓ DATE 3/14/01
PROJECT Private Fuel Storage Facility
SUBJECT Storage Pad Analysis and Design

CALC. NO. G(PO17)-2 REV. NO. 3
CHECKED AMM DATE 3/30/01
JOB NO. 1101-000
SHEET NO. 21

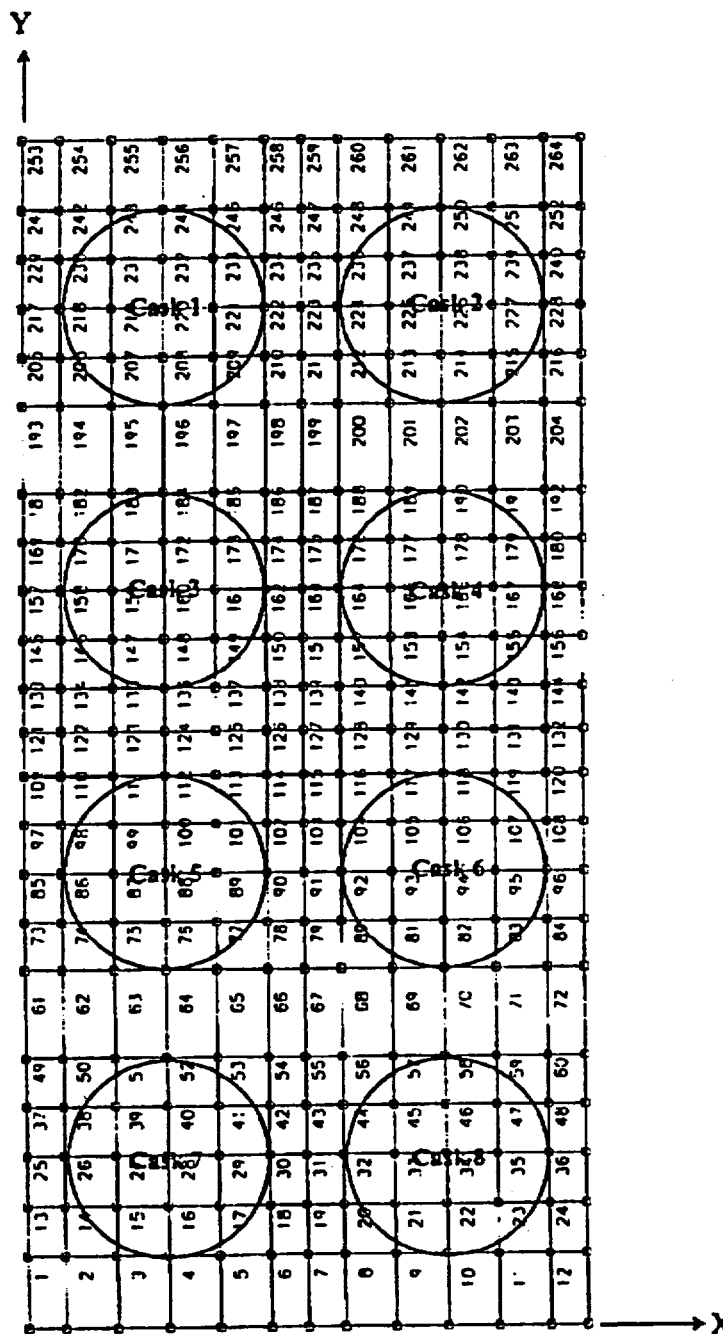


Figure 5.1-2 CECSAP Finite-Element Model with Element Numbers

CALCULATION COVER SHEET



PROJECT Private Fuel Storage Facility (PFSF)
 SUBJECT Storage Pad Analysis and Design

JOB NO. 1101-000
 FILE NO. _____
 CALC NO. G(PO17)-2
 NO. OF SHEETS 289

RECORD OF ISSUES

NO.	DESCRIPTION	BY	DATE	CHKD	DATE	APPRD	DATE
0	Initial Issue	DH	10/18/99	DH	10/18/99	HT	10/18/99
1	Revision 1 (see notes below)	DH	12/6/99	DH	12/6/99	HT	12/6/99
2	Revision 2 (see notes below)	DH	2/4/00	DH	2/4/00	HT	2/4/00
3	Revision 3 (see notes on Sheet 11)	DH	4/5/01	DH	4/5/01	HT	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.

NAME	INITIAL	SIGNATURE
Anwar Mirza (Preparer/Checker)	anm	Anwar Mirza
Donald Hamasaki (Preparer/Checker)	DH	Donald Hamasaki
Ming S. Yang (Preparer/Checker)	my	Ming S. Yang
Kiat Lilhanand (Preparer/Checker)	KL	K. Lilhanand
Wen S. Tseng (Independent Reviewer)	HT	W. S. Tseng

CALCULATION COVER SHEET



PROJECT Private Fuel Storage Facility (PFSF)
SUBJECT Storage Pad Analysis and Design

JOB NO. 1101-000
FILE NO. _____
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.

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

A531065

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.
05996.02

DIVISION & GROUP
GCB

CALCULATION NO.
05-2

OPTIONAL TASK CODE

PAGE 25K

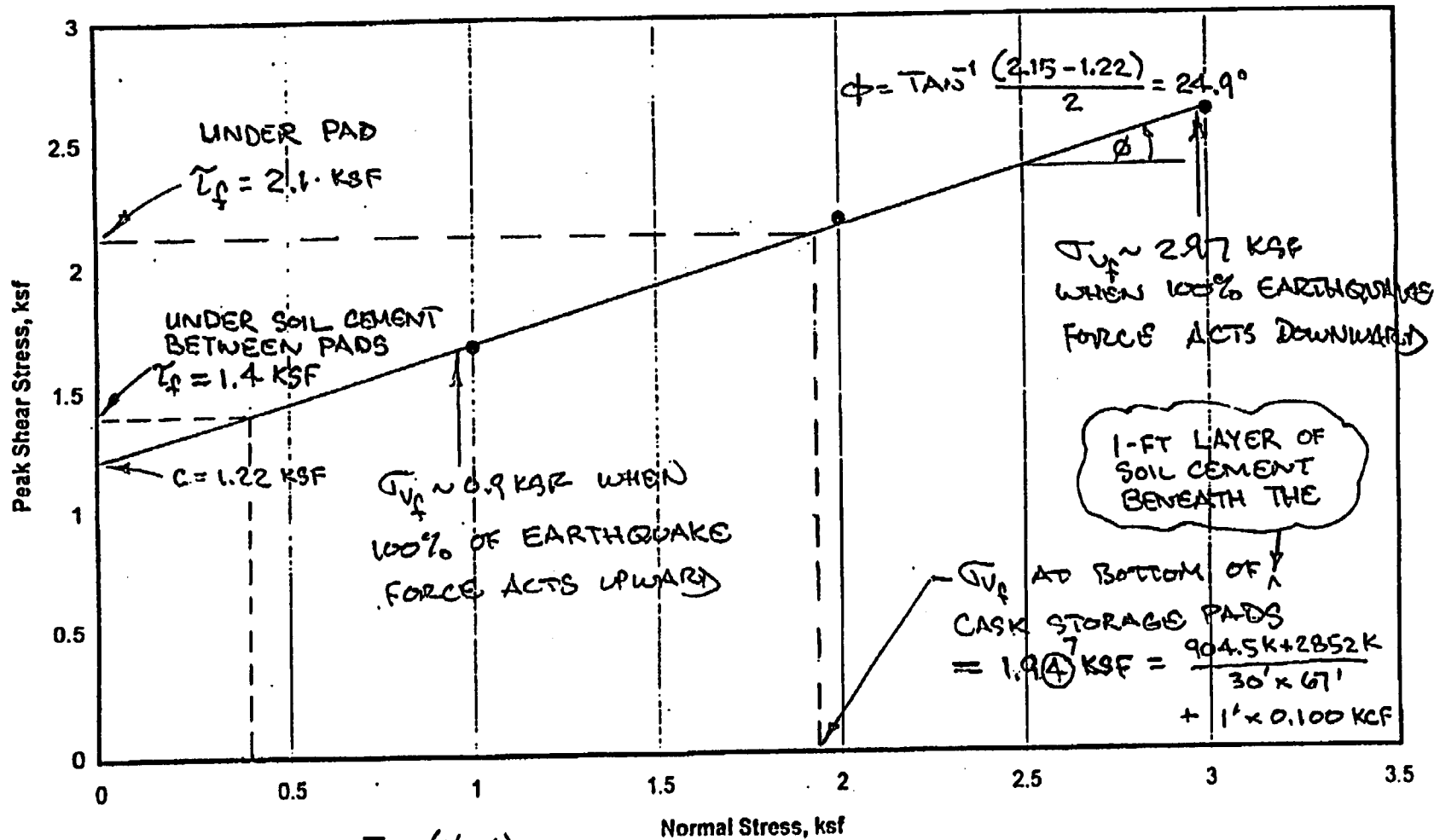
TABLE 6
SUMMARY OF TRIAXIAL TEST RESULTS FOR SOILS WITHIN ~10 FT
OF GROUND SURFACE AT THE SITE

Boring	Sample	Depth ft	Elev ft	w %	ATTERBERG LIMITS			USC Code	γ_m pcf	γ_d pcf	c_u	σ_v ksf	s_u ksf	ϵ_v %	Type	Date
					LL	PL	PI									
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-GCB)-04-9

FIGURE 7
DIRECT SHEAR TEST
Boring C-2, Sample U-1C
PAD EMPLACEMENT AREA



BETWEEN PADS $\bar{\tau}_v \sim (3' + 1') 0.100 \text{ KCF}$
 $= 0.4 \text{ KSF}$

REF SAR APP 2A ATT 7

ATTACHMENT C p C2
CALC 05996.02-G(B)-04-9

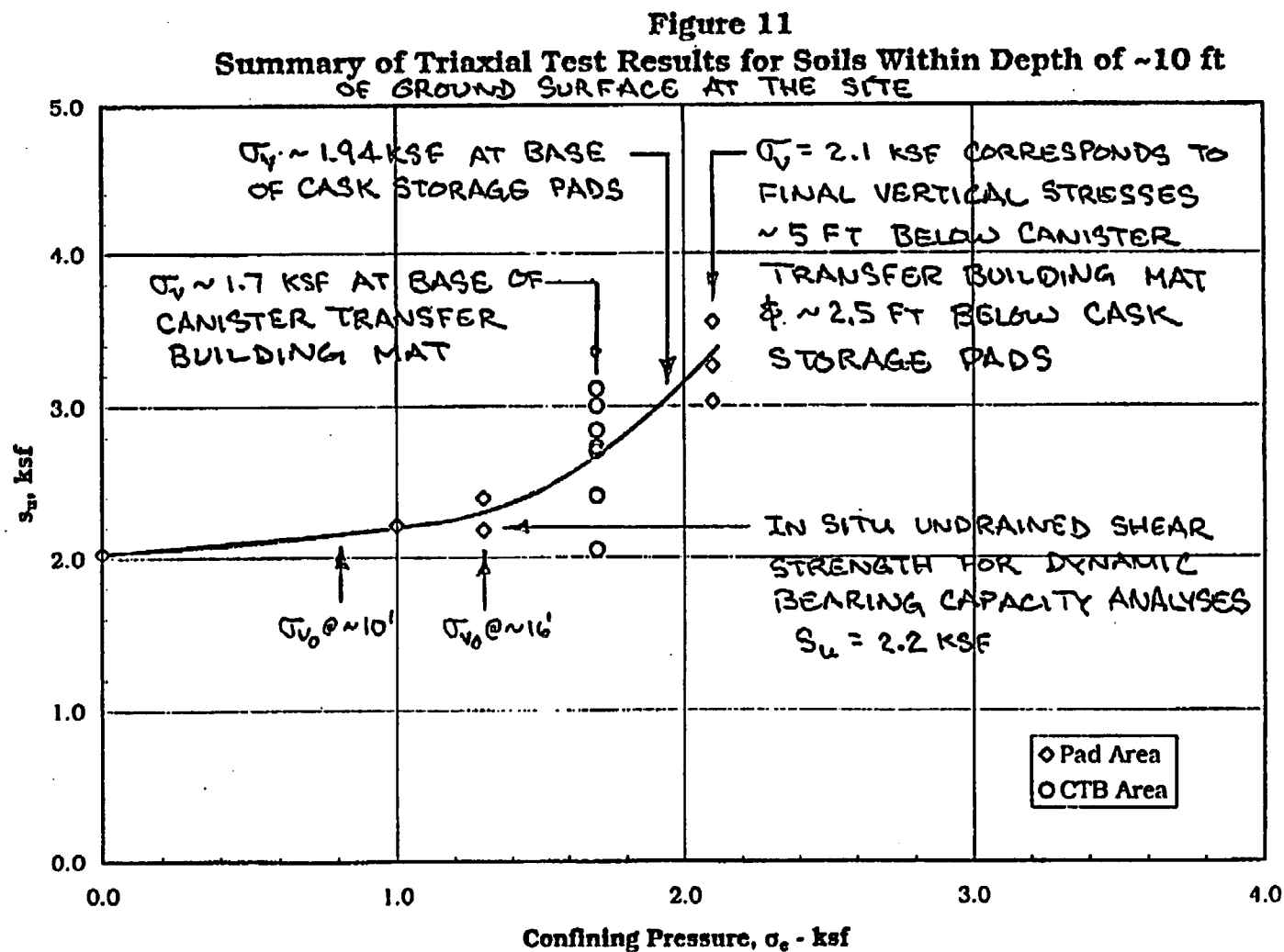
CALC 05996.02-G(B)-05-X

p 32

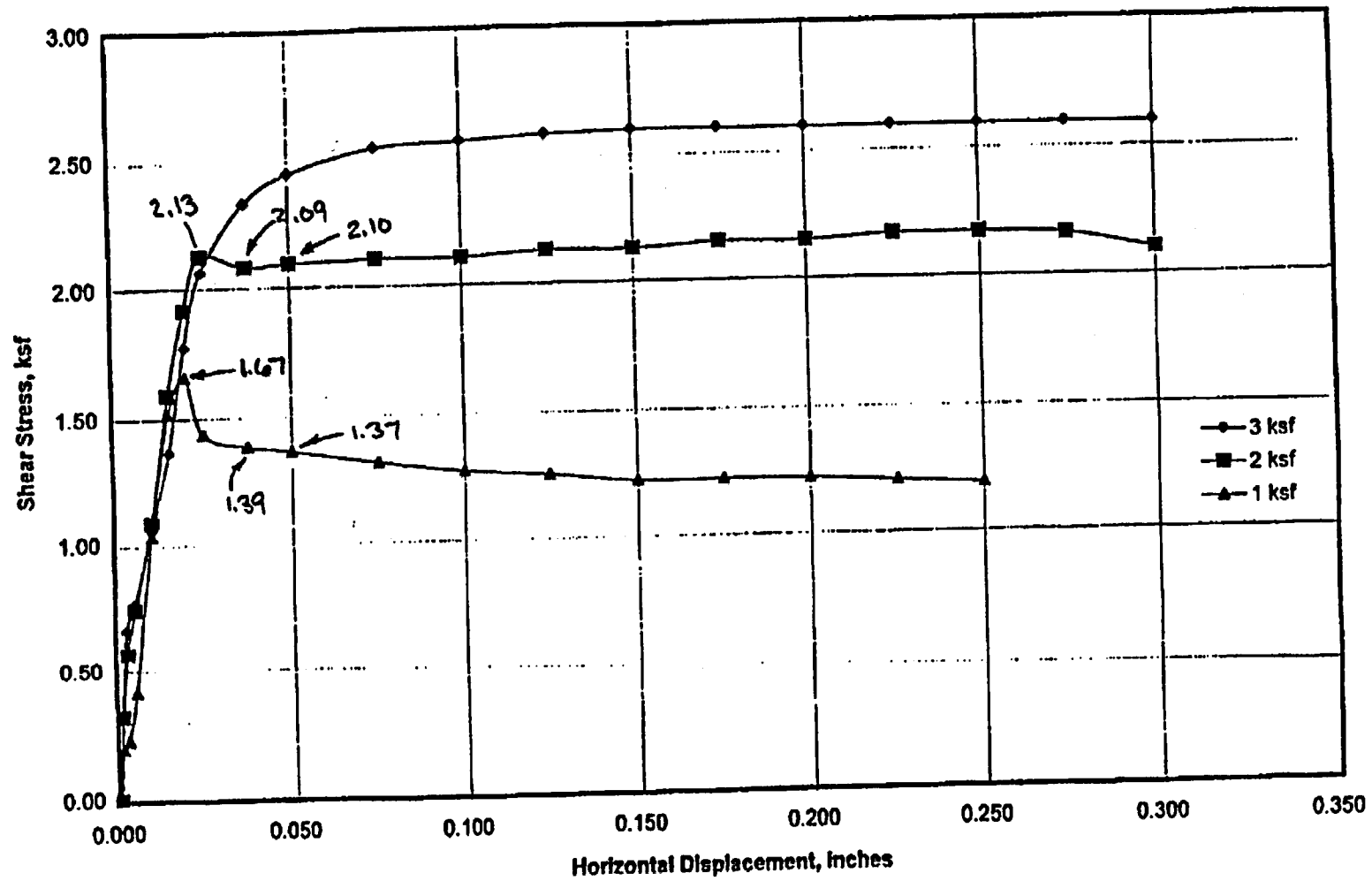
STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

A5010.65

J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION IDENTIFICATION NUMBER	CALCULATION NO. 05-2	OPTIONAL TASK CODE	PAGE 36
------------------------------	--------------------------	-----------------------------------	-------------------------	--------------------	------------

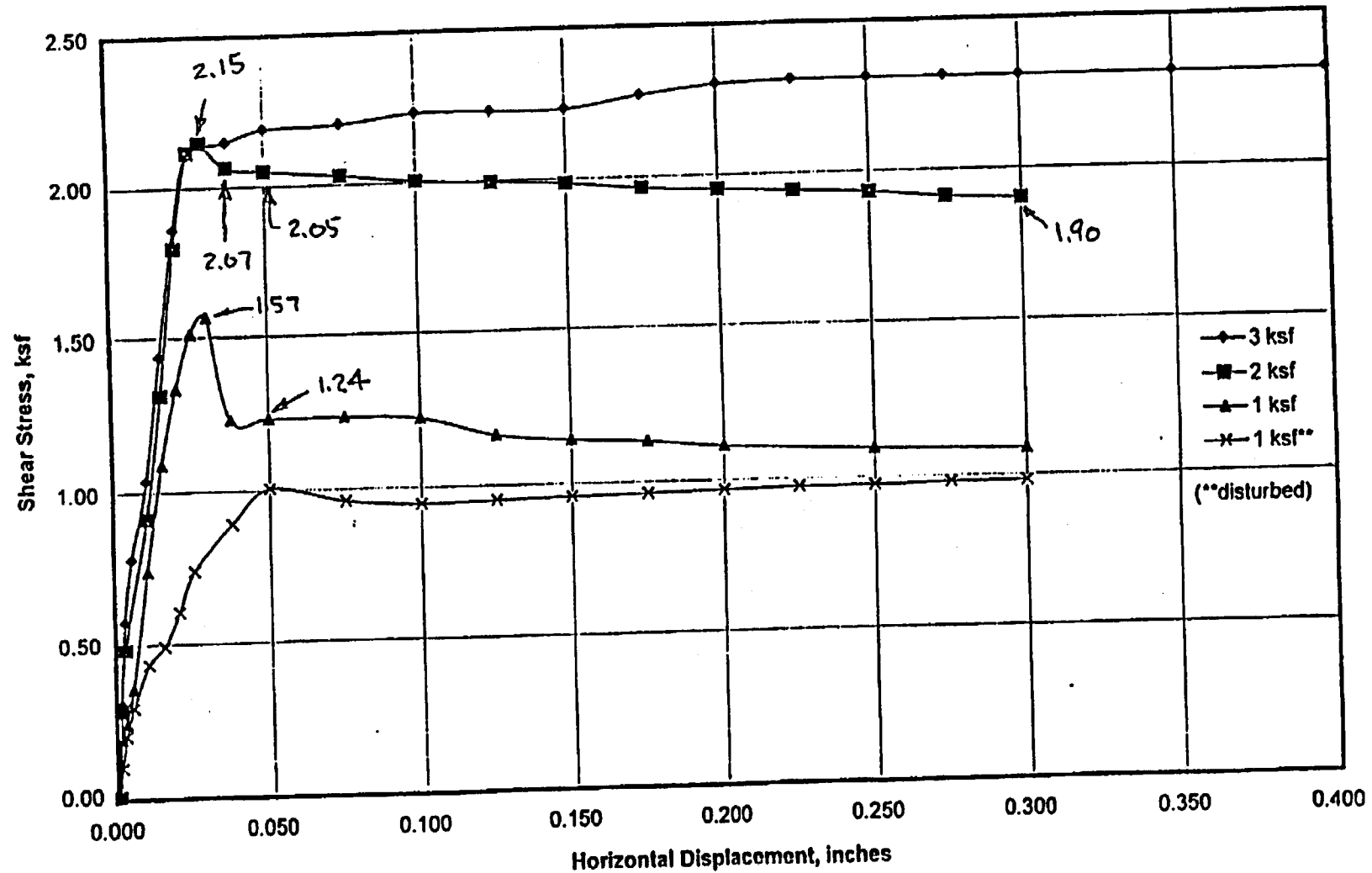


DIRECT SHEAR TEST
Boring C-2, Sample U-1C



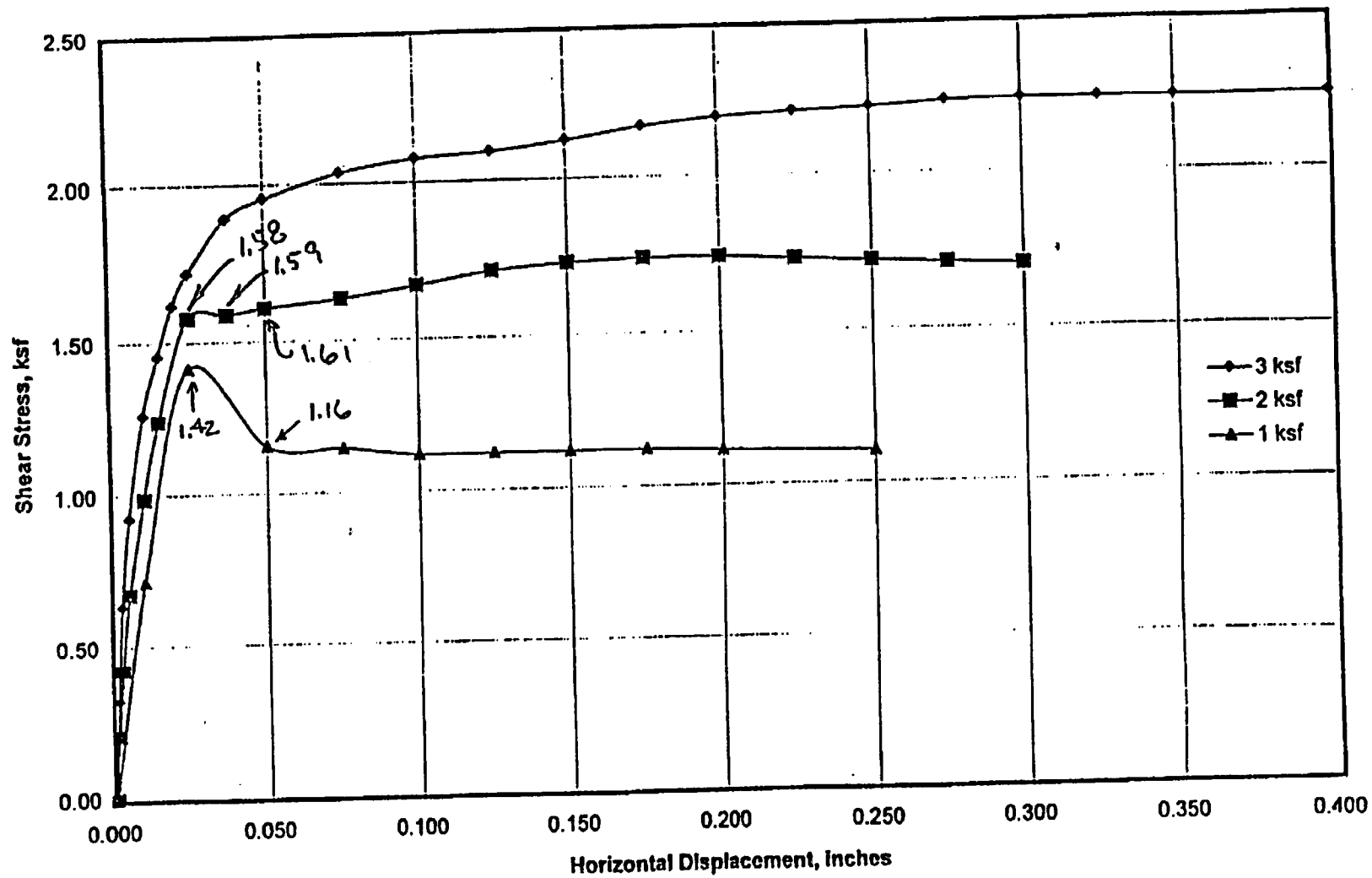
ATTACHMENT D TO CALL 05996.02-G(8)-4-9 p D1/3

DIRECT SHEAR TEST Boring CTB-6, Sample U-3B&C



ATTACHMENT D TO CALC 05996.02-G(B)-04-9 p D2

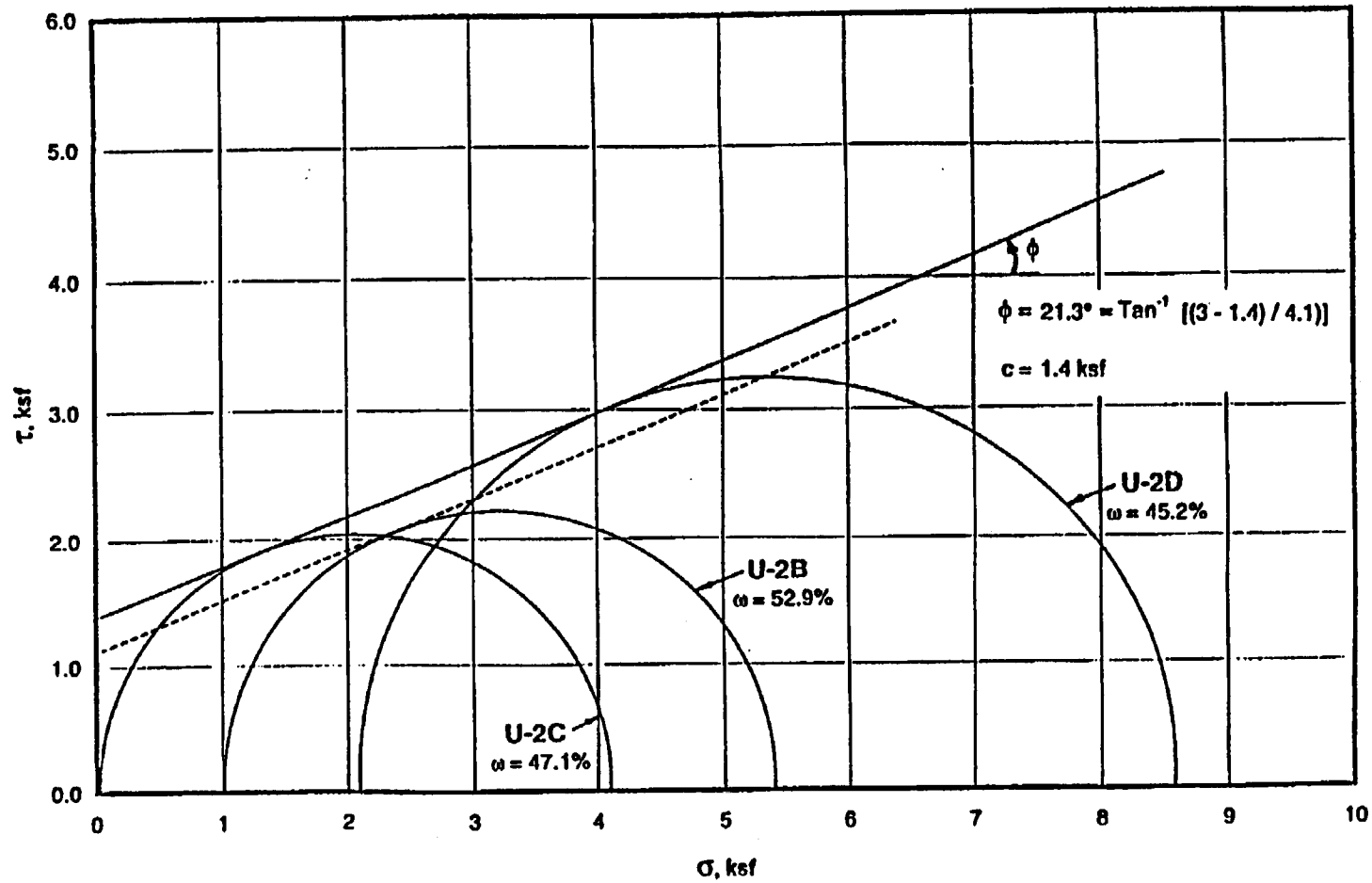
DIRECT SHEAR TEST
Boring CTB-S, Sample U-1AA



ATTACHMENT D TO CALC 05996.02-G(B)-04-9 p D3

STONE & WEBSTER ENGINEERING CORPORATION

Total Stress Mohr's Circles
Boring B-1, Sample U-2



Private Fuel Storage, LLC
PFSF, Skull Valley, UT
SAR APP 2A ATT 8

ATTACHMENT E TO CALC p E1 of 1
05996.02-G(B)-04-9

JO 05996.02
November 1999