Private Fuel Storage, L.L.C.

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U.S. Nuclear Regulatory Commission
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June 19, 2000

SUBMITTAL OF COMMITMENT RESOLUTION LETTER #34 INFORMATION
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: PFS Letter, Donnell to U.S. Nuclear Regulatory Commission, Commitment Resolution Letter # 34, dated June 2, 2000

In the referenced letter, Private Fuel Storage (PFS) committed to provide the NRC with information on tipover of a cask transporter, propane vapor cloud dispersion, and a revised calculation package associated with bearing capacity and sliding stability of the cask storage pads and the Canister Transfer Building. This letter provides the informational commitments and the calculation package.

Attachment 1 contains the calculation package that addresses bearing capacity and sliding stability analyses of the cask storage pads and the Canister Transfer Building. The package consists of the following three calculations which have been revised to address issues discussed in the referenced letter:

PFSF Calculation No. 05996.02-G(B)-4, Stability Analysis of Storage Pad, Rev. 6, Stone & Webster.

PFSF Calculation No. 05996.02-G(B)-5, Bases for Geotechnical Parameters Provided in Geotechnical Design Criteria, Rev. 2, Stone & Webster.

PFSF Calculation No. 05996.02-G(B)-13, Stability Analyses of the Canister Transfer Building Supported on a Mat Foundation, Rev. 3, Stone & Webster.

NMSS01 Public

Attachment 2 provides the results of an evaluation of the stability of the cask transporter when carrying a storage cask, assuming it is subjected to the PFSF design basis ground motion, or to the design tornado-driven missile. The evaluation concludes that the cask transporter and the storage cask will remain upright and not tip over when subjected to these events.

Attachment 3 contains the results of analyses of postulated propane releases from the relatively large propane storage tank(s) that will be located a minimum distance of 1,800 ft south or southwest of the Canister Transfer Building, considering dispersion and delayed ignition. The analyses assessed several different postulated propane leakage scenarios, including rupture of a 20,000 gallon propane tank, rupture of a 5,000 gallon propane tank, severance of a 2 inch vapor line at the tank, and severance of a 2 inch liquid line at the tank. As discussed in Section 8.2.4 of the PFSF SAR, propane vapor will be supplied from the storage tank(s) to the Canister Transfer Building and Security and Health Physics Building, using a compressor to provide the motive force. Based on building heating requirements, a 2 inch line is adequate for this purpose. Analysis of a 2 inch propane liquid line rupture was included for completeness, but liquid propane will not be supplied from the tank(s). It was assumed that variable winds were directed towards the Canister Transfer Building and cask storage area under stable atmospheric conditions (atmospheric stability class F), to minimize dispersion of the propane vapor in the plumes. In the analyses of plume formation for the postulated 2 inch line ruptures, wind speeds were varied between 1 to 5 meters per second to determine the wind speed that resulted in a concentration of gas at the lower explosive limit (LEL) approaching nearest to the Canister Transfer Building and cask storage area. A wind speed of 3 meters per second, combined with atmospheric stability class F, maximized this explosive concentration travel distance and was considered to represent the worst case meteorology.

In all cases analyzed, with the exception of postulated rupture of a 20,000 gallon tank, propane-air concentrations diminished to below the LEL at distances much shorter than the 1,800 ft minimum distance from the tank(s) to the Canister Transfer Building and the nearest storage casks. However, in the case of postulated rupture of a 20,000 gallon tank, explosive concentrations of propane traveled to distances beyond 1,800 ft under the worst case meteorological conditions evaluated. Therefore, PFS will design the propane storage for supplying propane to heat the Canister Transfer Building and Security and Health Physics Building with 4 separate tanks, with each tank having a capacity of less than or equal to 5,000 gallons for a total capacity of not more than 20,000 gallons. The 4 tanks shall be separated by missile walls to ensure that a single missile driven by the design tornado can not rupture more than one tank. The design will assure that it is not credible that more than one of the tanks could rupture at any given time.

Each propane tank shall have an excess flow shutoff valve that automatically isolates upon sensing high flow that could be due to a downstream line rupture or large leak. In addition, a single excess flow shutoff valve shall be located on the 2 inch piping header that supplies propane to the Canister Transfer Building and Security and Health Physics Building, downstream of the connection points of the lines from the 4 propane tanks. This valve shall also be designed to automatically close upon sensing high flow conditions indicative of a line rupture or large leak. This system of automatic isolation valves will serve to automatically isolate pipeline ruptures, thus preventing significant leakage of propane in the vicinity of the Canister Transfer Building or Security and Health Physics Building.

The analyses provided in Attachment 3 also assess overpressures that could occur from postulated propane vapor cloud explosions, assuming ignition occurs near the center of the plumes for each of the 4 propane release cases evaluated. The effects of explosions were analyzed using the TNT energy equivalent methodology, described in PFSF SAR Section 8.2.4. In all cases analyzed, with the exception of postulated rupture of a 20,000 gallon tank, overpressures decreased to less than 1 psi prior to reaching the Canister Transfer Building and nearest storage casks.

The PFSF license application will be updated as required to reflect the above information, and that included in the attachments to this letter, and submitted to the NRC by June 23, 2000.

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

Sincerely

John L. Donnell

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

Attachments

Copy to: Mark Delligatti-1/1
John Parkyn-1/0
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Sherwin Turk-1/0
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Murray Wade-1/0
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ATTACHMENT 1

CALCULATION PACKAGE ADDRESSING BEARING CAPACITY AND SLIDING STABILITY ANALYSES OF THE CASK STORAGE PADS AND THE CANISTER TRANSFER BUILDING

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

▲ 5010.64 (FRONT)

CLIENT & PROJECT	PAGE 1 OF
PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY	87+1A,1B,5A +15 pp OF ATTACHMENTS

CALCULATION TITLE (Indicative of the Objective):	QA CATEGORY (√)
STABILITY ANALYSES OF STORAGE PADS	☑ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ______ OTHER

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.01	G(B)	04		119

* APPROVALS - SIGNATURE & DATE						
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION * REQUIRED (√) YES	NO
TE SPONSELLER 2-18-97 Tom Sponseller; PJ TRUDEAU 2-24-97 Paul J. Trudeau	PAUL J. TRUDEAU 2-24-97 Paul J. Trudeau; TE SPONSELLER 2-24-97 Tom Sponseller	NURI T. GEORGES 2-27-97 Nuri T. Georges	0		✓	
T.E. SPONSELLER 4/30/97 Tom Sponseller; PAUL J. TRUDEAU 4/30/97 Paul J. Trudeau	PAUL J. TRUDEAU 4/30/97 Paul J. Trudeau; TE SPONSELLER 4/30/97 Tom Sponseller	Alan F Brown 5/8/97 Alan Brown	1	0		✓
PAUL J. TRUDEAU 6/20/97 Paul J. Trudeau	NURI T. GEORGES 6/20/97 Nuri T. Georges	ALAN F Brown 6/20/97 Alan Brown	2	1		✓

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CALCULATION TITLE (Indicative of the Objective):	QA CATEGORY (✓)
STABILITY ANALYSES OF STORAGE PADS	☑ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ______ OTHER

CALCULATION IDENTIFICATION NUMBER				
J. O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.01	G(B)	04		

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PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
CONTINUED FROM PAGE 1						
PAUL J. TRUDEAU 6/27/97 Paul J. Trudeau	LP SINGH [signature] 7-1-97	LP SINGH [signature] 7-1-97	3	2		✓
DL ALOYSIUS [signature] 9/3/99 S.Y. BOAKYE 9/3/99 [signature]	S.Y. BOAKYE 9/3/99 [signature] DL ALOYSIUS [signature]	T.Y. Chang 9/3/99 Thomas Y. Chang T.Y. Chang 9/3/99 Thomas Y. Chang	4	3	✓	
SEE PAGE 3 FOR IDENTIFICATION OF PREPARED/REVIEWED BY						
PJ TRUDEAU 1-26-2000 Paul J. Trudeau	SY BOAKYE 1-26-2000 TYC for S Y Boakye L. LIU 1-26-2000 [signature]	Thomas Y. Chang 1-26-2000 Thomas Y. Chang	5	4		✓

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CALCULATION TITLE (Indicative of the Objective):	QA CATEGORY (✓)
STABILITY ANALYSES OF STORAGE PADS	☑ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ______ OTHER

CALCULATION IDENTIFICATION NUMBER				
J. O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	G(B)	04		

* APPROVALS - SIGNATURE & DATE			REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION *REQUIRED (✓)	
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
PAUL J. TRUDEAU 6-16-00 Paul J. Trudeau	THOMAS Y. CHANG 6-16-00 Thomas Y. Chang	THOMAS Y. CHANG 6-16-00 Thomas Y. Chang	6	5		✓

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TABLE OF CONTENTS

RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1

Revision 1 was prepared to incorporate the following:

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

REVISION 2

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

REVISION 3

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

REVISION 4

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

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

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

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

Modified/updated conclusions.

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

Remaining pages prepared by DLAloysius and reviewed by SYBoakye.

REVISION 5

Major re-write of the calculation.

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

REVISION 6

1. Added "References" section.
2. Revised shear strength used in the sliding stability analyses of the soil cement/silty clay interface to be the strength measured in the direct shear tests performed on samples obtained from depths of ~5.8 ft in the pad emplacement area. The shear strength equaled that measured for stresses corresponding to the vertical stresses at the bottom of the fully loaded cask storage pads.
3. Removed static and dynamic bearing capacity analyses based on total-stress strengths and added dynamic bearing capacity analyses based on c_u = 2.2 ksf..
4. 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.

OBJECTIVE OF CALCULATION

Evaluate the static & seismic stability of the cask storage pad foundations at the proposed site, including overturning, sliding, and bearing capacity for static loads & for dynamic loads due to the design basis ground motion (PSHA 2,000-yr return period earthquake).

ASSUMPTIONS/DATA

The arrangement of the cask storage pads is shown on SWEC Drawing 0599601-EY-2-B. The spacing of the pads is such that each N-S row of pads may be treated as one long strip footing with B/L ~ 0 & B=30 ft for the bearing capacity analyses.

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

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

Figure 1 also illustrates the coordinate system used in these analyses. Note, the X-direction is N-S, the Y-direction is vertical, and the Z-direction is E-W. This is the same coordinate system that is used in the stability analyses of the Canister Transfer Building (Calculation 05996.02-G(B)-13-2, SWEC, 2000b).

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

Based on probabilistic seismic hazard analysis, the peak acceleration levels of 0.528g for horizontal ground motion and 0.533g 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, 1999b).

GEOTECHNICAL PROPERTIES

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

γ_{moist} = 80 pcf 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 Case IIIB, where B' = 16.3 ft. Figure 7 illustrates that the anticipated slip surface of the bearing capacity failure would be limited to the soils within the upper two-thirds of the upper layer. Therefore, in the bearing capacity analyses presented herein, the undrained strength measured in the UU triaxial tests was not increased to reflect the increase in strength observed for the deeper-lying soils in the cone penetration testing.

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

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

ksf is a very conservative value for use in the dynamic bearing capacity analyses of these structures.

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

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

The results of these tests are presented in Attachment 7 of the Appendix 2A of the SAR and they are plotted in Figure 7 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C). Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, sliding stability analyses included below of the cask storage pads constructed directly on the silty clay are performed using 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 ϕ = 30°. This value of ϕ is based on the PI values for these soils, which ranged between 5% and 23% (SWEC, 2000a), and the relationship between ϕ and PI presented in Figure 18.1 of Terzaghi & Peck (1967).

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

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

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

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

METHOD OF ANALYSIS

DESCRIPTION OF LOAD CASES

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

The following load combinations are analyzed:

Case I Static

Case II Static + dynamic horizontal forces due to the earthquake

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

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

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

Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.

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

OVERTURNING STABILITY OF THE CASK STORAGE PADS

The factor of safety against overturning is defined as:

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

The resisting moment is calculated as the 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 64 ft x 30 ft x 0.15 kips/ft^3 = 864 K, and the weight of 8 casks is 8 x 356.5 K/cask = 2,852 K. The moment arm for the resisting moment equals ½ of 30 ft, or 15 ft. Therefore,

$$\Sigma M_{Resisting} = [\underbrace{864\text{ K}}_{Wp} + \underbrace{2{,}852\text{K}}_{Wc}] \times \underbrace{15\text{ ft}}_{B/2} = 55{,}740\text{ ft-K}$$

The driving moment includes the moments due to the horizontal inertial force of the pad x ½ the height of the pad, the vertical inertial force of the pad plus casks x ½ the minimum width 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 - 0.533 x 2,852K) = 1,066 K. This is less than the maximum dynamic cask horizontal driving force of 1,855 K (Table D-1(c) in CEC, 1999). 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 1,066K.

$$\Sigma M_{Driving} = 1.5\text{ ft} \times \underbrace{0.528}_{a_h} \times \underbrace{864\text{ K}}_{Wp} + \underbrace{0.533}_{a_v} \times [\underbrace{864\text{ K}}_{Wp} + \underbrace{2{,}852\text{ K}}_{Wc}] \times \underbrace{15\text{ ft}}_{B/2} + 3\text{ ft} \times \underbrace{1{,}066\text{ K}}_{EQhc} = 33{,}592\text{ ft-K.}$$

$$\Rightarrow FS_{OT} = \frac{55{,}740\text{ ft} - \text{K}}{33{,}592\text{ ft} - \text{K}} = 1.66$$

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

SLIDING STABILITY OF THE CASK STORAGE PADS

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

FS = resisting force ÷ driving force

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

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

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

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

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

B = 30 feet

L = 64 feet

SLIDING STABILITY OF THE PADS CONSTRUCTED ON AND WITHIN SOIL CEMENT

Objective:

Determine the minimum required strength of the soil cement to provide a factor of safety against sliding of the cask storage pads of 1.1.

Method/Assumptions:

1. Assume that the resistance to sliding is provided only by the passive resistance of the soil-cement layer above the bottom of the pads, ignoring the contribution of the frictional portion of the strength.
2. Ignore the passive resistance of the overlying compacted aggregate.
3. Assume the active thrust of the compacted aggregate is less than the passive thrust and, thus, the active thrust can be ignored.
4. Use Eq 23.8a of Lambe & Whitman (1969) to calculate passive thrust, P_p, as follows:

$$P_P = ½\,\gamma_w\,H^2 + ½\,\gamma_b\,H^2\,N_\phi + q_s\,H\,N_\phi + 2\,\bar{c}\,H\sqrt{N_\phi}$$

where:

H	=	height of soil cement above bottom of pad
N_ϕ	=	K_p, coefficient of passive pressure, = 1 assuming $\phi = 0$.
q_s	=	uniform surcharge, = $(\gamma \times H)_{\text{compacted aggregate}}$, > 0.125 kcf x 0.71 ft = 0.09 ksf
$\bar{c}$	=	effective cohesion

Analysis:

Figure 3 presents an elevation view of the minimum thickness of soil cement in the vicinity of the cask storage pads. Figure 4 illustrates the passive pressures acting on the pads.

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

$$1.1 \times \left[3' \times 30' \times 64' \times 0.15 \frac{K}{ft^3} + 8 \text{ casks} \times 356.5 \frac{K}{\text{cask}} \right] \times 0.528$$

$$\therefore \quad T = 2{,}158 \text{ K}$$

Assuming this resisting force is provided only by the passive resistance provided by the 2-ft thick layer of soil cement adjacent to the pads, as shown in Figures 3 & 4, the minimum required strength of the soil cement is calculated as follows. Note, ignore buoyancy, since the depth to the water table is ~124.5 ft below grade, as measured in Observation Well CTB-5 OW.

$$P_P = \frac{1}{2} \gamma H^2 N_\phi + q_s H N_\phi + 2\bar{c} H \sqrt{N_\phi} \qquad \text{EQ 23.8a of Lambe \& Whitman (1969}$$

where $\quad q_s = (\gamma \cdot H)_{\substack{\text{compacted}\\ \text{aggregate}}} = 0.125 \frac{K}{ft^3} \times \frac{8.5 \text{ in.}}{12 \text{ in./ft}} = 0.09 \text{ ksf/LF}$, which is negligible.

Conservatively assuming $\phi = 0°$ for soil cement, $N_\phi = K_P = 1.0$.

Assuming sliding resistance is provided only by the passive resistance of the soil cement, the minimum resistance will exist for sliding in the N-S direction, because the width in the east-west direction (B=30') is less than the length in the north-south direction (L=64').

Find the minimum cohesion required to provide FS = 1.1.

$$P_P \text{ must be} \geq 2{,}158\text{K} = \frac{1}{2} \cdot \overset{\gamma}{0.100 \frac{K}{ft^3}} \times \overset{H^2}{(2 \text{ ft})^2} \times \overset{K_P}{1.0} + 2\bar{c} \cdot \overset{H}{2 \text{ ft}} \cdot \overset{\sqrt{N_\phi}}{\sqrt{1.0}}$$

$$\frac{2{,}158 \text{ K}}{30 \text{ ft}} = 0.2 \frac{K}{ft} + 4\bar{c} = 71.93 \frac{K}{LF} \Rightarrow 4\bar{c} = 71.73 \frac{K}{LF}$$

$$\therefore \quad \bar{c} \geq 17.93 \frac{\text{ksf}}{LF} \times \left(\frac{ft}{12 \text{ in.}} \right)^2 \times \frac{1{,}000 \#}{K} = 125 \text{ psi}$$

The unconfined compressive strength equals twice the cohesion, or 250 psi. Soil cement with strengths higher than this are readily achievable, as illustrated by the lowest curve in Figure 4.2 of ACI 230.1R-90, which applies for fine-grained soils similar to the eolian silt in the pad emplacement area. Note, $f_c = 40C$ where C = percent cement in the soil cement. Therefore, to obtain $f_c > 250$ psi, the percentage of cement required would be ~250/40 =

6.25%. This is even less cement than would typically be used in constructing soil cement for use as road base, and it would be even lower if shear resistance acting on the base of the pad was included or if K_p was calculated for $\phi > 0°$. Note, Tables 5 & 6 of Nussbaum & Colley (1971) indicate ϕ exceeds 40° for all A-4 soils (CL & ML) treated with cement. Therefore, soil cement will greatly improve the sliding stability of the cask storage 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 Figure's 2.6-5, Pad Emplacement Area Foundation Profiles, it will typically extend 3 to 5 ft below most of the pads. Thus, the area available to resist sliding will greatly exceed that of the pads alone. The soil cement will have higher shear strength than the underlying silty clay/clayey silt layer; therefore, the resistance to sliding on that interface will be limited by the shear strength of the silty clay/clayey silt. Direct shear tests on samples of the soils from the in the pad emplacement area indicate the shear strength available to resist sliding from loads due to the design basis ground motion 2.1 ksf as shown in Figure 7 of Calc 05996.02-G(B)-5-2 (copy included in Attachment C).

The following pages illustrate that there is an adequate factor or safety against sliding of the pads, postulating that they are constructed directly on the silty clay/clayey silt and neglecting the passive resistance provided by the soil cement that will be surrounding the pads. The factor of safety against sliding along the soil cement/silty clay interface will be much greater than this, because the shearing resistance will be available over the areas between the pads, as well as under the pads, and additional passive resistance will be provided by the continuous soil cement layer existing below the pads.

SLIDING STABILITY OF THE PADS CONSTRUCTED DIRECTLY ON SILTY CLAY/CLAYEY SILT

Material around the pad will be soil cement. In this analysis, the passive resistance provided by the soil cement is ignored to demonstrate that there is an acceptable factor of safety against sliding of the pads if they were founded directly on the silty clay/clayey silt. The soil cement is assumed to have the same properties that were used in Rev 4 of this calculation to model the crushed stone (compacted aggregate) that was originally proposed adjacent to the pads. These include:

γ = 125 pcf — Because of the low density of the eolian silts that will be used to construct the soil cement, it is likely that γ will be less than this value. It is conservative to use this higher value, because it 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.

ϕ = 40° — Tables 5 & 6 of Nussbaum & Colley (1971) indicate that ϕ exceeds 40° for all A-4 soils (CL & ML, similar to the eolian silts at the site) treated with cement; therefore, it is likely that ϕ will be higher than this value. This value is not used, however, in this analysis for calculating sliding resistance. It 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.

H = 3 ft — As shown in SAR Figure 4.2-7, the pad is 3 ft thick, but it is constructed such that the top is 3.5" above grade to accommodate potential settlement. The depth of the pad is used in this analysis only for calculating the maximum dynamic lateral earth pressure; therefore, it is conservative to ignore the 3.5" that the pad sticks out of the ground.

The resistance to sliding is lower when the forces due to the earthquake act upward; therefore, analyze the sliding stability for Load Case III, which has the dynamic forces due to the earthquake acting upward. To increase the conservatism of this analysis, assume 100% of the dynamic forces due to the earthquake act in both the N-S and Vertical directions at the same time. The length of the pad in the N-S direction (64 ft) is greater than twice the width in the E-W direction (30 ft); therefore, estimate the driving forces due to dynamic active earth pressures acting on the length of the pad, tending to cause sliding to occur in the E-W direction. The maximum dynamic cask driving force, however, acts in the N-S direction. To be conservative, assume that it acts in the E-W direction in this analysis of sliding stability. However, the maximum horizontal force that can be applied to the top of the pad by the casks is limited to the maximum value of the coefficient of friction between the cask and the top of the pad, which equals 0.8, multiplied by the cask normal force.

ACTIVE EARTH PRESSURE

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

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

$P_a = [0.5 \times 125 \text{ pcf} \times (3 \text{ ft})^2 \times 0.22] \times 64 \text{ ft (length)/storage pad} = 7{,}920 \text{ lbs.}$

DYNAMIC EARTH PRESSURE

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

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

where :

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

β = slope of ground behind wall,

α = slope of back of wall to vertical,

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

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

δ = angle of wall friction,

ϕ = friction angle of the soil,

g = acceleration due to gravity.

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

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

γ = unit weight of soil,

H = wall height, and

K_{AE} is calculated as shown above.

$\beta = \alpha = 0$

$$\theta = \tan^{-1} \times \left(\frac{0.528}{1 - 0.533} \right) = 48.5^\circ$$

$\phi = 40^\circ$

Approximating $\sin(\phi - \theta) \approx 0$ and $\cos(\phi - \theta) \approx 1$

$$K_{AE} = \frac{1 - \alpha_V}{\cos\theta \cdot \cos(\delta + \theta)}$$

$$\delta = \frac{\phi}{2} = 20^\circ$$

$$\therefore \quad K_{AE} = \frac{1 - 0.533}{\cos 48.5^\circ \cdot \cos(20^\circ + 48.5^\circ)} = 1.92$$

Therefore, the combined static and dynamic active lateral earth pressure force is:

$$F_{AE_{E\text{-}W}} = P_{AE} = \frac{1}{2} \times \overset{\gamma}{125\text{ pcf}} \times \overset{H^2}{(3\text{ ft})^2} \times \overset{K_{AE}}{1.92} \times \overset{L}{64\text{ ft}} / \text{storage pad} = 69.1\text{ K in E - W direction.}$$

$$F_{AE_{N\text{-}S}} = 69.1\text{ K} \times \frac{30\text{ ft}}{64\text{ ft}} = 32.4\text{ K in the N - S direction.}$$

WEIGHTS

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

Pad: W_p = 3 ft x 64 ft x 30 ft x 0.15 kips/ft^3 = 864 K

EARTHQUAKE ACCELERATIONS – PSHA 2,000-YR RETURN PERIOD

a_H = horizontal earthquake acceleration = 0.528g

a_V = vertical earthquake acceleration = 0.533g

CASK EARTHQUAKE LOADINGS

EQvc = -0.533 x 2,852 K = -1,520 K (minus sign signifies uplift force)

EQhc$_x$ = 1,855 K (acting short direction of pad, E-W) $Q_{xd\ max}$ in Table D-1(c) in Att B

EQhc$_y$ = 1,791 K (acting in long direction of pad, N-S) $Q_{yd\ max}$ in Table D-1(c) "

Note: These maximum horizontal dynamic cask driving forces are from Calc 05996.02-G(PO17)-2, (CEC, 1999), 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. For frictional materials, sliding is critical when the foundation is unloaded due to uplift forces from the earthquake. Therefore, EQh$_{c\ max}$ is limited to a maximum value of 1,066 K for Case III, based on the upper-bound value of μ = 0.8, as shown in the following table:

	WT K	EQ$_{vc}$ K	N K	0.2 x N K	0.8 x N K	EQ$_{hc\ max}$ K
Case III – Uplift	2,852	-1,520	1,332	266	1,066	**1,066**
Case IV - EQ$_v$ Down	2,852	1,520	4,372	874	3,498	**1,855** E-W 1,791 N-S

Note:

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

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

FOUNDATION PAD EARTHQUAKE LOADINGS

EQvp = -0.533 x 864 K = -461 K

EQhp = 0.528 x 864 K = 456 K

CASE III: 100% N-S, -100% VERTICAL, 0% E-W

Minimum sliding resistance exists when EQvc and EQvp act in an upward direction (Case III), tending to unload the pad. For this case,

$$N = \underset{Wc}{2{,}852\text{ K}} + \underset{Wp}{864\text{ K}} + \underset{EQvc}{(-1{,}520\text{ K})} + \underset{EQvp}{(-461\text{ K})} = 1{,}735\text{ K}$$

$$T = \underset{N}{1{,}735\text{ K}} \times \tan \underset{\phi}{0°} + \underset{c}{2.1\text{ ksf}} \times \underset{B}{30\text{ ft}} \times \underset{L}{64\text{ ft}} = 4{,}032\text{ K}$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + EQhc$$

The factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{4{,}032\text{ K}} \div (\underset{F_{AE}}{69.1\text{ K}} + \underset{EQhp}{456\text{ K}} + \underset{EQhc}{1{,}066\text{ K}}) = 2.53$$

For this analysis, the value of EQhc was limited to the upper-bound value of the coefficient of friction, $\mu = 0.8$, x the cask normal load, because if Qxd exceeds this value, the cask would slide. The factor of safety exceeds the minimum allowable value of 1.1; therefore the pads 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: 100% N-S, 100% VERTICAL, 0% E-W EARTHQUAKE FORCES ACT DOWNWARD

When the earthquake forces act in the downward direction:

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

where, $N \text{ (normal force)} = \sum Fv = Wc + Wp + EQvc + EQvp$

$$N = \underset{Wc}{2{,}852\text{ K}} + \underset{Wp}{864\text{ K}} + \underset{EQvc}{1{,}520\text{ K}} + \underset{EQvp}{461\text{ K}} = 5{,}697\text{ K}$$

$$T = \underset{N}{5{,}697\text{ K}} \times \tan \underset{\phi}{0°} + \underset{c}{2.1\text{ ksf}} \times \underset{B}{30\text{ ft}} \times \underset{L}{64\text{ ft}}] = 4{,}032\text{ K}$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + EQhc$$

The factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{4{,}032\text{ K}} \div (\underset{F_{AE}}{69.1\text{ K}} + \underset{EQhp}{456\text{ K}} + \underset{EQhc}{1{,}855\text{ K}}) = 1.69$$

For this analysis, the larger value of EQhc (i.e., acting in the short direction of the pad) was used, because it produces a lower and, thus, more conservative factor of safety. The factor of safety exceeds the minimum allowable value of 1.1; therefore the pads 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 ($\doteq$ 0.2), because the driving forces due to the casks would be reduced.

These analyses illustrate that if the cask storage pads constructed directly on the silty clay/clayey silt layer, they would have an adequate factor of safety against sliding due to loads from the design basis ground motion. Because the soil cement is continuous between the pads, its interface with the silty clay will be much larger than that provided by the footprint of the pads and used in the analyses presented in this section. The soil cement will be mixed and compacted into the upper layer of the silty clay, providing a bond at the interface that will exceed the strength of the silty clay. Therefore, this interface will have more resistance to sliding than is included in these analyses and, thus, there will be adequate resistance at this interface to preclude sliding of the pads due to the loads from the design basis ground motion.

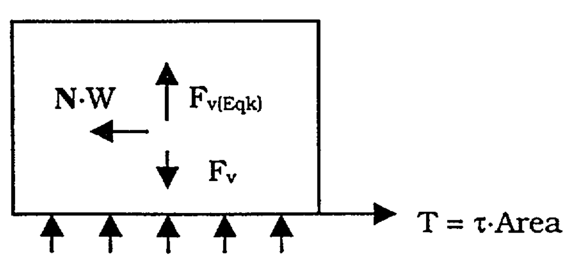
EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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

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

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

ESTIMATION OF HORIZONTAL DISPLACEMENT USING NEWMARK'S METHOD



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

For a block sliding on a horizontal surface, **N**·W = T,

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

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

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

$\sigma_n =$ Normal Stress

$\phi =$ Friction angle of cohesionless layer

$\sigma_n =$ Net Vertical Force/Area

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

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

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

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

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

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

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

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

MAXIMUM GROUND MOTIONS

The maximum ground accelerations used to estimate displacements of the cask storage pads were those due to the PSHA 2,000-yr return period earthquake; i.e., $a_H = 0.528g$ and $a_V = 0.533g$. 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 = 0.528 x 48 = 25.3 in./sec. Since the peak ground accelerations are the same in both horizontal directions, the velocities are the same as well.

LOAD CASES

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

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

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

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

GROUND MOTIONS FOR ANALYSIS

Load Case	North-South		Vertical	East-West	
	Accel g	Velocity in./sec	Accel g	Accel g	Velocity in./sec
IIIA	0.211g	10.1	0.533g	0.211g	10.1
IIIB	0.211g	10.1	0.213g	0.528g	25.3
IIIC	0.528g	25.3	0.213g	0.211g	10.1

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

Static Vertical Force, F_v = W = Weight of casks and pad = 2,852 K + 864 K = 3,716 kips

Earthquake Vertical Force, $F_{v\,Eqk} = a_v \times W/g$ = 0.533g x 3,716 K/g = 1,981 K

$\phi = 30°$

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

$$\mathbf{N} = [(\underbrace{3{,}716}_{F_v} - \underbrace{1{,}981}_{F_{v\,Eqk}}) \tan \underbrace{30°}_{\phi}] / \underbrace{3{,}716}_{W} = 0.270$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\underbrace{0.211^2}_{40\%\text{ N-S}} + \underbrace{0.211^2}_{40\%\text{ E-W}})} = 0.299g$

Resultant velocity in horizontal direction, $V = \sqrt{(\underbrace{10.1^2}_{40\%\text{ N-S}} + \underbrace{10.1^2}_{40\%\text{ E-W}})}$ = 14.3 in./sec

$$\Rightarrow \quad \mathbf{N} / A = 0.270 / 0.299 = 0.903$$

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.

$$\Rightarrow \quad u_m = \left(\frac{(14.3\,\text{in./sec})^2 \cdot (1-0.903)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.270}\right) = 0.1''$$

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 ~0.1 inches.

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

Static Vertical Force, F_v = W = 3,716 K

Earthquake Vertical Force, $F_{v(Eqk)}$ = 1,981 K x 0.40 = 792 K

$$\phi = 30°$$

$$\mathbf{N} = [(\underset{F_v}{3{,}716} - \underset{F_{v\,Eqk}}{792}) \tan \underset{\phi}{30°}] / \underset{W}{3{,}716} = 0.454$$

$$\text{Resultant acceleration in horizontal direction, } A = \sqrt{(\underset{40\%\ N\text{-}S}{0.211^2} + \underset{100\%\ E\text{-}W}{0.528^2})}\, g = 0.569g$$

$$\text{Resultant velocity in horizontal direction, } V = \sqrt{(\underset{40\%\ N\text{-}S}{10.1^2} + \underset{100\%\ E\text{-}W}{25.3^2})} = 27.2 \text{ in./sec}$$

$$\Rightarrow \quad \mathbf{N} / A = 0.454 / 0.569 = 0.798$$

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

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

$$\Rightarrow \quad u_m = \left(\frac{(27.2\,\text{in./sec})^2 \cdot (1-0.798)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.454}\right) = 0.43''$$

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 ~0.4 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

LOAD COMBINATION				DISPLACEMENT
Case IIIA	40% N-S	-100% Vert	40% E-W	0.1 inches
Case IIIB	40% N-S	-40% Vert	100% E-W	0.4 inches
Case IIIC	100% N-S	-40% Vert	40% E-W	0.4 inches

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with $\phi = 30°$, the estimated relative displacement of the pads due to the design basis ground motion based on Newmark's method of estimating displacements of embankments and dams due to earthquakes ranges from ~0.1 inches to 0.4 inches. Because there are no connections between the pads or between the pads and other structures, displacements of this magnitude, were they to occur, would not adversely impact the performance of the cask storage pads. There are several conservative assumptions that were made in determining these values and, therefore, the estimated displacements represent upper-bound values.

The soils in the layer that are assumed to be cohesionless, the one ~10 ft below the pads that is labeled "Clayey Silt/Silt & Some Sandy Silt" in the foundation profiles in the pad emplacement area (SAR Figures 2.6-5, Sheets 1 through 14), are clayey silts and silts, with some sandy silt. To be conservative in this analysis, these soils are assumed to have a friction angle of 30°. However, the results of the cone penetration testing (ConeTec, 1999) indicate that these soils have ϕ values that generally exceed 35 to 40°, as shown in Appendices D & F of ConeTec (1999). These high friction angles likely are the manifestation of cementation that was observed in many of the specimens obtained in split-barrel sampling and in the undisturbed tubes that were obtained for testing in the laboratory. Possible cementation of these soils is also ignored in this analysis, adding to the conservatism.

In addition, this analysis postulates that cohesionless soils exist directly at the base of the pads. In reality, the surface of these soils is 10 ft or more below the pads, and it is not likely to be continuous, as the soils in this layer are intermixed. For the pads to slide, a surface of sliding must be established between the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft below the pads, through the overlying clayey layer, and daylighting at grade. As shown in the analysis preceding this section, the overlying clayey layer is strong enough to resist sliding due to the earthquake forces. The contribution of the shear strength of the soils along this failure plane rising from the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft to the resistance to sliding is ignored in the simplified model used to estimate the relative displacement, further adding to the conservatism.

These analyses also conservatively ignore the presence of the soil cement under and adjacent to the cask storage pads. As shown above, this soil cement can easily be designed to provide all of the sliding resistance necessary to provide an adequate factor of safety, considering only the passive resistance acting on the sides of the pads, without relying on friction or cohesion along the base of the pads. Adding friction and cohesion along the base of the pads will increase the factor of safety against sliding.

Allowable Bearing Capacity of the Cask Storage Pads

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

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

where

q_{ult} = ultimate bearing capacity

c = cohesion or undrained strength

q = effective surcharge at bottom of foundation, = γD_f

γ = unit weight of soil

B = foundation width

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

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

i_c, i_q, i_γ = load inclination factors

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

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

Bearing Capacity Factors

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

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

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

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

SHAPE FACTORS (FOR *L>B*)

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

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

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

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

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

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

$$d_\gamma = 1$$

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.

STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

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

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

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

CALCULATION IDENTIFICATION NUMBER				PAGE 28
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

Allowable Bearing Capacity of Cask Storage Pads

Static Analysis: **Case IA - Static** **0 % in X, 0 % in Y, 0 % in Z**

Soil Properties: $c =$ **2,200** Cohesion (psf)

Undrained Strength $\phi =$ **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) $L' =$ **64.0** Length - ft (N-S

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ **0.0** Angle of load inclination from vertical (degrees)

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

$F_V =$ **3,716** k $EQ_V =$ **0** k

$EQ_{H\ E\text{-}W} =$ **0** k $EQ_{H\ N\text{-}S} =$ **0** k

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.81	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.04	Eq 3.27

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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**13,056**	**psf =**	**0**	**+**	**6,497**	**+**	**21,842**

q_{all} = 4,350 psf = q_{ult} / FS

q_{actual} = 1,936 psf = $(F_v + EQ_v) / (B' \times L')$

FS_{actual} = 6.75 = q_{ult} / q_{actual} > 3 Hence OK

Allowable Bearing Capacity of Cask Storage Pads

Static Analysis: **Case** **IB - Static** **0 % in X, 0 % in Y, 0 % in Z**

Soil Properties:	$c =$	**0** Cohesion (psf)	
Effective-Stress Strengths	$\phi =$	**30.0** Friction Angle (degrees)	
	$\gamma =$	**80** Unit weight of soil (pcf)	
	$\gamma_{surch} =$	**100** Unit weight of surcharge (pcf)	
Foundation Properties:	$B' =$	**30.0** Footing Width - ft (E-W)	$L' =$ **64.0** Length - ft (N-S
	$D_f =$	**2.7** Depth of Footing (ft)	
	$\beta =$	**0.0** Angle of load inclination from vertical (degrees)	
	$FS =$	**3.0** Factor of Safety required for $q_{allowable}$.	
	$F_V =$	**3,716** k	$EQ_V =$ **0** k
	$EQ_{H\ E\text{-}W} =$	**0** k	$EQ_{H\ N\text{-}S} =$ **0** k

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

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

$N_c = (N_q - 1)\cot(\phi)$, but $= 5.14$ for $\phi = 0$	=	30.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	=	18.40	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	22.40	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.29	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.27	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.81	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.03	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	1.03	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	N/A	Eq 3.27

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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**28,340**	**psf =**	**0**	**+**	**6,497**	**+**	**21,842**
q_{all} =	**9,440**	**psf = q_{ult} / FS**					
q_{actual} =	**1,936**	**psf = $(F_v + EQ_v) / (B' \times L')$**					
FS_{actual} =	**14.64**	**= q_{ult} / q_{actual}**			**> 3**		**Hence OK**

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

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS

Dynamic bearing capacity analyses are performed using two different sets of dynamic forces. In the first set of analyses, which are presented on Pages 32 to 45, 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, 1999), for the pad supporting 2 casks, 4 casks, and 8 casks.

BASED ON INERTIAL FORCES

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

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

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

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

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.

$$F_v = \underset{Wc}{2{,}852\text{ K}} + \underset{Wp}{864\text{ K}} = 3{,}716\text{ K and } EQ_v = 0 \text{ for this case.}$$

$$EQ_{H\ Pad} = \underset{a_H}{0.528} \times \underset{HT_{pad}}{3'} \times \underset{B}{30'} \times \underset{L}{64'} \times \underset{\gamma_{conc}}{0.15\text{ kcf}} = 456\text{ K}$$

$$EQhc = \text{Minimum of } [\underbrace{\underset{a_H}{0.528} \times \underset{Wc}{2{,}852\text{ K}}}_{1{,}506\text{ K}} \ \&\ \underbrace{\underset{\mu}{0.8} \times \underset{Nc}{2{,}852\text{ K}}}_{2{,}282\text{K}}] \quad \Rightarrow \quad EQhc = 1{,}506\text{ K}$$

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

$$EQ_{H\ N\text{-}S} = \underset{EQhp}{456\text{ K}} + \underset{EQhc}{1{,}506\text{ K}} = 1{,}962\text{ K}$$

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

Combine these horizontal components to calculate F_H:

$$\Rightarrow \quad F_H = \sqrt{EQ^2{}_{H\ E\text{-}W} + EQ^2{}_{H\ N\text{-}S}} = \sqrt{1{,}962^2 + 1{,}962^2} = 2{,}775\text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb.

$$\Delta b = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1{,}506\text{ K}}{2{,}852\text{ K} + 0} = 5.19\text{ ft}$$

$$\begin{aligned} \Sigma M_{@N\text{-}S} &= 1.5' \times \underset{a_H}{0.528} \times \underset{Wp}{864\text{ K}} + 3' \times \underset{EQhc}{1{,}506\text{ K}} + \underset{\Delta b}{5.19'} \times (\underset{Wc}{2{,}852\text{K}} + \underset{EQvc}{0}) \\ &= 684\text{ ft-K} + 4{,}518\text{ ft-K} + 14{,}804\text{ ft-K} = 20{,}006\text{ ft-K} \end{aligned}$$

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

$$\Sigma M_{@E\text{-}W} = \Sigma M_{@N\text{-}S} = 20{,}006\text{ ft-K}$$

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

CALCULATION IDENTIFICATION NUMBER				PAGE 33
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

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

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case II — **100 % in X, 0 % in Y, 100 % in Z**

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:

$B' =$ **19.2** Footing Width - ft (E-W) $L' =$ **53.2** Length - ft (N-S)

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ **27.8** Angle of load inclination from vertical (degrees)

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

$F_V =$ **3,716** k $EQ_V =$ **0** k

$EQ_{H\ E\text{-}W} =$ **1,962** k & $EQ_{H\ N\text{-}S} =$ **1,962** k $\rightarrow$ **2,775** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.07	Table 3.2
$s_q = 1 + (B/L)\ \tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\ (B/L)$	=	0.86	"
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
$m_B = (2 + B/L) / (1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B) / (1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	=	0.79 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H / [(F_v + EQ_v) + B'\ L'\ c \cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H / [(F_v + EQ_v) + B'\ L'\ c \cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\ F_H / B'\ L'\ c\ N_c)$	=	0.64	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**8,459**	**psf =**	**8,188**	**+**	**271**	**+**	**0**
q_{all} =	**7,690**	**psf = q_{ult} / FS**					
q_{actual} =	**3,630**	**psf = $(F_v + EQ_v) / (B' \times L')$**					
FS_{actual} =	**2.33**	**= q_{ult} / q_{actual}**			**> 1.1**	**Hence OK**	

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

Determine forces and moments due to earthquake.

$$EQ_V = -100\% \times \overset{a_V}{0.533} \times (\overset{Wp}{864\ K} + \overset{Wc}{2{,}852\ K}) = -1{,}981\ K$$

$$EQhp = \overset{a_H}{0.528} \times \overset{Wc}{864\ K} = 456\ K$$

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

— Cask EQvc = -1. x 0.533 x 2,852 K = - 1,520 K = $a_V \times Wc$

$\Rightarrow$ Nc = 1,332 K

$\Rightarrow$ $F_{EQ\ \mu=0.8} = 0.8 \times 1{,}332\ K = 1{,}066\ K$

$$EQhc = \text{Minimum of } [\underbrace{\overset{a_H}{0.528} \times \overset{Wc}{2{,}852\ K}}_{1{,}506\ K}\ \&\ \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{1{,}332K}}_{1{,}066K}]$$

Note: Use only 40% of the horizontal earthquake forces in this case.

40% of 1,506 K = 602 K, which is < $F_{EQ\ \mu=0.8}$; therefore, EQhc =1,506 K

$$\Rightarrow\ EQ_{H\ N\text{-}S} = \overset{40\%\ of}{0.4} \times [\overset{EQhp}{456\ K} + \overset{EQhc}{1{,}506\ K}] = 785\ K$$

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

$$\Rightarrow\ F_H = \sqrt{EQ^2{}_{H\ E\text{-}W} + EQ^2{}_{H\ N\text{-}S}} = \sqrt{785^2 + 785^2} = 1{,}110\ K$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E-W} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\ K}{2{,}852\ K - 1. \times 0.533 \times 2{,}852\ K} = 4.45\ ft$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{40\%\ a_H}{0.4 \times 0.528} \times \overset{Wp}{864\ K} + \overset{40\%\ EQhc}{0.4 \times 3' \times 1{,}506\ K} + \overset{\Delta b}{4.45'} \times (\overset{Wc}{2{,}852K} - \overset{EQvc}{1{,}520\ K})$$

$$= 274\ ft\text{-}K + 1{,}807\ ft\text{-}K + 5{,}927\ ft\text{-}K = 8{,}008\ ft\text{-}K$$

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

$$\Sigma M_{@E\text{-}W} = \Sigma M_{@N\text{-}S} = 8{,}008\ ft\text{-}K$$

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

CALCULATION IDENTIFICATION NUMBER				PAGE 35
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

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

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IIIA **40 % in X, -100 % in Y, 40 % in Z**

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:
- $B' =$ **20.8** Footing Width - ft (E-W) $L' =$ **54.8** Length - ft (N-S)
- $D_f =$ **2.7** Depth of Footing (ft)
- $\beta =$ **24.3** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **3,716** k $EQ_V =$ **-1,981** k
- $EQ_{H\,E\text{-}W} =$ **785** k & $EQ_{H\,N\text{-}S} =$ **785** k $\rightarrow$ **1,110** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.07	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.85	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.79 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.87	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	11,394	psf =	11,123	+	271	+	0
q_{all} =	10,350	psf = q_{ult} / FS					
q_{actual} =	1,525	psf = $(F_v + EQ_v)/(B' \times L')$					
FS_{actual} =	7.47	= q_{ult}/q_{actual}			> 1.1	Hence OK	

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.

$$EQ_V = -40\% \times \underset{a_v}{0.533} \times (\underset{Wp}{864\text{ K}} + \underset{Wc}{2{,}852\text{ K}}) = -792\text{ K}$$

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

— 40% of Cask EQvc = -0.4 x 0.533 x 2,852 K = - 608 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 2,244 K

$\Rightarrow$ $F_{EQ\,\mu=0.8} = 0.8 \times 2{,}224\text{ K} = 1{,}795\text{ K}$

$$EQhc = \text{Min of } [\underbrace{\underset{a_H}{0.528} \times \underset{Wc}{2{,}852\text{ K}}}_{1{,}506\text{ K}} \;\&\; \underbrace{\underset{\mu}{0.8} \times \underset{Nc}{2{,}244\text{ K}}}_{1{,}795\text{K}}] \Rightarrow EQhc = 1{,}506\text{ K, since it is} < F_{EQ\,\mu=0.8}$$

Using 40% of N-S: 40% of [EQhp EQhc]

$$\Rightarrow EQ_{H\,N\text{-}S} = 0.4 \times [456\text{ K} + 1{,}506\text{ K}] = 785\text{ K}$$

Using 100% of E-W: 100% of [EQhp EQhc]

$$\Rightarrow EQ_{H\,E\text{-}W} = 1.0 \times [456\text{ K} + 1{,}506\text{ K}] = 1{,}962\text{ K}$$

$$\Rightarrow F_H = \sqrt{EQ^2{}_{H\,E\text{-}W} + EQ^2{}_{H\,N\text{-}S}} = \sqrt{1{,}962^2 + 785^2} = 2{,}113\text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E-W} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1.0 \times 1{,}506\text{ K}}{2{,}852\text{ K} - 0.4 \times 0.533 \times 2{,}852\text{ K}} = 6.60\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \underset{100\%\,a_H}{0.528} \times \underset{Wp}{864\text{ K}} + 3' \times \underset{100\%\,EQhc}{1{,}506\text{ K}} + \underset{\Delta b}{6.60'} \times (\underset{Wc}{2{,}852\text{K}} - \underset{40\%\,EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 684\text{ ft-K} + 4{,}518\text{ ft-K} + 14{,}810\text{ ft-K} = 20{,}012\text{ ft-K}$$

$$\Delta b_{N-S} = \frac{9.83' \times 40\% EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\text{ K}}{2{,}852\text{ K} - 0.4 \times 0.533 \times 2{,}852\text{ K}} = 2.64\text{ ft}$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \underset{40\%\,a_H}{0.4 \times 0.528} \times \underset{Wp}{864\text{ K}} + 3' \times \underset{40\%\,EQhc}{0.4 \times 1{,}506\text{ K}} + \underset{\Delta b}{2.64'} \times (\underset{Wc}{2{,}852\text{K}} - \underset{40\%\,EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 274\text{ ft-K} + 1{,}807\text{ ft-K} + 5{,}924\text{ ft-K} = 8{,}005\text{ ft-K}$$

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

CALCULATION IDENTIFICATION NUMBER				PAGE 37
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

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

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IIIB — **40 % in X, -40 % in Y, 100 % in Z**

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:

$B' =$ **16.3** Footing Width - ft (E-W) $L' = 58.5$ Length - ft (N-S)

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ 33.9 Angle of load inclination from vertical (degrees)

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

$F_V =$ **3,716** k $EQ_V =$ **-792** k

$EQ_{H\,E\text{-}W} =$ **1,962** k & $EQ_{H\,N\text{-}S} =$ **785** k $\rightarrow$ **2,113** 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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.07	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	1.19	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.63	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 $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.68	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	8,926	psf =	8,655	+	271	+	0
q_{all} =	8,110	psf = q_{ult} / FS					
q_{actual} =	3,062	psf = $(F_v + EQ_v)/(B' \times L')$					
FS_{actual} =	2.92	= q_{ult}/q_{actual}			> 1.1	Hence OK	

CALCULATION IDENTIFICATION NUMBER				PAGE 38
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

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.

$$EQ_V = -40\% \times \overset{a_V}{0.533} \times (\overset{Wp}{864\text{ K}} + \overset{Wc}{2{,}852\text{ K}}) = -792\text{ K}$$

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

— 40% of Cask EQvc = -0.4 x 0.533 x 2,852 K = - 608 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 2,244 K

$\Rightarrow$ $F_{EQ\,\mu=0.8} = 0.8 \times 2{,}224\text{ K} = 1{,}795\text{ K}$

$$EQhc = \text{Min of } [\underbrace{\overset{a_H}{0.528} \times \overset{Wc}{2{,}852\text{ K}}}_{1{,}506\text{ K}} \;\&\; \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{2{,}244\text{ K}}}_{1{,}795\text{K}}] \Rightarrow EQhc = 1{,}506\text{ K, since it is} < F_{EQ\,\mu=0.8}$$

Using 100% of N-S:

$$\Rightarrow EQ_{H\,N\text{-}S} = \overset{100\%\text{ of}}{1.0} \times [\overset{EQhp}{456\text{ K}} + \overset{EQhc}{1{,}506\text{ K}}] = 1{,}962\text{ K}$$

Using 40% of E-W:

$$\Rightarrow EQ_{H\,E\text{-}W} = \overset{40\%\text{ of}}{0.4} \times [\overset{EQhp}{456\text{ K}} + \overset{EQhc}{1{,}506\text{ K}}] = 785\text{ K}$$

$$\Rightarrow F_H = \sqrt{EQ^2{}_{H\,E\text{-}W} + EQ^2{}_{H\,N\text{-}S}} = \sqrt{785^2 + 1{,}962^2} = 2{,}113\text{ K}$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E\text{-}W} = \frac{9.83' \times 40\%\, EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\text{ K}}{2{,}852\text{ K} - 0.4 \times 0.533 \times 2{,}852\text{ K}} = 2.64\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{40\%\,a_H}{0.4\times0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{40\%\,EQhc}{0.4\times1{,}506\text{ K}} + \overset{\Delta b}{2.64'} \times (\overset{Wc}{2{,}852\text{K}} - \overset{40\%\,EQvc}{0.4\times1{,}520\text{ K}})$$

$$= 274\text{ ft-K} + 1{,}807\text{ ft-K} + 5{,}924\text{ ft-K} = 8{,}005\text{ ft-K}$$

$$\Delta b_{N\text{-}S} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1.0 \times 1{,}506\text{ K}}{2{,}852\text{ K} - 0.4 \times 0.533 \times 2{,}852\text{ K}} = 6.60\text{ ft}$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \overset{100\%\,a_H}{0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{100\%\,EQhc}{1{,}506\text{ K}} + \overset{\Delta b}{6.60'} \times (\overset{Wc}{2{,}852\text{K}} - \overset{40\%\,EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 684\text{ ft-K} + 4{,}518\text{ ft-K} + 14{,}810\text{ ft-K} = 20{,}012\text{ ft-K}$$

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

CALCULATION IDENTIFICATION NUMBER				PAGE 39
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

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

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IIIC — **100 % in X, -40 % in Y, 40 % in Z**

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:
- $B' =$ **24.5** Footing Width - ft (E-W) $L' =$ **50.3** Length - ft (N-S)
- $D_f =$ **2.7** Depth of Footing (ft)
- $\beta =$ **15.0** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **3,716** k $EQ_V =$ **-792** k
- $EQ_{H\ E\text{-}W} =$ **785** k & $EQ_{H\ N\text{-}S} =$ **1,962** k → **2,113** k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.81	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1-\sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: d_c $= d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: d_c $= 1 + 0.4(D_f/B)$	=	1.04	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.38	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.37	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 $\phi = 0$: $i_c = 1 - (m F_H / B'L'c N_c)$	=	0.79	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**10,518**	**psf =**	**10,247**	+	**271**	+	**0**
q_{all} =	**9,560**	**psf = q_{ult} / FS**					
q_{actual} =	**2,369**	**psf = $(F_v + EQ_v)/(B' \times L')$**					
FS_{actual} =	**4.44**	**= q_{ult}/q_{actual}**					

> 1.1 Hence OK

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

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

Determine forces and moments due to earthquake.

$$EQ_V = 100\% \times \underset{a_V}{0.533} \times (\underset{Wp}{864\text{ K}} + \underset{Wc}{2{,}852\text{ K}}) = 1{,}981\text{ K}$$

$$EQhp = \underset{a_H}{0.528} \times \underset{Wc}{864\text{ K}} = 456\text{ K}$$

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

+ Cask EQvc = 1. x 0.533 x 2,852 K = + 1,520 K = $a_V \times Wc$

⇒ Nc = 3,498 K

⇒ $F_{EQ\,\mu=0.8} = 0.8 \times 4{,}372\text{ K} = 3{,}498\text{ K}$

$$EQhc = \text{Min of } [\underset{1{,}506\text{ K}}{\underset{a_H}{0.528} \times \underset{Wc}{2{,}852\text{ K}}} \;\&\; \underset{3{,}498\text{K}}{\underset{\mu}{0.8} \times \underset{Nc}{4{,}372\text{ K}}}] \Rightarrow EQhc = 1{,}506\text{ K, since it is} < F_{EQ\,\mu=0.8}$$

$$\Rightarrow EQ_{H\,N\text{-}S} = \underset{40\%\text{ of}}{0.4} \times [\underset{EQhp}{456\text{ K}} + \underset{EQhc}{1{,}506\text{ K}}] = 785\text{ K}$$

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

$$\Rightarrow F_H = \sqrt{EQ^2{}_{H\,E\text{-}W} + EQ^2{}_{H\,N\text{-}S}} = \sqrt{785^2 + 785^2} = 1{,}110\text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E-W} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\text{ K}}{2{,}852\text{ K} + 1. \times 0.533 \times 2{,}852\text{ K}} = 1.35\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \underset{40\%\ a_H}{0.4 \times 0.528} \times \underset{Wp}{864\text{ K}} + \underset{40\%\ EQhc}{0.4 \times 3' \times 1{,}506\text{ K}} + \underset{\Delta b}{1.35'} \times (\underset{Wc}{2{,}852\text{K}} + \underset{EQvc}{1{,}520\text{ K}})$$

$$= 274\text{ ft-K} + 1{,}807\text{ ft-K} + 5{,}921\text{ ft-K} = 8{,}002\text{ ft-K}$$

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

$$\Sigma M_{@E\text{-}W} = \Sigma M_{@N\text{-}S} = 8{,}002\text{ ft-K}$$

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

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

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IVA — **40 % in X, 100 % in Y, 40 % in Z**

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:

- $B' =$ **27.2** Footing Width - ft (E-W) $L' =$ **61.2** Length - ft (N-S)
- $D_f =$ **2.7** Depth of Footing (ft)
- $\beta =$ **7.8** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **3,716** k $EQ_V =$ **1,981** k
- $EQ_{H\ E\text{-}W} =$ **785** k & $EQ_{H\ N\text{-}S} =$ **785** k → **1,110** k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.82	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1-\sin\phi)^2\,D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.04	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.79	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.91	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**11,915**	**psf =**	**11,645**	+	**271**	+	**0**
q_{all} =	**10,830**	**psf = q_{ult} / FS**					
q_{actual} =	**3,424**	**psf = $(F_v + EQ_v)/(B' \times L')$**					
FS_{actual} =	**3.48**	**= q_{ult}/q_{actual}**					

> 1.1 Hence OK

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

Determine forces and moments due to earthquake.

$$EQ_V = 0.4 \times \overset{a_v}{0.533} \times (\overset{Wp}{864\ K} + \overset{Wc}{2{,}852\ K}) = 792\ K$$

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

+ 40% of Cask EQvc = +0.4 x 0.533 x 2,852 K = + 608 K = 40% of a_V x Wc

⇒ Nc = 3,460 K

$$\Rightarrow \quad F_{EQ\ \mu=0.8} = 0.8 \times 3{,}460\ K = 2{,}768\ K$$

$$EQhc = \text{Min of } [\underbrace{\overset{a_H}{0.528} \times \overset{Wc}{2{,}852\ K}}_{1{,}506\ K} \ \&\ \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{3{,}460\ K}}_{2{,}768K}] \Rightarrow EQhc = 1{,}506\ K, \text{ since it is} < F_{EQ\ \mu=0.8}$$

Using 40% of N-S:

$$\Rightarrow \quad EQ_{H\ N\text{-}S} = \overset{40\%\ of}{0.4} \times [\overset{EQhp}{456\ K} + \overset{EQhc}{1{,}506\ K}] = 785\ K$$

Using 100% of E-W:

$$\Rightarrow \quad EQ_{H\ E\text{-}W} = \overset{100\%\ of}{1.0} \times [\overset{EQhp}{456\ K} + \overset{EQhc}{1{,}506\ K}] = 1{,}962\ K$$

$$\Rightarrow \quad F_H = \sqrt{EQ^2{}_{H\ E\text{-}W} + EQ^2{}_{H\ N\text{-}S}} = \sqrt{1{,}962^2 + 785^2} = 2{,}113\ K$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E-W} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1.0 \times 1{,}506\ K}{2{,}852\ K + 0.4 \times 0.533 \times 2{,}852\ K} = 4.28\ ft$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{100\%\ a_H}{0.528} \times \overset{Wp}{864\ K} + 3' \times \overset{100\%\ EQhc}{1{,}506\ K} + \overset{\Delta b}{4.28'} \times (\overset{Wc}{2{,}852K} + \overset{40\%\ EQvc}{0.4 \times 1{,}520\ K})$$

$$= 684\ ft\text{-}K + 4{,}518\ ft\text{-}K + 14{,}810\ ft\text{-}K = 20{,}012\ ft\text{-}K$$

$$\Delta b_{N-S} = \frac{9.83' \times 40\%\ EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\ K}{2{,}852\ K + 0.4 \times 0.533 \times 2{,}852\ K} = 1.71\ ft$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \overset{40\%\ a_H}{0.4 \times 0.528} \times \overset{Wp}{864\ K} + 3' \times \overset{40\%\ EQhc}{0.4 \times 1{,}506\ K} + \overset{\Delta b}{1.71'} \times (\overset{Wc}{2{,}852K} + \overset{40\%\ EQvc}{0.4 \times 1{,}520\ K})$$

$$= 274\ ft\text{-}K + 1{,}807\ ft\text{-}K + 5{,}917\ ft\text{-}K = 7{,}998\ ft\text{-}K$$

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

CALCULATION IDENTIFICATION NUMBER				PAGE 43
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

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

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IVB **40 % in X, 40 % in Y, 100 % in Z**

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:
- $B' =$ **21.1** Footing Width - ft (E-W) $L' =$ **60.5** Length - ft (N-S)
- $D_f =$ **2.7** Depth of Footing (ft)
- $\beta =$ **23.5** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **3,716** k $EQ_V =$ 792 k
- $EQ_{H\ E\text{-}W} =$ **1,962** k & $EQ_{H\ N\text{-}S} =$ 785 k → **2,113** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.07	Table 3.2
$s_q = 1 + (B/L)\ \tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\ (B/L)$	=	0.86	"
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
$m_B = (2 + B/L) / (1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B) / (1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	=	1.19	rad
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.63	Eq 3.18c
$i_q = \{1 - F_H / [(F_v + EQ_v) + B'\ L'\ c\ \cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H / [(F_v + EQ_v) + B'\ L'\ c\ \cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\ F_H / B'\ L'\ c\ N_c)$	=	0.76	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**9,937**	**psf =**	**9,666**	**+**	**271**	**+**	**0**
q_{all} =	**9,030**	**psf = q_{ult} / FS**					
q_{actual} =	**3,530**	**psf = $(F_v + EQ_v) / (B' \times L')$**					
FS_{actual} =	**2.81**	**= q_{ult} / q_{actual}**					

> 1.1 Hence OK

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

Determine forces and moments due to earthquake.

$$EQ_V = 0.4 \times \overset{av}{0.533} \times (\overset{Wp}{864\text{ K}} + \overset{Wc}{2{,}852\text{ K}}) = 792\text{ K}$$

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

+ 40% of Cask EQvc = -0.4 x 0.533 x 2,852 K = + 608 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 3,460 K

$$\Rightarrow \quad F_{EQ\,\mu=0.8} = 0.8 \times 3{,}460\text{ K} = 2{,}768\text{ K}$$

$$EQhc = \text{Min of } [\underset{1{,}506\text{ K}}{\overset{a_H}{0.528} \times \overset{Wc}{2{,}852\text{ K}}} \;\&\; \underset{2{,}768\text{ K}}{\overset{\mu}{0.8} \times \overset{Nc}{3{,}460\text{ K}}}] \Rightarrow EQhc = 1{,}506\text{ K, since it is} < F_{EQ\,\mu=0.8}$$

Using 100% of N-S:

$$\Rightarrow \quad EQ_{H\,N\text{-}S} = \overset{100\%\text{ of}}{1.0} \times [\overset{EQhp}{456\text{ K}} + \overset{EQhc}{1{,}506\text{ K}}] = 1{,}962\text{ K}$$

Using 40% of E-W:

$$\Rightarrow \quad EQ_{H\,E\text{-}W} = \overset{40\%\text{ of}}{0.4} \times [\overset{EQhp}{456\text{ K}} + \overset{EQhc}{1{,}506\text{ K}}] = 785\text{ K}$$

$$\Rightarrow \quad F_H = \sqrt{EQ^2_{H\,E\text{-}W} + EQ^2_{H\,N\text{-}S}} = \sqrt{785^2 + 1{,}962^2} = 2{,}113\text{ K}$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E-W} = \frac{9.83' \times 40\%\, EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\text{ K}}{2{,}852\text{ K} + 0.4 \times 0.533 \times 2{,}852\text{ K}} = 1.71\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{40\%\,a_H}{0.4 \times 0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{40\%\,EQhc}{0.4 \times 1{,}506\text{ K}} + \overset{\Delta b}{1.71'} \times (\overset{Wc}{2{,}852\text{K}} + \overset{40\%\,EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 274\text{ ft-K} + 1{,}807\text{ ft-K} + 5{,}917\text{ ft-K} = 7{,}998\text{ ft-K}$$

$$\Delta b_{N-S} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1.0 \times 1{,}506\text{ K}}{2{,}852\text{ K} + 0.4 \times 0.533 \times 2{,}852\text{ K}} = 4.28\text{ ft}$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \overset{100\%\,a_H}{0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{100\%\,EQhc}{1{,}506\text{ K}} + \overset{\Delta b}{4.28'} \times (\overset{Wc}{2{,}852\text{K}} + \overset{40\%\,EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 684\text{ ft-K} + 4{,}518\text{ ft-K} + 14{,}808\text{ ft-K} = 20{,}010\text{ ft-K}$$

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

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

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IVC | **100 % in X, 40 % in Y, 40 % in Z**

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:

$B' =$	26.5	Footing Width - ft (E-W)	$L' =$ 55.1	Length - ft (N-S)
$D_f =$	2.7	Depth of Footing (ft)		
$\beta =$	9.9	Angle of load inclination from vertical (degrees)		
$FS =$	1.1	Factor of Safety required for $q_{allowable}$.		
$F_V =$	3,716 k	$EQ_V =$	792 k	
$EQ_{H\ E\text{-}W} =$	785 k	& $EQ_{H\ N\text{-}S} =$	1,962 k →	2,113 k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.81	"
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 $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.04	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.38 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.37	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 $\phi = 0$: $i_c = 1 - (m F_H / B' L' c N_c)$	=	0.82	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**10,882**	**psf =**	**10,612**	**+**	**271**	**+**	**0**
q_{all} =	**9,890**	**psf = q_{ult} / FS**					
q_{actual} =	**3,092**	**psf = $(F_v + EQ_v)/(B' \times L')$**					
FS_{actual} =	**3.52**	**= q_{ult}/q_{actual}**			**> 1.1 Hence OK**		

[geot]j05996\calc\brng_cap\Pad\cu_phi.xls

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 7.7 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 condition was 2.3, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

The following pages determine the allowable bearing capacity for the cask storage pads with respect to the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 1999) 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 these analyses is the same as that used for the analyses discussed above, which is shown in Figure 1. Note, this is different than the coordinate system used in Calculation 05996.02-G(PO17)-2 (CEC, 1999), which is shown on Page B11. Therefore, in the following pages, the X direction is N-S, the Y direction is vertical, and the Z direction is E-W.

These maximum dynamic cask driving forces were confirmed to be less than the maximum force that can be transmitted from the cask to the pad through friction acting at the base of the cask for each of these load cases. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad (μ = 0.8, as shown in SAR Section 8.2.1.2) x the normal force acting between the casks and the pad. These maximum dynamic cask driving forces can be transmitted to the pad through friction only when the inertial vertical forces act downward; therefore, these analyses are performed only for Load Case IV. The analyses conservatively assume that 100% of the horizontal forces act in the E-W and vertical directions at the same time. The width (30 ft) is less in the E-W direction than the length N-S (64 ft); therefore, the E-W direction is the critical direction with respect to a bearing capacity failure.

NOTED ~~JUN 1 9 1997~~ P. J. Trudeau
1-26-00

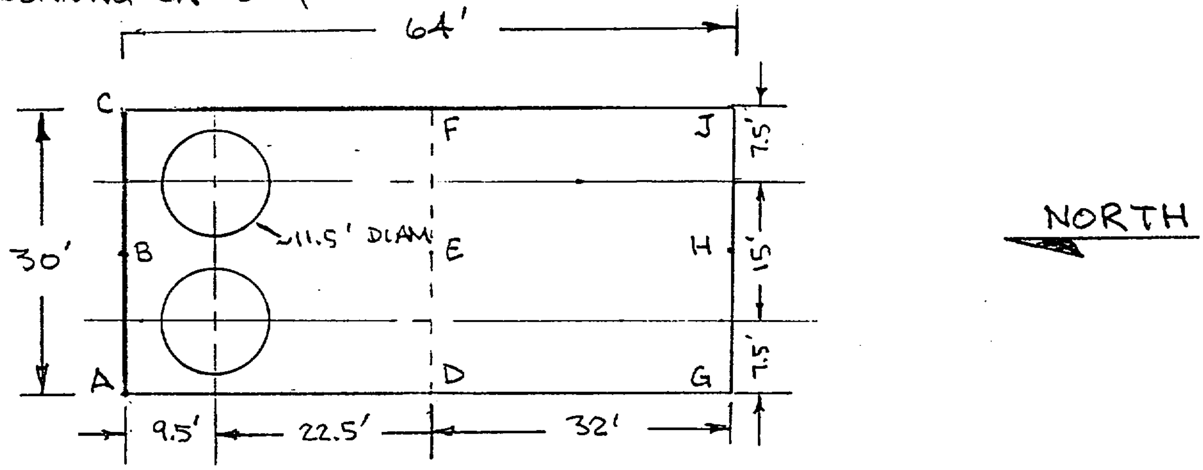
STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

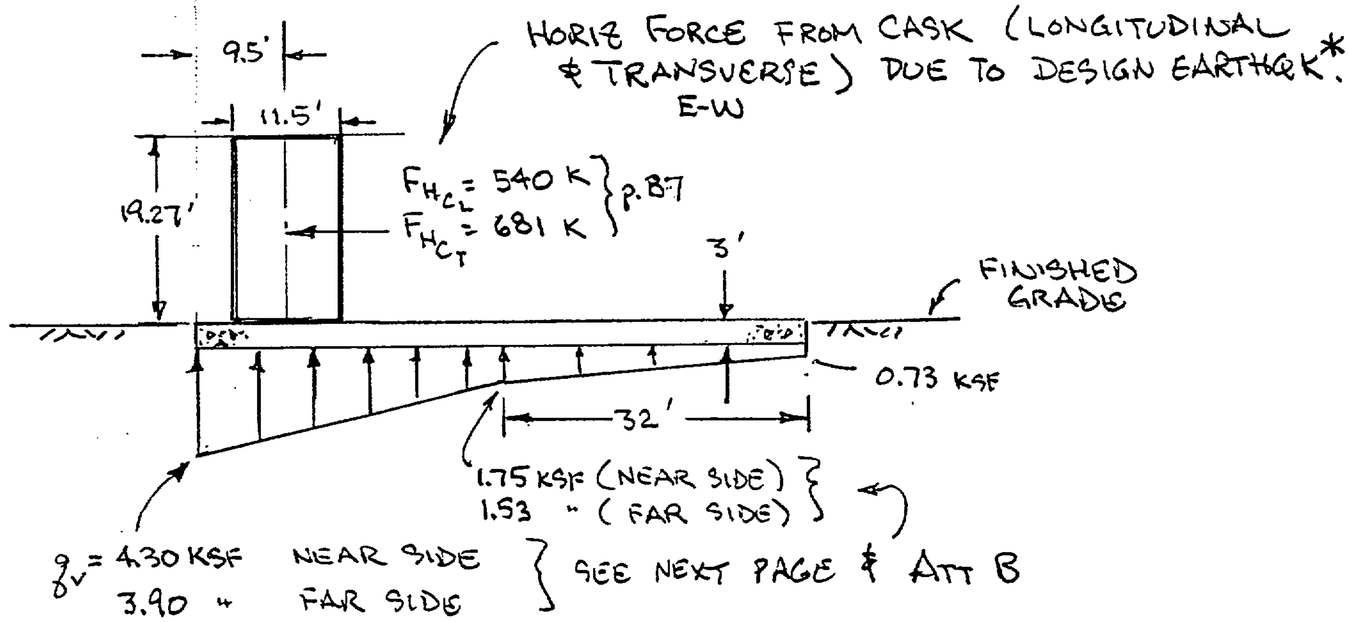
▲5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 47
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 2-CASK CASE



PLAN



ELEV

* STRESSES AT PAD/SOIL INTERFACE OBTAINED FROM CEC (1999) CALC 05996.02-G(P017)-2, REV 1 - COPIES OF PERTINENT PAGES ARE INCLUDED IN ATT B OF THIS CALC

00866

DYN BEARING CAPACITY OF PAD: 2-CASK CASE

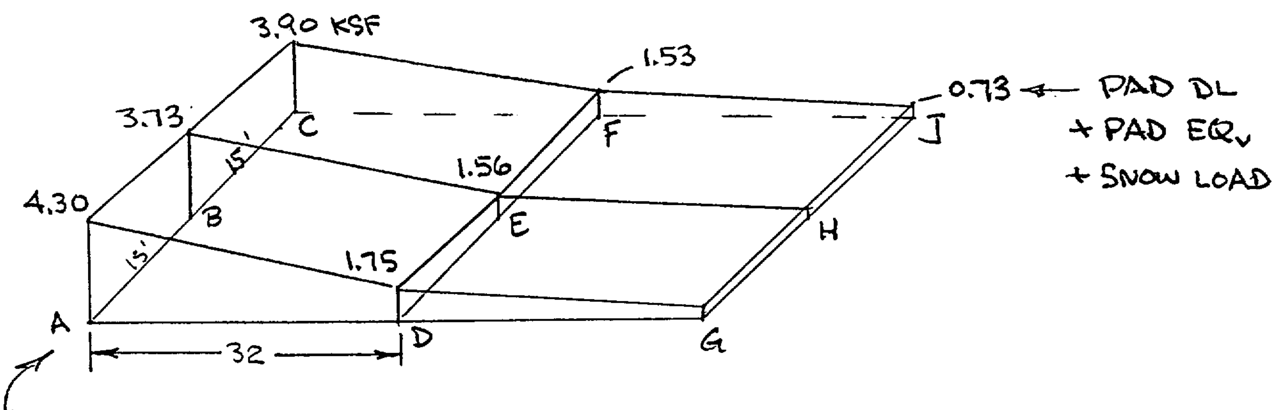
SOIL BEARING PRESSURES ARE BASED ON INFO FROM CEC (1999) INCLUDED IN ATT B AND ARE SUMMARIZED IN TABLE 1.

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

CASK LL = 1.36 KSF ALONG LINE AC & IS ASSUMED TO DECREASE LINEARLY TO 0 ALONG LINE DF.

CASK EQ PRESSURES ARE SHOWN ON TABLE 1.

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



FOR LOADED HALF OF PAD:

$$F_v \simeq \left[\frac{15'}{2}\times\overbrace{(4.30+2\times3.73+3.90)}^{[117.45}{}^{KSF}+\frac{15'}{2}\overbrace{(1.75+2\times1.56+1.53)}^{48.00]}{}^{KSF}\right]\frac{32'}{2}$$

$F_v \simeq 2647$ K FOR LOADED 1/2 OF PAD

$$A_{AC} \sim 117.45\ \frac{K}{FT} = 30'\times q_{AVG_{AC}} \Rightarrow q_{AVG_{AC}} = 3.92\ KSF$$

$$A_{DF} \sim 48.00\ \frac{K}{FT} = 30\ q_{AVG_{DF}} \Rightarrow q_{AVG_{DF}} = 1.60\ KSF$$

STONE & WEBSTER ENGINEERING CORPORATION

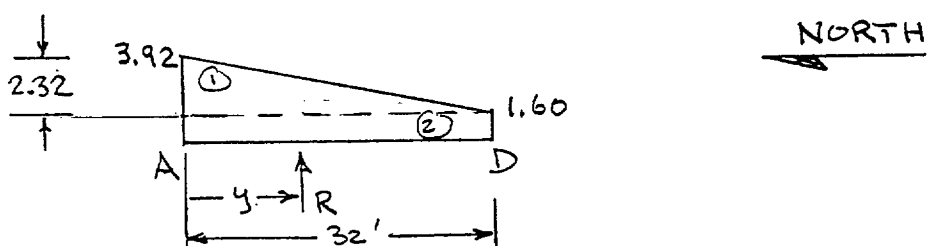
CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 49
8				
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 2-CASK CASE

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



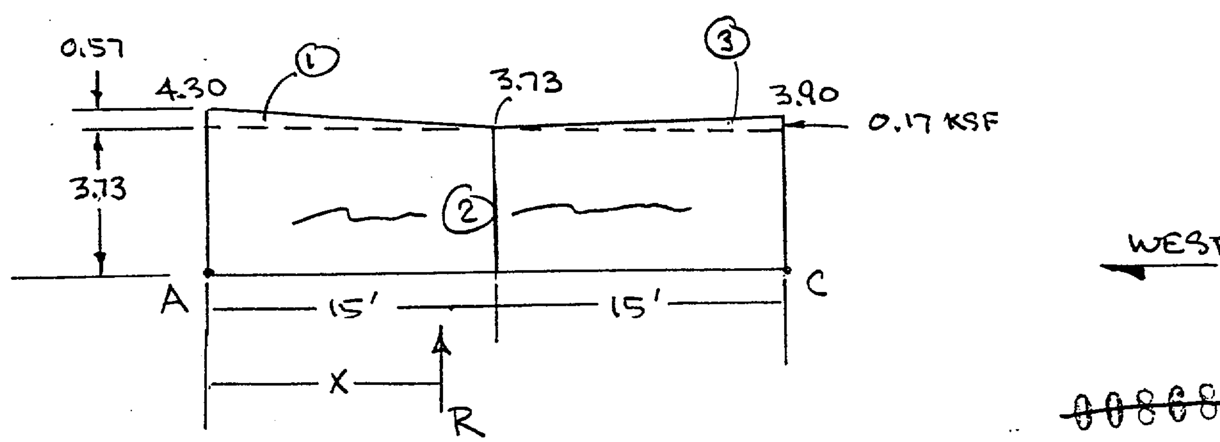
ΣM_A TO FIND LOC'N OF F_V

$$Ry = \tfrac{1}{2}\, 2.32 \times 32 \times \tfrac{1}{3} 32 + 1.60 \times 32 \times \tfrac{32}{2}$$

$$R = \tfrac{1}{2}\, 2.32 \times 32 + 1.60 \times 32 = 37.12 + 51.20 = 88.32\,K$$

$$y = \frac{395.95 + 819.20}{88.32} \text{ K-FT} = 13.76 \text{ FT}$$

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



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CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 50
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

9

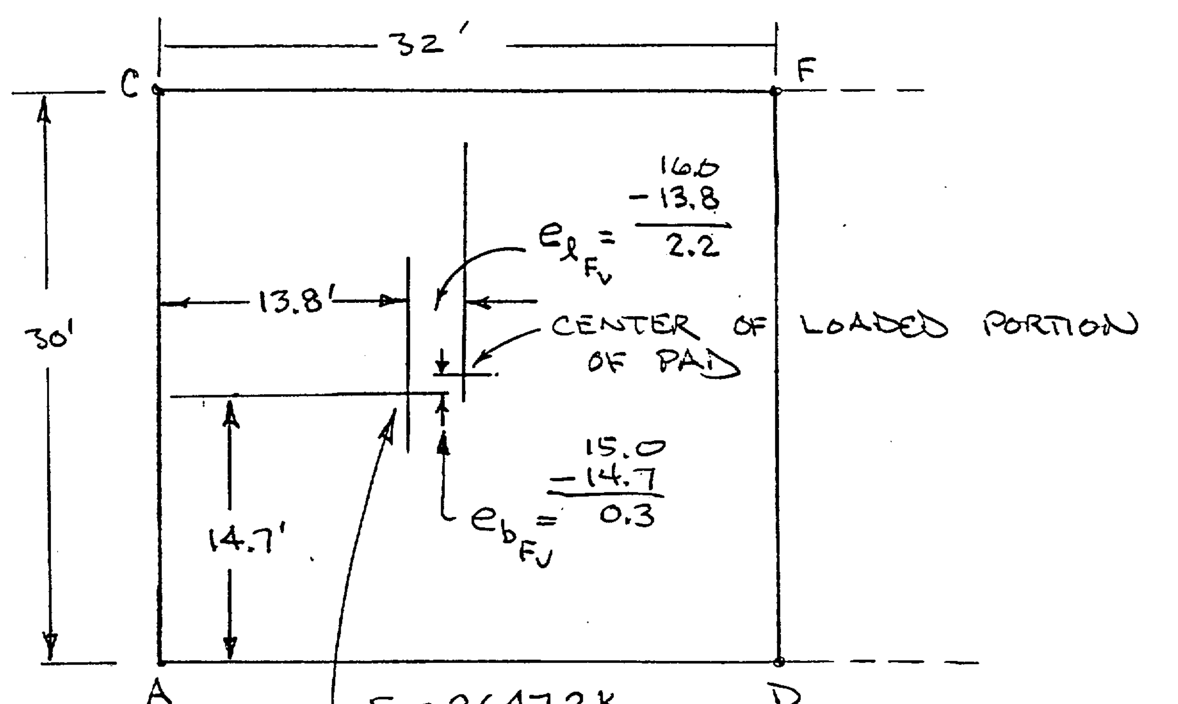
DYN BEARING CAPACITY OF PAD : 2-CASK CASE

ΣM_A

	AREA (K/FT)	MOMENT ARM (FT)	MOMENT K-FT/FT
1	$\frac{1}{2}$ 0.57 KSF × 15' = 4.28	$\frac{1}{3} \cdot 15' = 5'$	21.40
2	3.73 KSF × 30' = 111.90	$\frac{1}{2} \cdot 30' = 15'$	1678.50
3	$\frac{1}{2}$ 0.17 KSF × 15' = 1.28	$15' + \frac{1}{3} 15 = 20'$	25.60

$\Sigma F_V = R = 117.45$ K/FT — 1725.50

$$\therefore \quad x = \frac{\Sigma M_A}{\Sigma F_V} = \frac{1725.50 \text{ K-FT/FT}}{117.45 \text{ K/FT}} = 14.69'$$



$F_V = 2647.2$ K

POINT OF APPLICATION OF F_V DUE TO PAD (DL+EQ) & CASKS (LL+EQ) ~~00800~~ FOR 2-CASK CASE

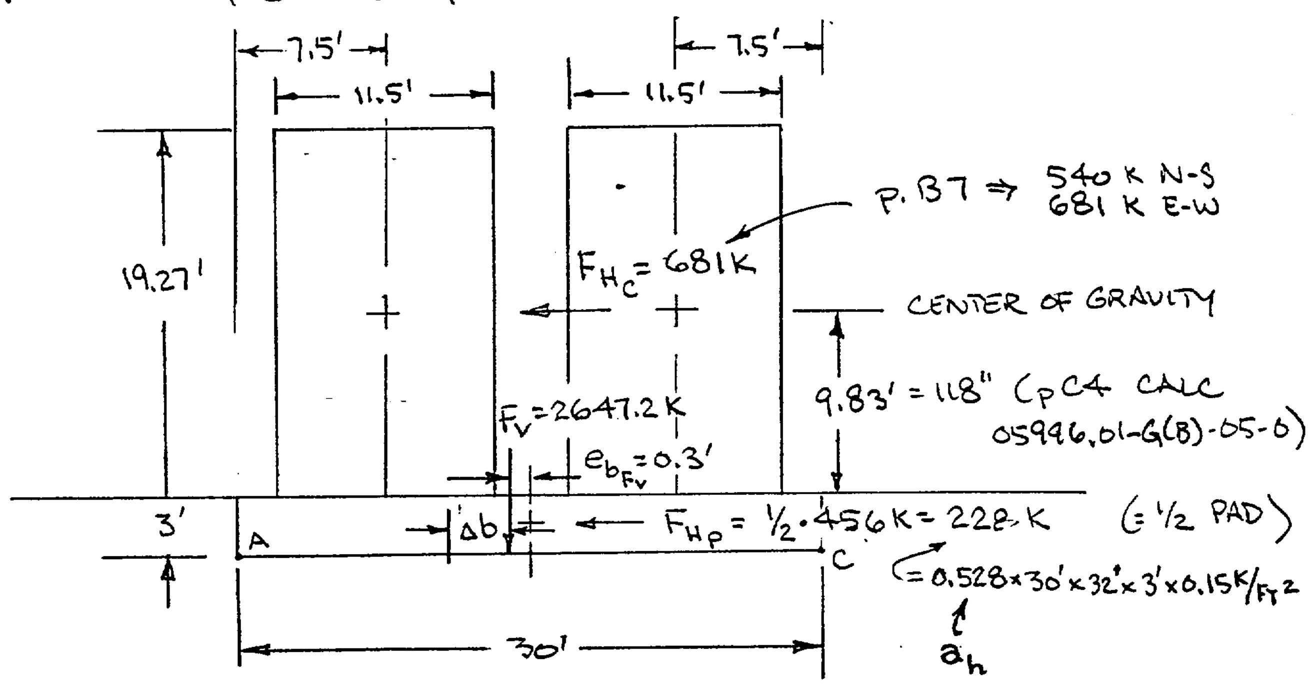
STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

10	CALCULATION IDENTIFICATION NUMBER			PAGE 51
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 2-CASK CASE



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

$$R \Delta b = 1.5' \times \overset{\text{½ PAD *}}{\frac{456^K}{2}} + (3' + 9.83') \overset{\text{2-CASKS TRANSVERSE LOADING (E-W)}}{681 \text{ K}}$$

$R = F_V = 2647.2 K$

$$\therefore \Delta b = \frac{342.0 + 8737.2}{2647.2 \text{ K}} \text{ K-FT} = 3.43 \text{ FT}$$

ADD $e_{b_{F_V}} = 0.3 \Rightarrow e_b = 3.43' + 0.3' = 3.73'$

$$B' = B - 2e_b = 30' - 2 \times 3.73' = 22.54'$$

* NOTE: HORIZ INERTIA OF OTHER ½ OF PAD $\left(\frac{228K}{30' \times 32'} = 0.24 \text{ KSF} \right)$ IS RESISTED BY C = 1.4 KSF & N tan φ w/ φ = 21.3° ALONG BASE OF THAT ½ OF PAD

~~00870~~

DYN BEARING CAPACITY OF PAD: 2-CASK CASE

CALCULATE L′ SIMILARLY FOR LONGITUDINAL DIRECTION.

$F_{H_{C_L}} = 540\ K$ $(= Q_{yd\,max}$ FROM p 87 FOR 2 CASKS$)$

ΣM_{F_V}

2-CASKS LONGITUDINAL LOADG

$R\,\Delta l = \underbrace{1.5' \times \frac{456\,k}{2}}_{\text{1/2 PAD}} + (3' + 9.83')(540\ K)$

$R = F_V = 2647.2\,K$

$\therefore \Delta l = \dfrac{342.00\ K\text{-}FT + 6928\ K\text{-}FT}{2647.2\ K} = 2.75\ FT$

ADD $e_{l_{F_V}} = 2.2' \Rightarrow e_l = 2.75' + 2.2' = 4.95'$

$L' = L - 2e_l = 32' - 2 \times 4.95' = 22.11'$ ← $< 22.54'$ $\therefore$ THIS $= B'$ & $L' = 22.54'$

$q_{ACTUAL} = \dfrac{F_V}{B' \times L'} = \dfrac{2647.2\ K}{22.11' \times 22.54'} = 5.31\ KSF$

CALC q_{ALLOW} FOR THE FOLLOWING: $B' = 22.1'$ $L' = 22.5'$

$F_H = 681\ K + 228\ K = 909\ K$ MAX vs 540K N-S + 228K PAD = 768K N-S

2-CASK EQ_h (p 87) ↑ 1/2 PAD EQ_h

FS = 1.1

$F_V = 2647.2\ K$ FOR 2-CASK (STATIC + DYN)

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

$D_f = 3' - \frac{3.5''}{12''/'} = 2.7'$ (TOP OF PAD 3.5″ ABOVE GRADE)

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

6

CALCULATION IDENTIFICATION NUMBER				PAGE 53
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

Dynamic Bearing Capacity of the Cask Storage Pads Based on Maximum Cask Dynamic Forces from the SSI Analysis

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 2 CASKS

PSHA 2,000-Yr Earthquake: Case IV — **100 % in X, 100 % in Y, 100 % in Z**

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:

$B' =$ **22.1** Footing Width - ft (E-W) $L' =$ **22.5** Length - ft (N-S)

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ **19.0** Angle of load inclination from vertical (degrees)

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

$F_V =$ **2,647** k (Includes EQ_V)

$EQ_{H\,E\text{-}W} =$ **909** k & $EQ_{H\,N\text{-}S} =$ **768** k → **1,190** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.19	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.61	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.87	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.53	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m F_H/B'L'c N_c)$	=	0.68	Eq 3.16a

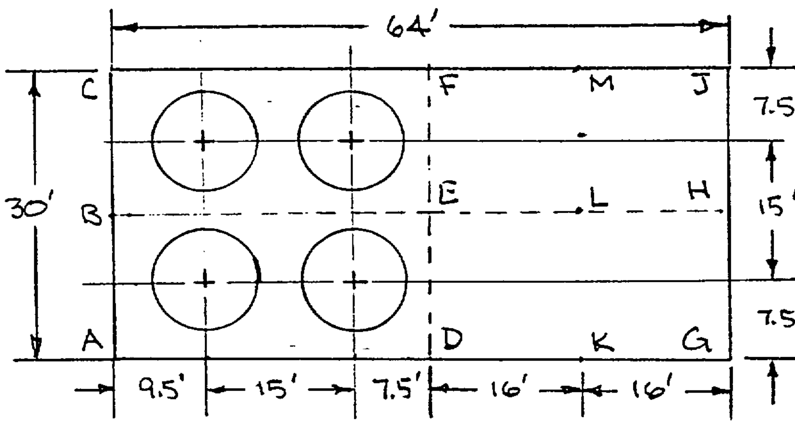
			N_c term		N_q term		N_γ term
Gross q_{ult} =	**9,824**	**psf =**	**9,554**	**+**	**271**	**+**	**0**
q_{all} =	**8,930**	**psf = q_{ult} / FS**					
q_{actual} =	**5,323**	**psf = $(F_v + EQ_v)/(B' \times L')$**					
FS_{actual} =	**1.85**	**= q_{ult}/q_{actual}**					

> 1.1 Hence OK

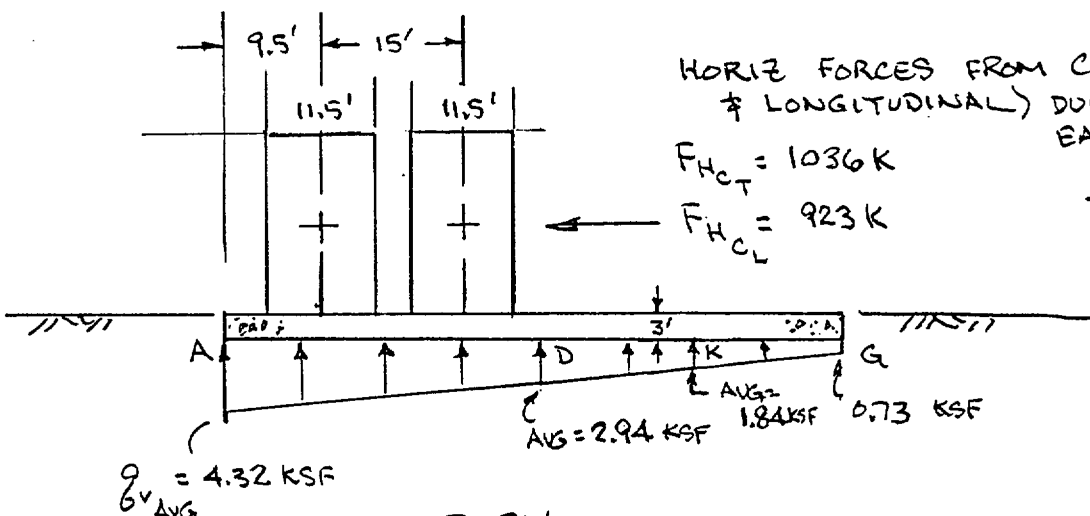
[geot]j05996\calc\brng_cap\Pad\cu_phi.xls

CALCULATION IDENTIFICATION NUMBER				PAGE 54
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 4-CASK CASE



PLAN



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

$F_{H_{C_T}}$ = 1036 K

$F_{H_{C_L}}$ = 923 K

TABLE D-1(c) CEC (1999)

ELEV

STRESSES AT PAD/SOIL INTERFACE FROM CEC (1999)
SEE ATTACHMENT B AND NEXT 2 PAGES.

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CALCULATION IDENTIFICATION NUMBER A				PAGE 55
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

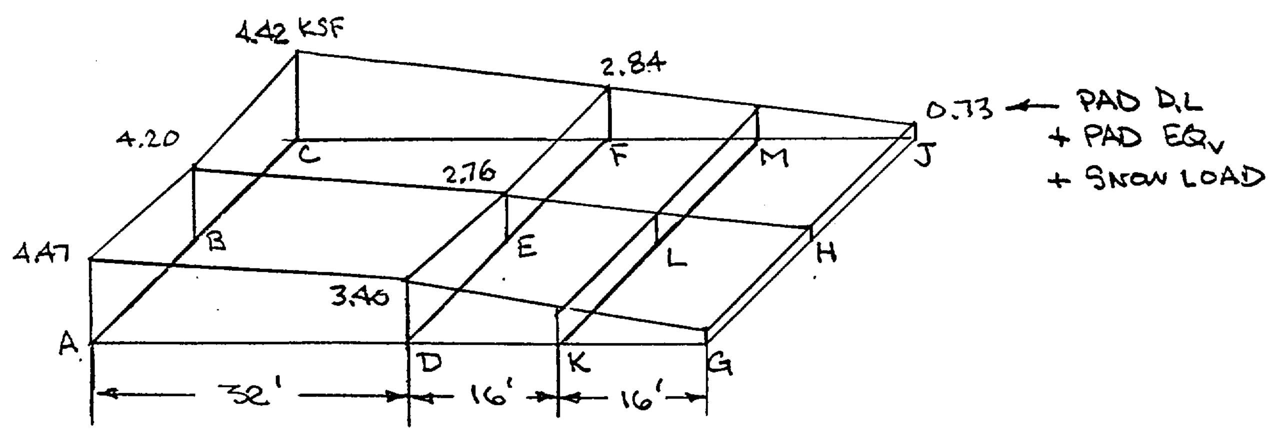
SOIL BEARING PRESSURES ARE BASED ON INFO FROM CEC (1999) INCLUDED IN ATT B AND ARE SUMMARIZED IN TABLE 1.

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

LL OF CASKS = 1.77 KSF ALONG LINE AC & IS ASSUMED TO DECREASE LINEARLY TO 0 ALONG LINE GJ.

CASK EQ PRESSURES ARE SHOWN ON TABLE 1

RESULTING PRESSURE DISTRIBUTION:



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

$\therefore \quad B = 30' \qquad L = \frac{3}{4}\,64 = 48'$

LINEARLY DISTRIBUTE STATIC + DYN LOADING FROM LINE DF TO 48' AWAY FROM LINE AC & DETERMINE FV

VERT STRESSES		KSF	POINT
0.5 (3.40 + 0.73)	=	2.07	K
0.5 (2.76 + 0.73)	=	1.75	L
0.5 (2.84 + 0.73)	=	1.79	M

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B CALCULATION IDENTIFICATION NUMBER				PAGE 56
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

CALCULATE F_V

ALONG LINE	AREA = K/FT	
AC	$\frac{15'}{2}(4.47 + 2\times4.20 + 4.42)$ KSF	= 129.68 K/FT
DF	$\frac{15'}{2}(3.40 + 2\times2.76 + 2.84)$	= 88.20
KM	$\frac{15'}{2}(2.07 + 2\times1.75 + 1.79)$	= 55.20

$$F_V \sim \frac{32'}{2}(129.68 + 88.20)^{K}/_{1} + \frac{16'}{2}(88.20 + 55.20)^{K}/_{1}$$

$$F_V = 3486.1\ K + 1147.2\ K = \underline{4633.3\ K}$$

ESTIMATE LOCATION WHERE F_V ACTS ON 30'×48' PORTION OF PAD.

NOTE AVG VERT STRESS ALONG LINES.

LINE

$$AC = \frac{129.68\ K/FT}{30\ FT} = 4.32\ KSF$$

$$DF = \frac{88.20\ K/FT}{30'} = 2.94\ KSF$$

$$KM = \frac{55.20\ K/FT}{30'} = 1.84\ KSF$$

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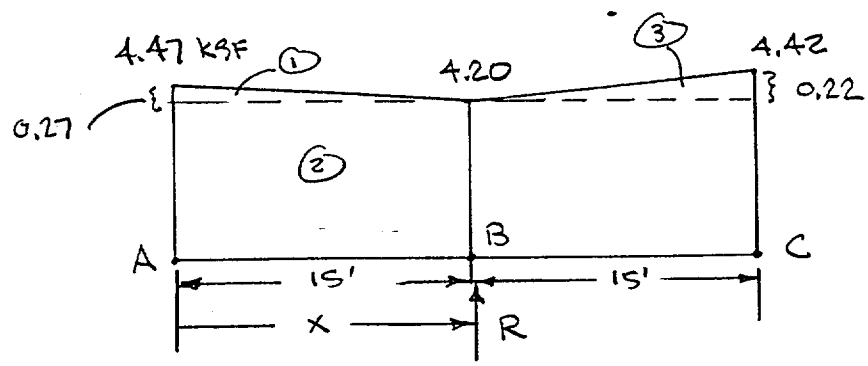
CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	PAGE 57

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

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

ALONG LINE AC



ΣM_A

AREA	K/FT	MOMENT ARM (FT)	MOMENT
①	$\frac{1}{2} \times 0.27 \frac{K}{FT^2} \times 15' = 2.03$	$\frac{1}{3} \times 15' = 5'$	10.13
②	$4.20 \frac{K}{FT^2} \times 30' = 126.60$	$\frac{1}{2} \times 30' = 15'$	1890.0
③	$\frac{1}{2}\, 0.22 \frac{K}{FT^2} \times 15' = 1.65$	$15' + \frac{2}{3} 15' = 25'$	41.3
	$F_V = \Sigma = 129.68\ K/FT$		$RX = \Sigma = 1941.4$

$$\therefore \quad X = \frac{1941.4 \ \ K\text{-}FT/FT}{129.68 \ \ K/FT} = 15.0'$$

$$e_{B_{F_V}} = \frac{B}{2} - X = 15' - 15.0' = 0'$$

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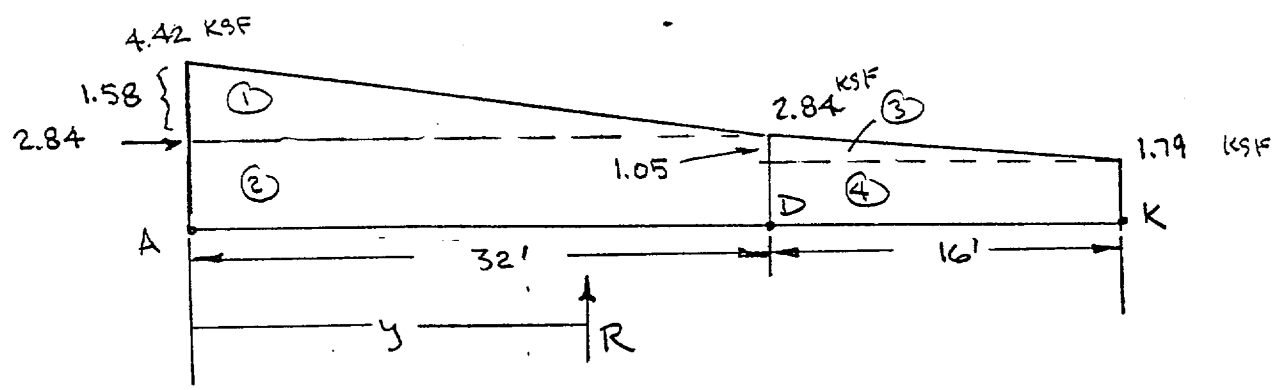
5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 58
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 4-CASK 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	$\frac{1}{2}$ 1.58 × 32' = 25.28	$\frac{1}{3}$ × 32' = 10.67'	269.7
②	2.84 × 32 = 90.88	$\frac{1}{2}$ × 32' = 16'	1454.0
③	$\frac{1}{2}$ 1.05 × 16 = 8.40	32' + $\frac{1}{3}$ 16' = 37.33'	313.6
④	1.79 × 16 = 28.64	32' + $\frac{16'}{2}$ = 40'	1145.6

$R = \Sigma F_v = 153.20$ K/' $Ry = \Sigma M = 3182.9$

$$y = \frac{\Sigma M}{\Sigma F_v} = \frac{3182.9 \text{ K-FT/LF}}{153.20 \text{ K/LF}} = 20.78 \text{ FT}$$

$$e_{L_{F_v}} = \frac{L}{2} - y = 24.0' - 20.78' = 3.22'$$ AS SHOWN ON NEXT PAGE

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CALCULATION SHEET

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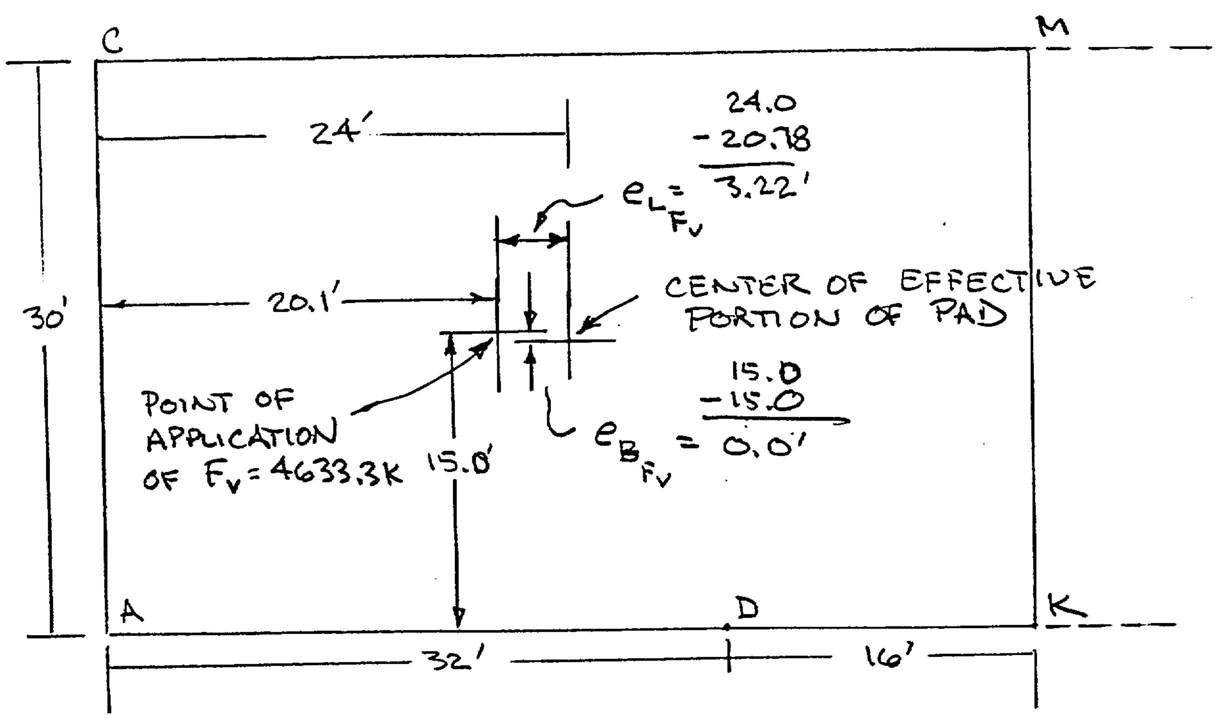
CALCULATION IDENTIFICATION NUMBER D				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 59
05996.02	G(B)	04-6		

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



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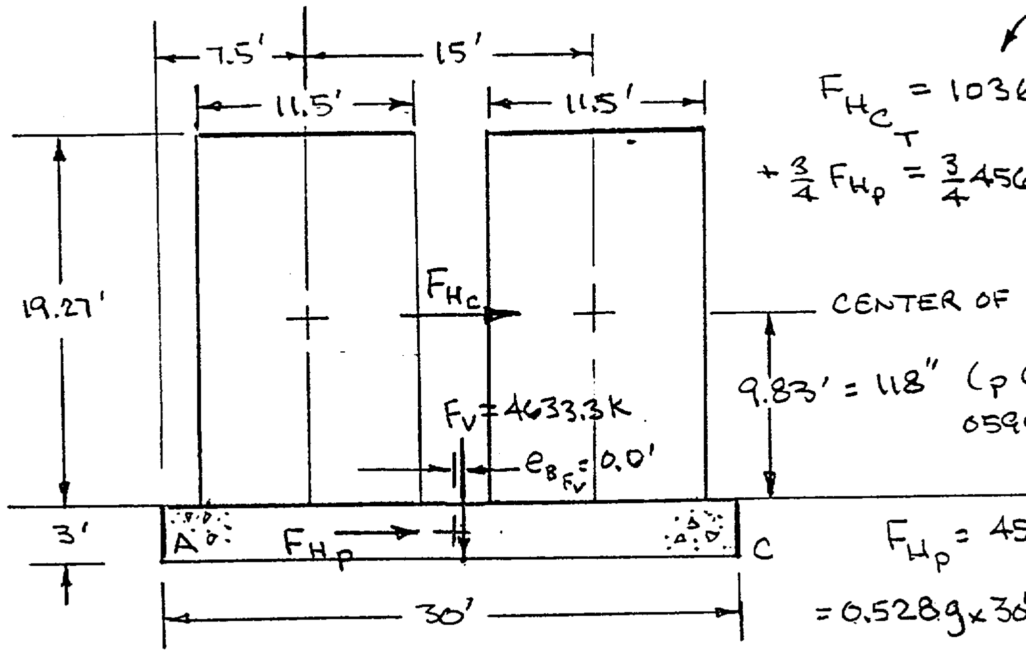
CALCULATION SHEET

▲ 5010.65

E — CALCULATION IDENTIFICATION NUMBER				PAGE 60
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 4-CASK CASE



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

$F_{H_{C_T}} = 1036K$ (p B7)

$+ \frac{3}{4} F_{H_P} = \frac{3}{4} 456 \Rightarrow 1378k = EQ_{H\ E\text{-}W}$

CENTER OF GRAVITY OF CASK

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

$F_{H_P} = 456^K$ FOR 30'×64'

$= 0.528g \times 30' \times 64' \times 3' \times 0.15^{K/FT^3} / g$

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

$$R\,\Delta b = 1.5' \times \underbrace{\tfrac{3}{4} \times 456^K}_{3/4\ \text{PAD}} + (3' + 9.83')\ \underbrace{1036K}_{\text{4-CASK TRANSVERSE } F_H}$$

$R = F_V = 4633.3^K$ ON 30'×48' PORTION OF PAD

$$\therefore\ \Delta b = \frac{513.2 + 13{,}292\ \text{K-FT}}{4633.3\ K} = 2.98' = e_{B_{F_H}}$$

$$e_b = e_{B_{F_V}} + e_{B_{F_H}} = 0.0' + 2.98' = 2.98'$$

$$B' = B - 2e_B = 30' - 2 \times 2.98' = \underline{24.04'}$$

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CALCULATION SHEET

▲ 5010.65

F CALCULATION IDENTIFICATION NUMBER				PAGE 61
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

CALCULATE L' SIMILARLY FOR LONGITUDINAL F_H

$F_{H_{C_L}} = 923\ K = Q_{yd_{MAX\ 4\ CASKS}}$ TABLE D-1(6) P B7

$+\frac{3}{4} F_{HP} = \frac{3}{4} 456 \Rightarrow EQ_{H\ N\text{-}S} = 1265K$

$F_{H_{C_L}}$ 4 CASKS

ΣM_{F_V} $\quad R\,\Delta l = 1.5' \times \frac{3}{4} \times 456\ K + (3' + 9.83')\ 923\ K$

$\quad R = F_V = 4633.3K$ ON EFFECTIVE PORTION OF PAD (30' x 48')

$$\Delta l = \frac{513.2\ K\text{-}FT + 11{,}842.1\ K\text{-}FT}{4633.3\ K} = 2.67' = e_{L_{F_H}}$$

$$e_L = e_{L_{F_V}} + e_{L_{F_H}} = 3.22' + 2.67' = 5.89'$$

$$L' = L - 2e_L = 48' - 2 \times 5.89' = 36.23'$$

$$q_{ACTUAL} = \frac{F_V}{B' \times L'} = \frac{4633.3K}{24.04' \times 36.23'} = \underline{\underline{5.32\ KSF}}$$

CALC q_{ALLOW} FOR THE FOLLOWING: $B' = 24.04'$ $L' = 36.23'$

$F_V = 4633.3\ K$ FOR 4-CASK CASE (STATIC + DYN)

$$EQ_{H\ E\text{-}W} = \overset{PAD}{\frac{3}{4} 456} + \overset{F_{HC}}{1036} = 1378\ K\ E\text{-}W$$

$$EQ_{H\ N\text{-}S} = \text{"} + 923 = 1265\ N\text{-}S$$

$FS = 1.1$ $\quad \gamma_{SURCH} = 100\ PCF$ $\quad \gamma = 80\ PCF$ $\quad D_f = 3' - \frac{3.5''}{12''/'} = 2.7'$

$\phi = 0°$ $\quad c = 2.2\ KSF$

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CALCULATION IDENTIFICATION NUMBER				PAGE 62
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

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

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 4 CASKS

PSHA 2,000-Yr Earthquake: Case IV — **100 % in X, 100 % in Y, 100 % in Z**

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:

$B' =$ **24.0** Footing Width - ft (E-W) $L' =$ **36.2** Length - ft (N-S)

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ **16.6** Angle of load inclination from vertical (degrees)

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

$F_V =$ **4,633** k (Includes EQ_V)

$EQ_{H\ E\text{-}W} =$ **1,378** k & $EQ_{H\ N\text{-}S} =$ **1,265** k → **1,871** k for F_H

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

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

$N_c = (N_q - 1)\ \cot(\phi)$, but $= 5.14$ for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\,(N_q + 1)\ \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.13	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.73	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\,D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.83 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.52	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 $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.71	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	9,773	psf =	9,503	+	271	+	0
q_{all} =	8,880	psf = q_{ult}/FS					
q_{actual} =	5,320	psf = $(F_v + EQ_v)/(B' \times L')$					
FS_{actual} =	1.84	= q_{ult}/q_{actual}			> 1.1 Hence OK		

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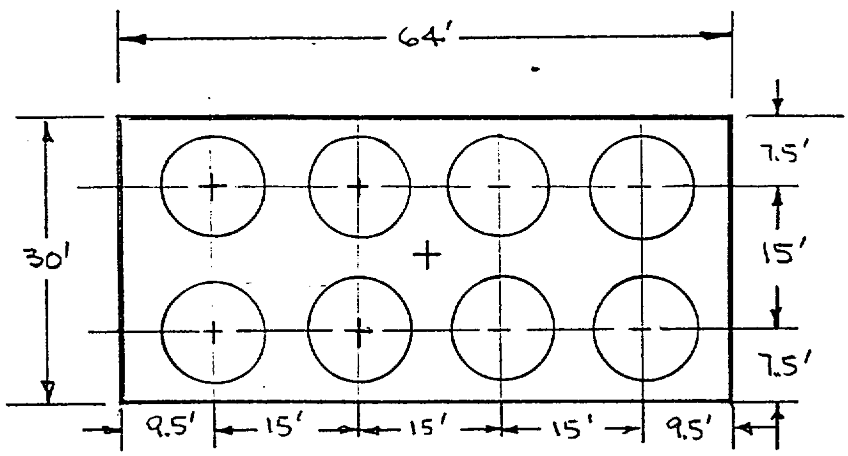
CALCULATION SHEET

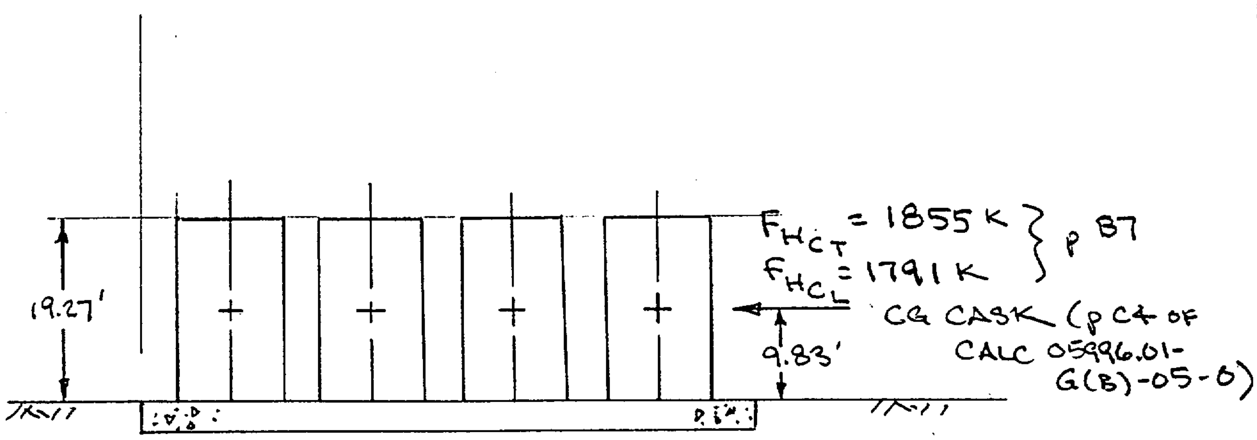
▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 63
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 8-CASK CASE



PLAN



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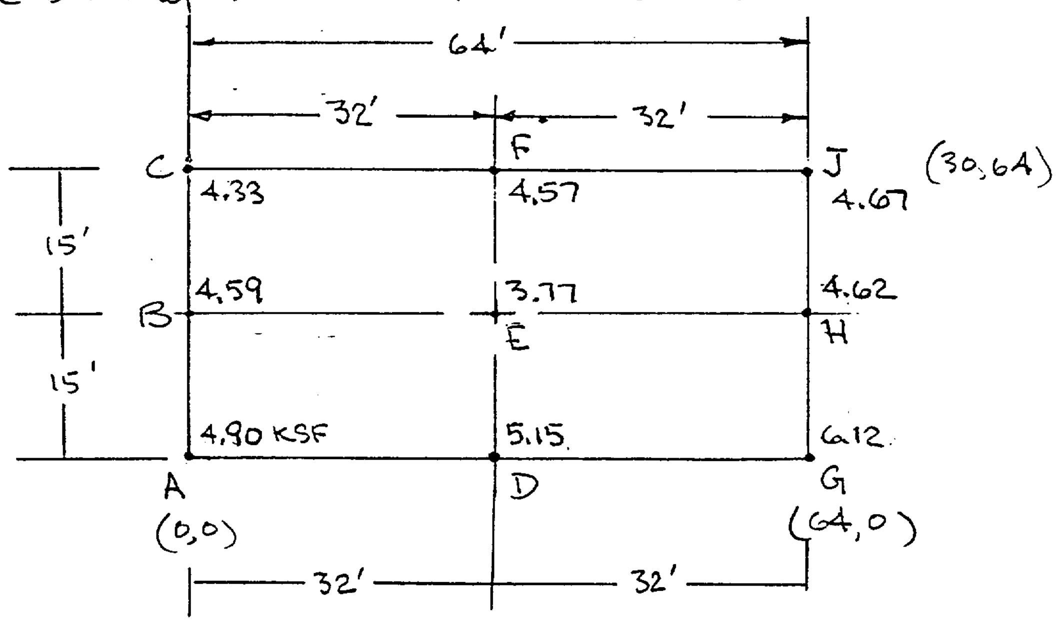
CALCULATION SHEET

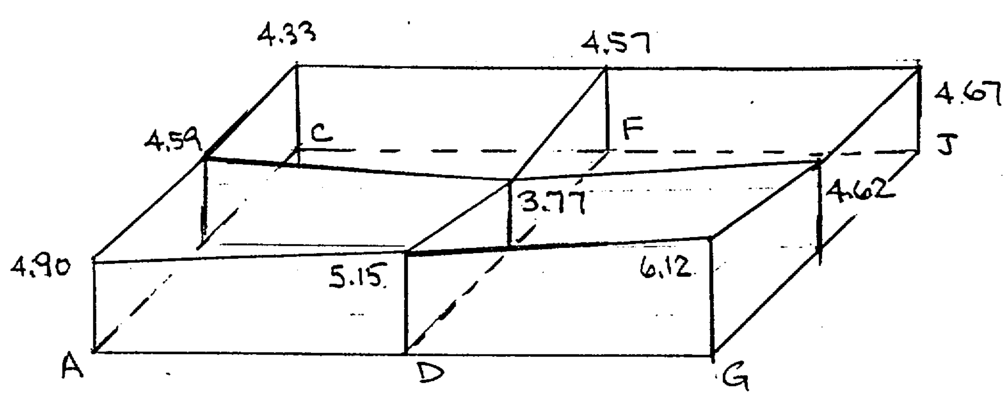
▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 64
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(8)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 8-CASK CASE

SOIL BEARING PRESSURES FROM TABLE 1





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CALCULATION SHEET

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CALCULATION IDENTIFICATION NUMBER				PAGE 65
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

DYN BEARING CAPACITY OF PAD: 8-CASK CASE

CALCULATE F_V :

ALONG LINE	AREA = K/FT	F_V (K/FT)	q_{AVG} (KSF)
AC	$\frac{15}{2}(4.90 + 2\times 4.59 + 4.33)$	= 138.08	4.60
DF	$\frac{15}{2}(5.15 + 2\times 3.77 + 4.57)$	= 129.45	4.32
GJ	$\frac{15}{2}(6.12 + 2\times 4.62 + 4.67)$	= 150.23	5.01

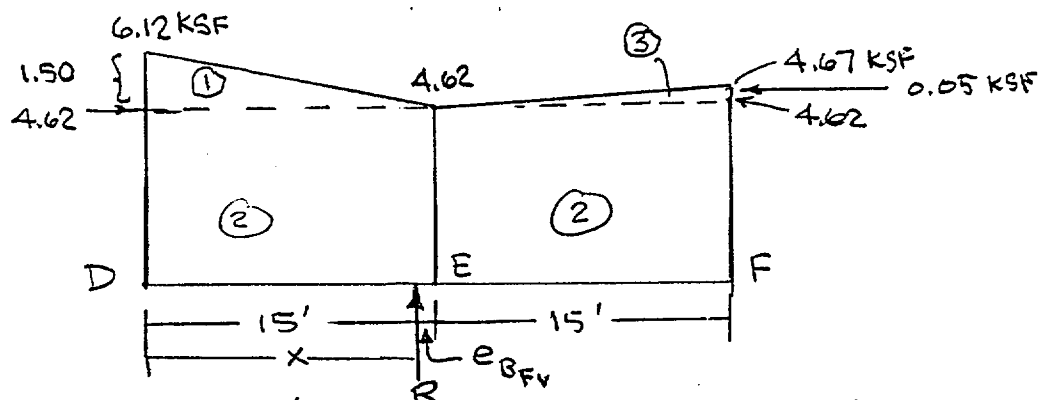
$$F_V \sim \frac{32'}{2}(138.08 + 2\times 129.45 + 150.23) = \underline{8755\ K}$$

ESTIMATE LOCATION WHERE F_V ACTS.

DETERMINE ECCENTRICITY OF F_V WRT B, $e_{BF_V} = \frac{B}{2} - x$

ALONG LINE GJ, WHICH HAS THE GREATEST STRESSES

ΣM_D



AREA	K/FT	MOMENT ARM (FT)	MOMENT K-FT/FT
① $\frac{1}{2}$ 1.50 KSF × 15'	= 11.25	$\frac{1}{3}\times 15' = 5'$	56.25
② 4.62 × 30'	= 138.60	$\frac{1}{2}\times 30' = 15'$	2079.00
③ $\frac{1}{2}$ 0.05 × 15'	= 0.38	$15' + \frac{2}{3}\times 15' = 25'$	9.38
	$R = \Sigma = 150.23$		$Rx = \Sigma = 2144.63$ ~~00886~~

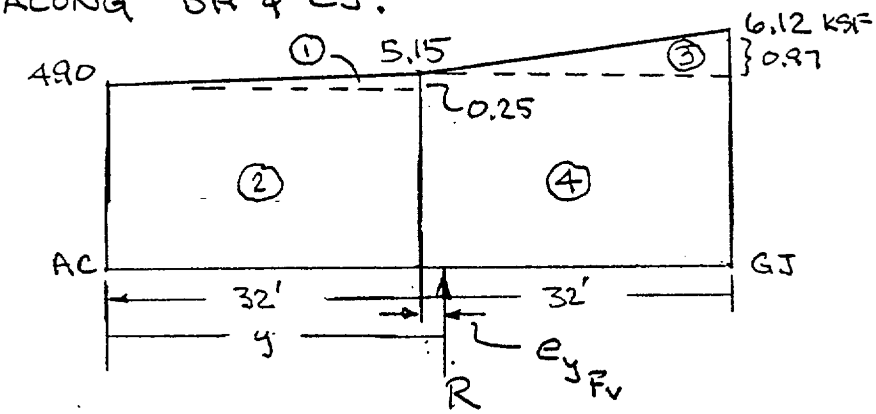
$$\therefore x = \frac{Rx}{R} = \frac{2144.63\ \text{K-FT/FT}}{150.23\ \text{K/FT}} = 14.28'$$

$$e_{BF_V} = \frac{30}{2} - 14.28' = 0.72'$$

CALCULATION IDENTIFICATION NUMBER				PAGE 66
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-6		

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 $\frac{K\text{-}FT}{FT}$
① $\frac{1}{2}(0.25\text{ KSF}) \times 32' =$	4.0	$\frac{2}{3} \times 32' = 21.33'$	85.33
② $4.90\text{ KSF} \times 32' =$	156.80	$\frac{1}{2} \times 32' = 16'$	2508.80
③ $\frac{1}{2}(6.12 - 5.15)^{KSF} \times 32' =$	15.52	$32' + \frac{2}{3}32' = 53.33'$	827.68
④ $5.15\text{ KSF} \times 32' =$	164.80	$32' + \frac{1}{2}32' = 48.0'$	7910.40
	R = 341.12 K/FT		$Ry = 11{,}332.2 \frac{K\text{-}FT}{FT}$

$$\therefore y = \frac{Ry}{R} = \frac{11{,}332.2\ K\text{-}FT/FT}{341.12\ K/FT} = 33.22'$$

$$e_{L_{F_V}} = y - \frac{L}{2} = 33.22' - \frac{64'}{2} = 1.22'$$

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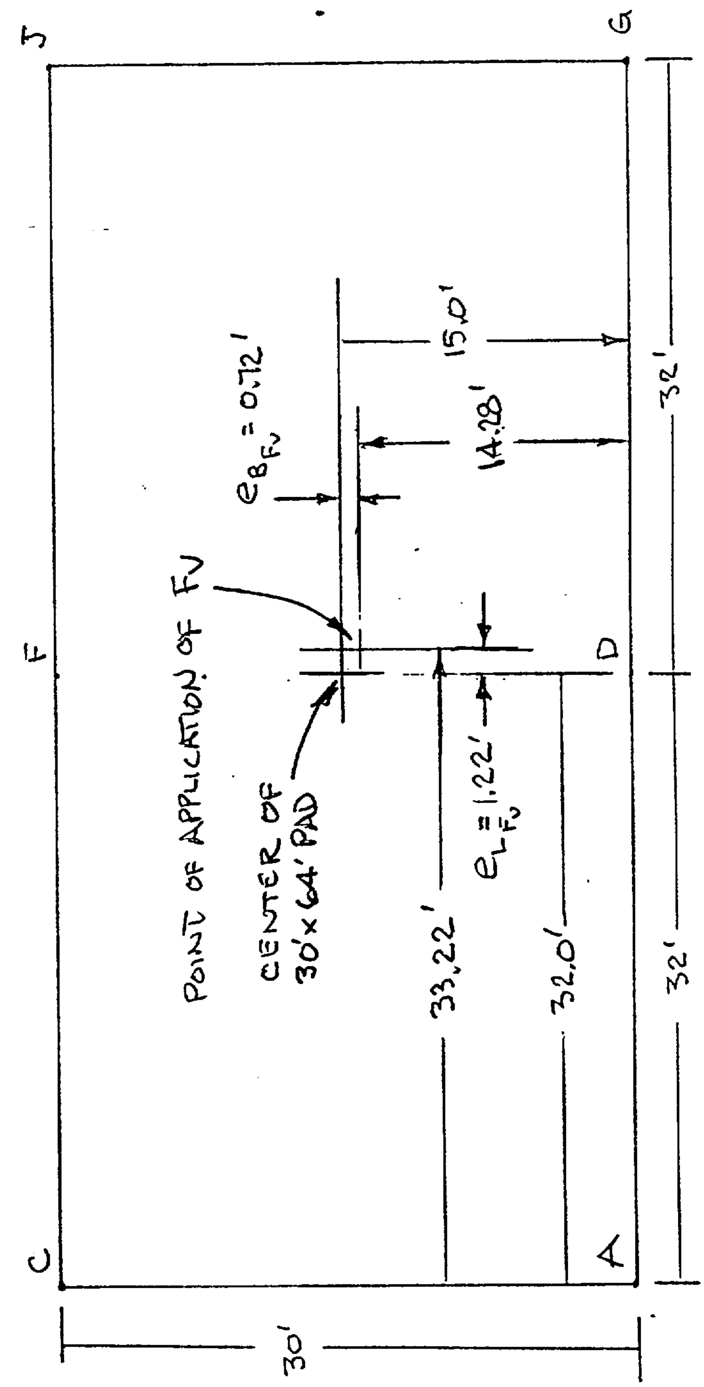
CALCULATION SHEET

▲ 5010.65

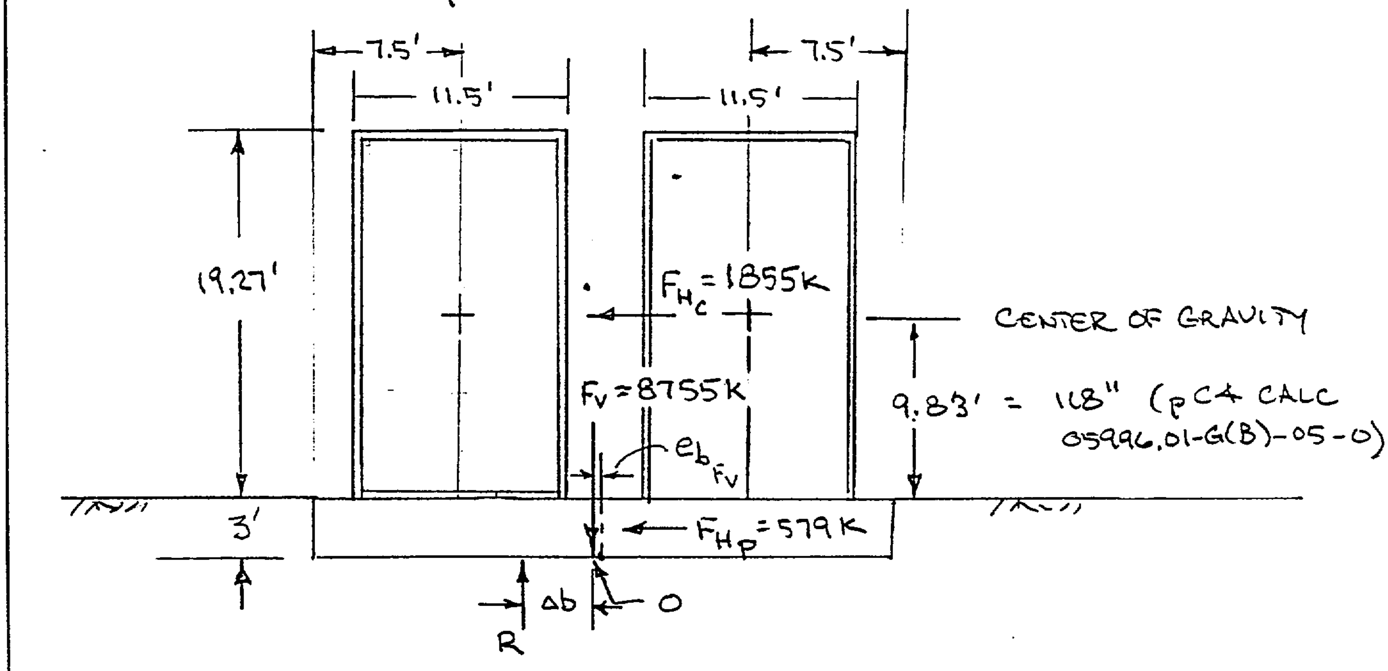
CALCULATION IDENTIFICATION NUMBER				PAGE 67
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 04-6	OPTIONAL TASK CODE	

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



~~00888~~

DYN BEARING CAPACITY OF PAD: 8-CASK CASE



F_{H_C} = 1855 K FOR 8-CASK CASE (p B7)
TRANVERSE DIRECTION E-W

$F_{H_P} = 0.528\,g \times 30' \times 64' \times 3' \times \frac{0.15\ \text{KCF}}{g} = 456\ \text{K}$ (a_h = 0.528 g)

$\Sigma = EQ_{H_{E\text{-}W}} = 2311\ \text{K}$

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

$\underset{=F_V}{R}\,\Delta b = \underbrace{1.5' \times 456\ \text{K}}_{\text{PAD}} + \underbrace{(3' + 9.83')\ 1855\text{K}}_{\text{CASKS}}$

$\Delta b = \dfrac{684 + 23{,}800\ \text{K-FT}}{8755\ \text{K}} = 2.80\ \text{FT}$

ADD $e_{b_{F_V}} = 0.72$ FT $\Rightarrow$ $e_b = 2.80 + 0.72 = 3.52$ FT

$B' = B - 2e_b = 30' - 2 \times 3.52' = 22.97$ FT

~~00889~~

3 CALCULATION IDENTIFICATION NUMBER				PAGE 69
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-6		

DYN BEARING CAPACITY OF PAD: 8-CASK CASE

SIMILARLY FOR LONGITUDINAL DIRECTION

$F_{H_C} = 1791$ K ADD $F_{HP} = 456$K $\Rightarrow EQ_{H_{N-S}} = 2247$K
℄ p 87

ΣM_o $\overset{R=F_V}{8755 \text{ K}}\ \Delta l = \overset{\text{PAD}}{1.5' \times 456\text{K}} + \overset{\text{CASKS}}{(3'+9.83')(1791\text{K})}$

$$\Delta l = \frac{684 + 22{,}979 \text{ K-FT}}{8755 \text{ K}} = 2.70'$$

ADD $e_{l_{F_v}} = 1.22$ FT $\Rightarrow e_l = 2.70' + 1.22' = 3.92'$

$L' = L - 2e_l = 64' - 2 \times 3.92' = 56.15$ FT

$$q_{ACTUAL} = \frac{F_V}{B' \times L'} = \frac{8755\text{K}}{22.97' \times 56.15'} = 6.79 \text{ KSF}$$

CALC q_{ALLOW} FOR FS = 1.1 $B' = 22.97'$ $L' = 56.15'$

$F_V = 8755$ K (STATIC + DYN 8 CASKS)

$EQ_{H_{E-W}} = \overset{F_{HP}}{456\text{K}} + \overset{F_{HC}}{1855\text{K}} = 2311$K

$EQ_{H_{N-S}} = 456\text{K} + 1791\text{K} = 2247$K

$\gamma_{SURCH} = 100$ PCF $\gamma = 80$ PCF $D_f = 3' - \frac{3.5''}{12''/'} = 2.7'$

$\phi = 0°$ C = 2.2 KSF

$\boxed{6}$

~~00890~~

CALCULATION IDENTIFICATION NUMBER				PAGE 70
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

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

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 8 CASKS

PSHA 2,000-Yr Earthquake: Case IV — **100 % in X, 100 % in Y, 100 % in Z**

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:

- $B' =$ **23.0** Footing Width - ft (E-W) $L' =$ **56.2** Length - ft (N-S)
- $D_f =$ **2.7** Depth of Footing (ft)
- $\beta =$ **14.8** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **8,755** k (Includes EQ_V)
- $EQ_{H\,E\text{-}W} =$ **2,311** k & $EQ_{H\,N\text{-}S} =$ **2,247** k → **3,223** k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, but $= 5.14$ for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\,(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.08	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.84	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q) / (N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L) / (1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B) / (1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W} / EQ_{H\,N\text{-}S})$	=	0.80 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.51	Eq 3.18c
$i_q = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.67	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	8,802	psf =	8,531	+	271	+	0
q_{all} =	8,000	psf = q_{ult} / FS					
q_{actual} =	6,788	psf = $(F_v + EQ_v) / (B' \times L')$					
FS_{actual} =	1.30	= q_{ult} / q_{actual}			> 1.1 Hence OK		

[geot]j05996\calc\brng_cap\Pad\cu_phi.xls

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, 1999) for the pad supporting 2 casks, 4 casks, and 8 casks. Details of these analyses are presented on the preceding pages. 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 8.0 ksf for the 2-cask, 4-cask, and 8-cask loading cases. The minimum allowable value (8.0 ksf) was obtained for the 8-cask loading. The actual factor of safety for this case was 1.3, which is greater than the criterion for dynamic bearing capacity (FS $\geq$ 1.1).

CONCLUSIONS

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

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

For Case II, 100% of the dynamic lateral forces in both 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); 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 1.66. 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 described in Sections 2.6.1.7 and 2.6.4.11 of the SAR and as illustrated in Figure 4.2-7 of the SAR. Analyses presented above demonstrate that, using only the passive resistance of the soil cement above the bottom of the pads, the soil cement can be designed to provide sufficient resistance to sliding of the pads to readily achieve the minimum required factor of safety of 1.1. Thus, embedding the pads in soil cement will greatly enhance their resistance to sliding due to dynamic loads from the design basis ground motion. Additional analyses are included that demonstrate that sliding will not occur along deeper surfaces within the profile underlying the cask storage pads. First, the sliding resistance of the *in situ* silty clay/clayey silt layer is addressed to demonstrate that sliding will not occur along the interface between the bottom of the soil cement and those soils. These analyses demonstrate that if the pads were founded directly on the silty clay/clayey silt layer, the minimum factor of safety against sliding would be ~1.7. Therefore, the cask storage pads, embedded in soil cement, will have an adequate factor of safety against sliding.

Adequate factors of safety against sliding due to maximum forces from the design basis ground motion were obtained assuming that the storage pads were founded directly on the silty clay/clayey silt layer and conservatively ignoring the passive resistance of the soil cement that will be placed under and adjacent to the pads. In this case, much of the shearing resistance is provided by the cohesive portion of the 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, the sliding stability of the cask storage pads was analyzed assuming 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.

Analyses were performed to 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 (0.53g) due to the design basis ground motion at this site, the frictional resistance available 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 less than ½ inch. 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. Furthermore, the pads will be constructed on and within soil cement, which will be strong enough to resist sliding of the pads using only the passive resistance of the soil cement. This soil cement will effectively lock the pads in their respective locations, so that they can not move relative to one another.

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 6.3 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 14 when the effective-stress strength of $\phi = 30°$ is used. Therefore, cases result in factors of safety against a bearing capacity failure that exceed the minimum allowable value of 3 for static loads. The minimum gross allowable bearing capacity exceeds 4 ksf 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, 1999) for the pad supporting 2 casks, 4 casks, and 8 casks. These latter 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.

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 7.7 ksf for all loading cases identified above. The minimum allowable value was obtained for Load Case II, wherein 100% of the earthquake loads act in the N-S and E-W directions and 0% acts in the Vertical direction, tending to rotate the cask storage pad about the N-S axis. The actual factor of safety for this condition was 2.3, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

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, 1999) for the pad supporting 2 casks, 4 casks, and 8 casks. 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 8 ksf for the 2-cask, 4-cask, and 8-cask loading cases. The minimum allowable value (8.0 ksf) was obtained for the 8-cask loading. The actual factor of safety for this case was 1.3, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

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.

CEC, 1999, PFSF Calculation 05996.02-G(PO17)-2, Rev. 1, "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.

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, 1999b, "Development of Design Ground Motions for the Private Fuel Storage Facility," prepared for Stone and Webster Engineering Corp, Denver, CO, 19 pp.

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

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.

SWEC, 1982, Geotechnical Technical Guideline 6.15-1, "Determination of Lateral Pressures on Buried Structures in Granular Soils," Stone & Webster Engineering Corporation, Boston, MA.

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.

SWEC, 2000b, Calculation No. 05996.02-G(B)-13, Revision 2, "Stability Analyses of the Canister Transfer Building Supported on a Mat Foundation," Stone & Webster Engineering Corporation, Boston, MA.

Terzaghi, K., and Peck, R. B., 1967, Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, NY, p 112.

Vesic, A. S., 1973, "Analysis of Ultimate Loads on Shallow Foundations," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol 99, No. SM1, pp 45-73.

Winterkorn, H. F., and Fang, H., 1975, Foundation Engineering Handbook, Van Nostrand Reinhold Co., New York, NY,.

TABLE 1

Summary of Vertical Soil Bearing Pressures (ksf) from Calc 05996.02-G(PO17)-2, Rev 1

(After adjusting snow loads to 0.045 ksf)

Loading	Point	A (287)	B (293)	C (299)	D (144)	E (150)	F (156)	G (1)	H (7)	J (13)
2-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.35	1.36	1.36	0.35	0.35	0.35	0.00	0.00	0.00
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24
	Cask EQ	2.22	1.64	1.81	0.67	0.48	0.45	0.00	0.00	0.00
	100% Vert	**4.30**	**3.73**	**3.90**	**1.75**	**1.56**	**1.53**	**0.73**	**0.73**	**0.73**
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	1.77	1.77	1.77	0.80	0.80	0.80	0.00	0.00	0.00
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24
	Cask EQ	1.97	1.70	1.92	1.87	1.23	1.31	0.00	0.00	0.00
	100% Vert	**4.47**	**4.20**	**4.42**	**3.40**	**2.76**	**2.84**	**0.73**	**0.73**	**0.73**
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	1.47	1.47	1.47	1.60	1.60	1.60	1.47	1.47	1.47
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24
	Cask EQ	2.70	2.39	2.13	2.82	1.44	2.24	3.92	2.42	2.47
	100% Vert	**4.90**	**4.59**	**4.33**	**5.15**	**3.77**	**4.57**	**6.12**	**4.62**	**4.67**

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

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\ E\text{-}W}$	β_L $EQ_{H\ N\text{-}S}$	GROSS		e_B	e_L	EFFECTIVE			FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	q_{ult} ksf	q_{all} ksf	ft	ft	B' ft	L' ft	q_{actual} ksf	
IA - Static Undrained Strength	3,716	0	0	0	0	0.0	0.0	13.05	**4.35**	0.0	0.0	30.0	64.0	**1.94**	**6.7**
IB - Static Effective Strength	3,716	0	0	0	0	0.0	0.0	28.34	**9.44**	0.0	0.0	30.0	64.0	**1.94**	**14.6**

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

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

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

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

L = **64** Footing length (ft)

$D_f =$ **2.7** Depth of footing (ft)

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

FS = **3** Factor of safety for static loads.

F_V = Vertical load (Static + EQ_V)

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

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

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

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

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

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

[geotl\05996\calc\brng_cap\Pad\cu_phi.xls Table 2.6-6

TABLE 2.6-7

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Inertial Forces Due to Design Earthquake: PSHA 2,000-Yr Return Period

Case	F_V	$EQ_{H\,N\text{-}S}$	$EQ_{H\,E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\,E\text{-}W}$	β_L $EQ_{H\,N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
II	3,716	1,962	1,962	20,006	20,006	27.8	27.8	8.46	**7.68**	5.4	5.4	19.2	53.2	**3.63**	**2.3**
IIIA	1,735	785	785	8,002	8,002	24.3	24.3	11.39	**10.35**	4.6	4.6	20.8	54.8	**1.52**	**7.5**
IIIB	2,924	785	1,962	20,006	8,002	33.9	15.0	8.92	**8.10**	6.8	2.7	16.3	58.5	**3.06**	**2.9**
IIIC	2,924	1,962	785	8,002	20,006	15.0	33.9	10.52	**9.56**	2.7	6.8	24.5	50.3	**2.37**	**4.4**
IVA	5,697	785	785	8,002	8,002	7.8	7.8	11.91	**10.83**	1.4	1.4	27.2	61.2	**3.42**	**3.5**
IVB	4,508	785	1,962	20,006	8,002	23.5	9.9	9.93	**9.03**	4.4	1.8	21.1	60.5	**3.53**	**2.8**
IVC	4,508	1,962	785	8,002	20,006	9.9	23.5	10.88	**9.89**	1.8	4.4	26.5	55.1	**3.09**	**3.5**

$c =$	**2,200**	Total stress cohesion (psf)	F_V = Vertical load (Static + EQ_V)
$\phi =$	**0.0**	Total stress friction angle (deg)	**EQ_H** = Earthquake: Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$
$B =$	**30**	Footing width (ft)	$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).
$L =$	**64**	Footing length (ft)	$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).
$D_f =$	**2.7**	Depth of footing (ft)	$e_B = \Sigma M_{@N\text{-}S} / F_V$ $e_L = \Sigma M_{@E\text{-}W} / F_V$
$\gamma =$	**80**	Unit weight of soil (pcf)	$B' = B - 2\,e_B$ $L' = L - 2\,e_L$
$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)	$q_{actual} = F_V / (B' \times L')$
$FS =$	**1.1**	Factor of safety for dynamic loads.	

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: 100% N-S, 100% Vertical, and 100% E-W

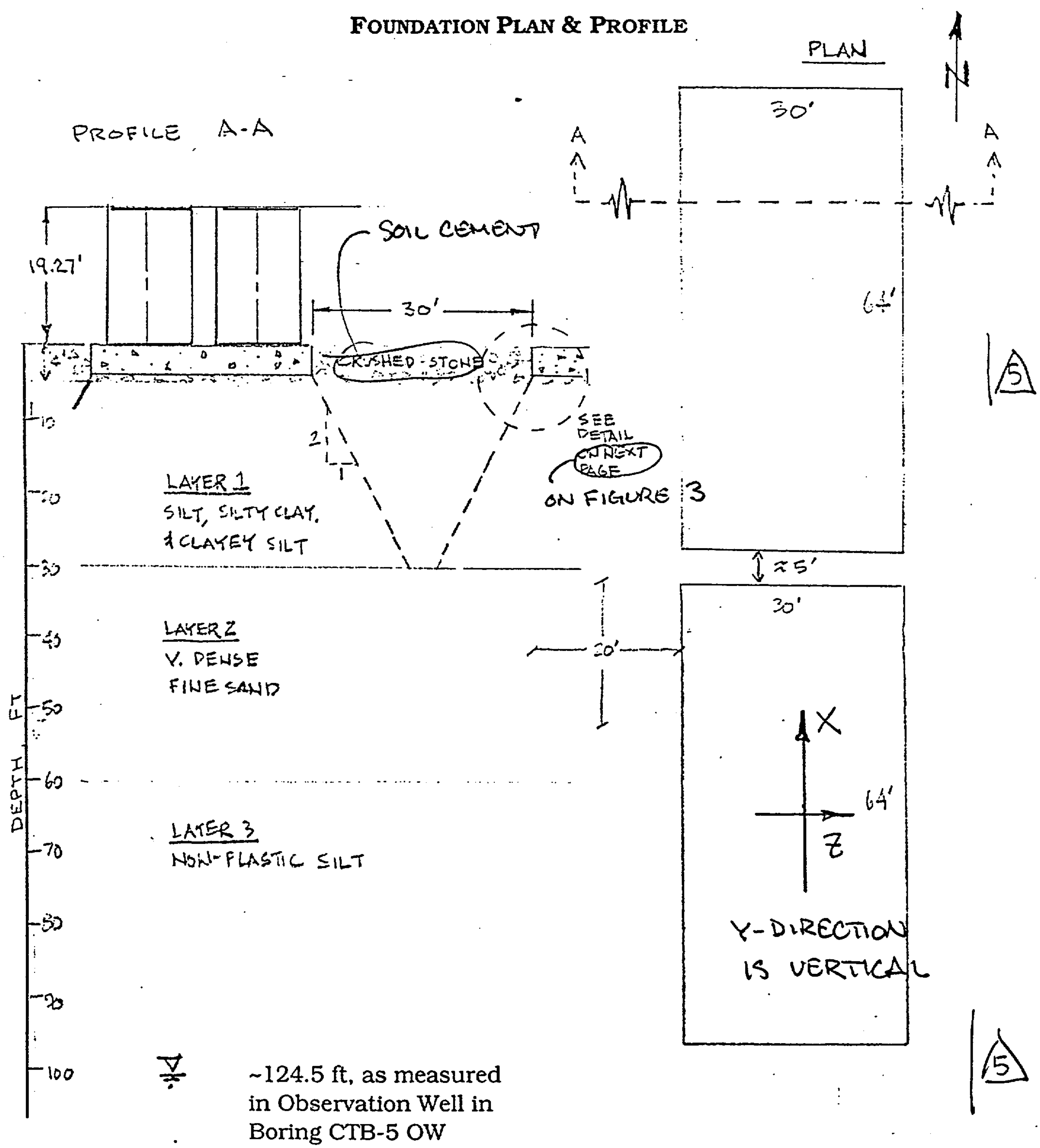
Case IV	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\ E\text{-}W}$	β_L $EQ_{H\ N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
2 Casks	2,647	768	909	9,873	13,103	19.0	16.2	9.82	**8.93**	3.73	4.95	22.1	22.5	**5.32**	**1.8**
4 Casks	4,633	1,265	1,378	13,807	27,290	16.6	15.3	9.77	**8.88**	2.98	5.89	24.0	36.2	**5.32**	**1.8**
8 Casks	8,755	2,247	2,311	30,818	34,320	14.8	14.4	8.80	**8.00**	3.52	3.92	23.0	56.2	**6.79**	**1.3**

c =	**2,200**	Total stress cohesion (psf)	F_V = Vertical load (Static + EQ_V)
ϕ =	**0.0**	Total stress friction angle (deg)	EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E\text{-}W}$ or $EQ_{H\ N\text{-}S}$
B =	**30**	Footing width (ft)	$\beta_B = \tan^{-1}[(EQ_{H\ E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).
L =	**Varies**	Footing length (ft)	$\beta_L = \tan^{-1}[(EQ_{H\ N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).
D_f =	**2.7**	Depth of footing (ft)	$\Sigma M_{@N\text{-}S} = e_B \times F_V$ $\Sigma M_{@E\text{-}W} = e_L \times F_V$
γ =	**80**	Unit weight of soil (pcf)	$B' = B - 2e_B$ $L' = L - 2e_L$
γ_{surch} =	**100**	Unit weight of surcharge (pcf)	$q_{actual} = F_V / (B' \times L')$
FS =	**1.1**	Factor of safety for dynamic loads.	

[gcot]\05996\calc\brng_cap\Pad\cu_phi.xls Table 2.6-8

FIGURE 1

FOUNDATION PLAN & PROFILE



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

FIGURE 2

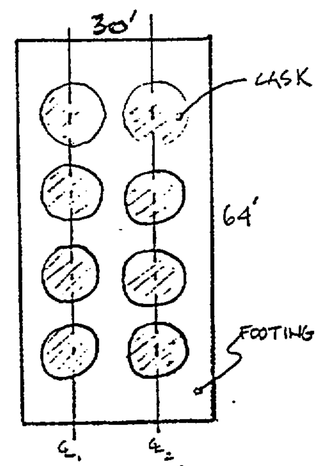
STATIC FOUNDATION LOAD / PRESSURE

TOTAL LOAD:

8 CASKS @ 356.5K = 2852 K

30' × 64' × 3' × 0.15 K/ft³ = 864 K

∴ TOTAL LOAD = 3716 K



PLAN VIEW
OF FOOTING

BEARING PRESSURE:

P_{actual} = LOAD/AREA

P_{actual} = 3716 K / 30' × 64'

P_{actual} = 1.94 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.

CALCULATION IDENTIFICATION NUMBER				PAGE 84
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

FIGURE 3

DETAIL OF SOIL CEMENT UNDER & ADJACENT TO CASK STORAGE PADS

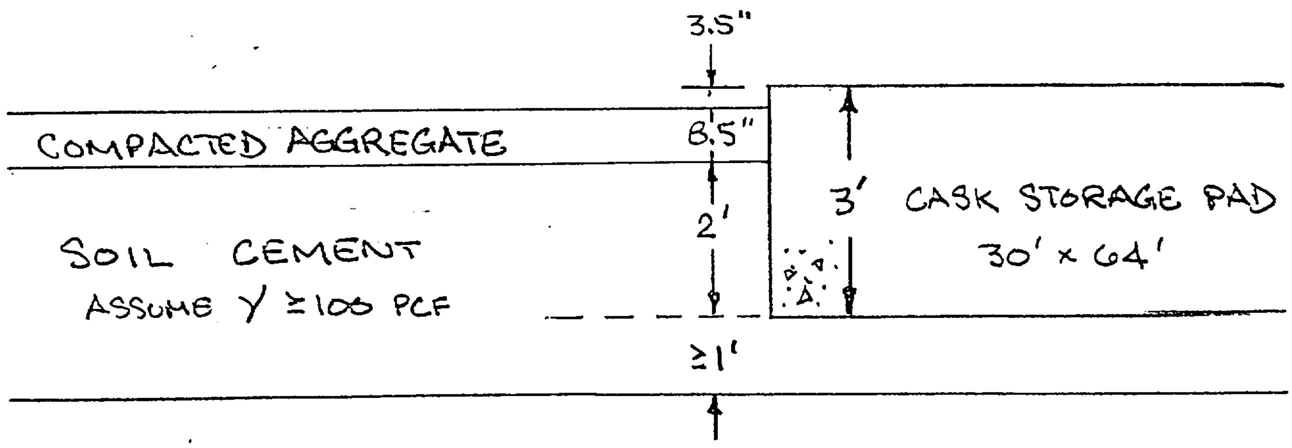
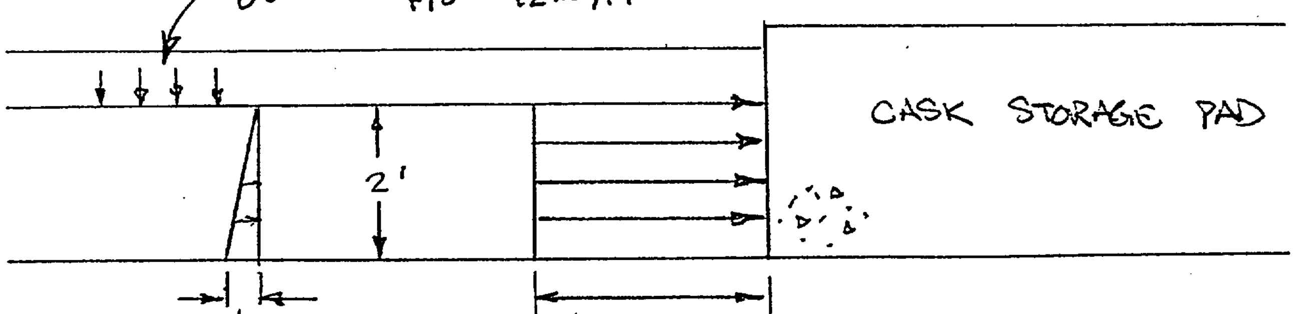


FIGURE 4

PASSIVE PRESSURE ACTING ON CASK STORAGE PADS

$$q_s = 0.125 \frac{K}{FT^3} \times \frac{8.5\ IN.}{12\ IN./FT} = 0.09\ KSF$$



$$p_p = \gamma H K_p$$

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

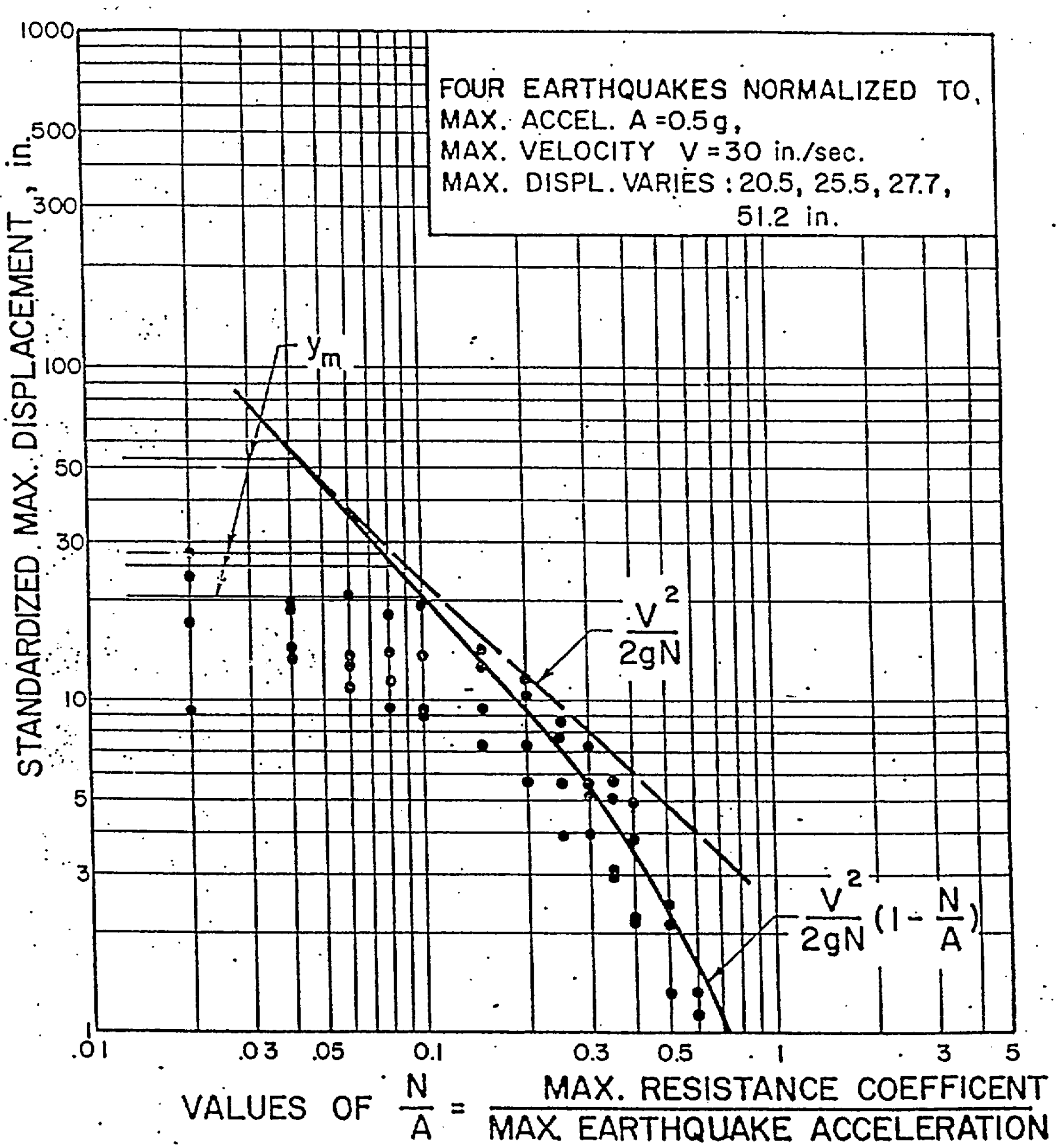
$$= (2\bar{c} + 0.09)\ KSF \times 2\ FT \quad \text{FOR } \phi = 0$$

$$K_p = 1.0$$

CALCULATION IDENTIFICATION NUMBER				PAGE 85
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

FIGURE 5

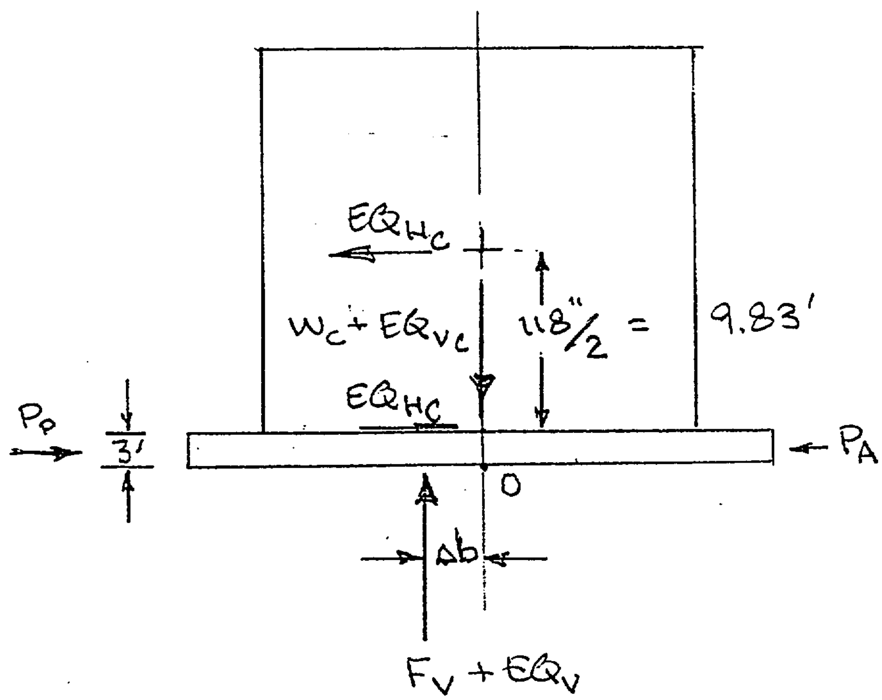
STANDARDIZED DISPLACEMENT FOR NORMALIZED EARTHQUAKES (SYMMETRICAL RESISTANCE)



From Newmark (1965)

FIGURE 6

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



$P_A << P_P$; therefore, It's conservative to Ignore both in ΣM.

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

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

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

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

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

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

IED JUN 1 2 2000 P. J. Trudeau

STONE & WEBSTER ENGINEERING CORPORATION

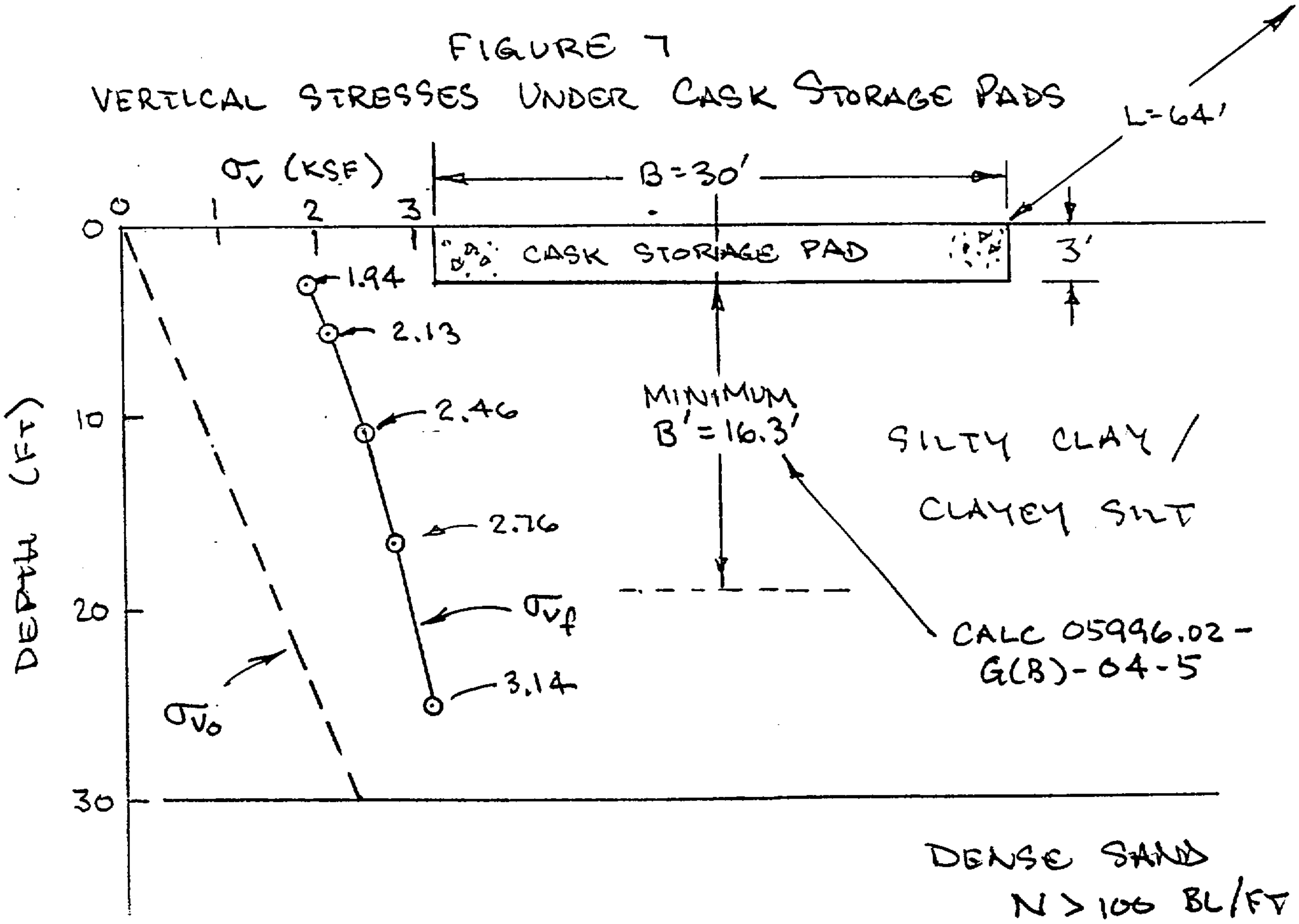
CALCULATION SHEET

▲5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 87
05996.02	G(B)	04-6		

FIGURE 7

VERTICAL STRESSES UNDER CASK STORAGE PADS



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

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

- TABLE 3 => σ_{vf} AS $f(z)$

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

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

ATTACHMENT A TO CALC 05996.01-G(B)-04-5 p A1/1

NOTES OF TELEPHONE CONVERSATION

JO No. 05996.01

PRIVATE FUEL STORAGE, LLC
PRIVATE FUEL STORAGE FACILITY

Date: 06-19-97
Time: 2:45 PM EDT

FROM:	Stan M. Macie	SWEC-Denver 1E		Tie Line 321-7305
	Wen Tseng	(ICEC)	Voice	(510) 841-7328
			(FAX)	(510) 841-7438
TO:	Paul J. Trudeau	SWEC-Boston 245/03		(617) 589-8473

SUBJECT: DYNAMIC BEARING CAPACITY OF PAD

DISCUSSION:

WTseng reported that his pad design analyses are being prepared for three loading cases: 2 casks, 4 casks, and 8 casks. The dynamic loads that he is using are based on the forcing time histories he received from Holtec. These forcing time histories were developed using a coefficient of friction between the cask and the pad of 0.2 and 0.8, where 0.2 provides the lower bound and 0.8 provides the upper bound loads from the cask to the pad.

He indicated that the bearing pressures at the base of the pad are greatest for the 2-cask dynamic loading case for $\mu = 0.8$ between the cask and the pad, because of eccentricity of the loading. For this case, the vertical pressures at the 30' wide loaded end of the pad are 5.77 ksf at one corner and 3.87 ksf at the other. He reported that it is reasonable to assume this pressure decreases linearly to 0 at a distance of ~32 ft; i.e., approximately half of the pad is loaded in this case. He also indicated that the horizontal pressure at the base of the pad is 1.04 ksf at the 30' wide end of the pad that is loaded by the 2 casks, and that this pressure decreases linearly over a distance of ~40' from the loaded end. He noted that the vertical pressures include the loadings (DL + dynamic loadings) of the casks and the pad, but the horizontal pressures apply only to the casks. Therefore, the inertia force of the whole pad must be added to the horizontal loads calculated based on the horizontal pressure distribution described above.

Since the table of allowable bearing pressures as a function of coefficient of friction between the cask and the pad that is in the design criteria does not include a value for $\mu = 0.8$, WTseng asked PJTrudeau to provide the allowable bearing pressure for this case.

SUPERSEDED BY ATT B

ACTION ITEMS:

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

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

~~00901~~

CEC

CALCULATION SHEET

ORIGINATOR		DATE 9/20/99	CALC. NO. G(PO17)-2	REV. NO.	0
			CHECKED	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	234

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.

Post-it® Fax Note 7671	Date 10-28-99 # of pages ▶ 11
To Paul Trudeau	From J.L. Cooper
Co./Dept.	Co.
Phone #	Phone #
Fax # 619-589-2959	Fax #

NOTE: CALC 05996.02-G(PO17)-2 REV 1
DATED 12-6-99 PER CALC INDEX

NOTED OCT 28 1999 P. J. Trudeau

CEC

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SUBJECT	Storage Pad Analysis and Design			SHEET	235

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

	k_s = 2.75 kcf	k_s = 26.2 kcf
Z_w(ft) =	0.164	0.0172
q_{zw}(ksf) =	0.45	0.45

Notes: 1. Z_w = maximum vertical displacement due to dead load (wt. of the pad only).

2. q_{zw} = vertical soil bearing pressure = $k_s \times Z_w$, where k_s = subgrade moduli = 2.75 and 26.2 kcf for lower-bound and upper-bound soils, respectively, and Z_w are obtained from CECSAP analysis results (Att. A).

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR	W	DATE 9/20/99	CHECKED	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	236

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	13.54	11.2	-53.28	-60.55	0.7244	1.22	-4.959	-5.451
7	13.5	11.19	-53.27	-44	0.7026	1.206	-4.966	-4.481
13	13.54	11.2	-53.28	-27.42	0.7244	1.22	-4.959	-3.479
144	-12.65	-27.63	-55.27	-81.67	-0.8428	-3.061	-6.121	-8.451
150	-12.74	-27.62	-55.24	-63.97	-0.8975	-3.061	-6.119	-6.723
156	-12.65	-27.63	-55.27	-46.31	-0.8428	-3.061	-6.121	-5.01
287	-43.58	-64.48	-53.28	-103	-5.152	-6.179	-4.959	-11.85
293	-43.63	-64.46	-53.27	-83.3	-5.178	-6.172	-4.966	-8.549
299	-43.58	-64.48	-53.28	-63.94	-5.152	-6.179	-4.959	-5.58
	Maximum Soil Bearing Pressure q_{zi}[1] (ksf)							
1	0	0	-1.465	-1.665	0	0	-1.299	-1.428
7	0	0	-1.465	-1.210	0	0	-1.301	-1.174
13	0	0	-1.465	-0.754	0	0	-1.299	-0.911
144	0.348	-0.760	-1.520	-2.246	0.221	0.802	-1.604	-2.214
150	0.350	-0.760	-1.519	-1.759	0.235	-0.802	-1.603	-1.761
156	0.348	-0.760	-1.520	-1.274	0.221	-0.802	-1.604	-1.313
287	-1.198	-1.773	-1.465	-2.833	-1.350	-1.619	-1.299	-3.105
293	-1.200	-1.773	-1.465	-2.291	-1.357	-1.617	-1.301	-2.240
299	-1.198	-1.773	-1.465	-1.758	-1.350	-1.619	-1.299	-1.462

Note:

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 displacement values listed are taken from the selected 9 nodes. They are Node 1, 7, 13, 144, 150, 156, 287, 293, and 299. The locations of these nodes are shown in Figure 1. Their maximum displacement values may not be the local maxima. By close examination, it is determined that the nine values taken for each loading case have encompassed the maximum value for that case.
4. For snow load, the soil bearing pressure is .45 ksf (Ref. 5).

45 psf = 0.045 KSF PER REV 3 OF DESIGN CRITERIA

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR		DATE 9/20/99	CHECKED	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	237

5.3.2 Dynamic Horizontal and Vertical Soil Pressures

Calculations of horizontal and vertical soil pressures due to dynamic cask driving forces resulting from earthquake motions are given in the following tables:

Table D-1(a) shows calculation of total maximum horizontal dynamic soil reactions in the X-direction (short direction of pad).

Table D-1(b) shows calculation of total maximum horizontal dynamic soil reactions in the Y-direction (long direction of pad).

Table D-1(c) shows a summary of total maximum horizontal dynamic soil reactions.

Table D-1(d) shows calculation of maximum vertical dynamic soil bearing pressures.

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
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PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	238

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

Node Number	Maximum Displacement Xd (x10^{-3} ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	6.106	3.738	33.63	3.256	1.974	17.72	1.673	1.380	10.29
7	6.110	3.738	33.68	3.256	1.975	17.73	1.674	1.379	10.31
13	6.106	3.739	33.64	3.256	1.972	17.73	1.673	1.377	10.30
144	8.131	15.69	16.72	4.406	8.923	17.88	2.335	5.129	10.75
150	8.130	15.69	16.72	4.408	8.928	17.89	2.333	5.097	10.76
156	8.137	15.69	16.70	4.409	8.933	17.89	2.338	5.061	10.75
287	22.76	34.77	34.90	12.26	19.48	18.14	6.776	10.68	10.89
293	22.76	34.78	34.92	12.27	19.48	18.16	6.777	10.70	10.90
299	22.76	34.76	34.91	12.27	19.46	18.16	6.776	10.68	10.89
Average	12.333	18.068	28.424	6.643	10.125	17.922	3.595	5.720	10.65
Kxd (kips/ft)	55188	55188	55188	102288	102288	102288	174240	174240	174240
Qxd (kips)	681	997	1569	680	1036	1833	626	997	1855

Notes:

1. Average = {sum(Xd)$_i$}/N; Xd=max. x-displ.; i=nodes 1,7,13,144,150,156,287,293,299; and N=9.

2. Qxd = Kxd x Average = total maximum horizontal-x soil reaction in Kips due to dynamic loading.

3. Kxd for LB, BE, and UB soils are dynamic horizontal-x soil spring stiffnesses given below:

(Kxd)LB =	4.60E+06 lb/in	Kxd)BE =	8.52E+06 lb/in	Kxd)UB =	1.45E+07 lb/in
	5.52E+04 Kips/ft		1.02E+05 Kips/ft		1.74E+05 Kips/ft

4. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.

5. Xd are obtained from CECSAP analysis results given in Att. A.

6. The maximum nodal displacements listed may not be concurrent. However, they are assumed to be concurrent for conservatism.

7. Node numbers are shown in Figure 1.

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR	KW	DATE 9/20/99	CHECKED	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	239

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

Node Number	Max. Displacement Yd ($x10^{-3}$ ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	9.382	17.42	29.04	5.446	10.100	17.04	3.550	5.444	10.67
7	7.698	14.54	17.42	4.581	8.865	17.23	2.829	5.085	10.80
13	9.788	14.65	20.90	5.119	9.150	17.41	3.116	5.711	10.92
144	9.472	17.51	29.08	5.563	10.240	17.07	3.588	5.602	10.71
150	7.746	14.66	17.40	4.660	8.984	17.24	2.889	5.226	10.83
156	9.856	14.76	20.72	5.225	9.310	17.42	3.245	5.874	10.95
287	9.570	17.54	29.13	5.671	10.380	17.06	3.767	5.734	10.71
293	7.833	14.72	17.39	4.803	9.120	17.23	3.001	5.348	10.81
299	10.000	14.89	20.54	5.346	9.366	17.41	3.370	5.890	10.93
Average	9.036	15.632	22.402	5.157	9.502	17.234	3.262	5.546	10.814
Kyd (kips/ft)	52428	52428	52428	97176	97176	97176	165600	165600	165600
Qyd (kips)	474	820	1175	501	923	1675	540	918	1791

Notes:

1. Average=(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 Average = total 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 for entire pad given below:

(Kyd)LB =	4.37E+06 lb/in	(Kyd)BE	8.10E+06 lb/in	(Kyd)UB	1.38E+07 lb/in
	5.24.E+04 Kips/ft		9.72.E+04 Kips/ft		1.66.E+05 Kips/ft

4. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.
5. Yd are obtained from CECSAP analysis results given in Att. A.
6. The maximum nodal displacement listed may not be concurrent. However, they are assumed to be concurrent for conservatism.
7. Node numbers are shown in Figure 1.

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR		DATE 9/20/99	CHECKED	DATE	9-23-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	240

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 =	681	997	1569	680	1036	1833	626	997	1855	E-W
Qyd =	474	820	1175	501	923	1675	540	918	1791	N-S

Notes:

1. Qxd and Qyd in Kips are calculated in Tables D-1(a) and (b), respectively.

2. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.

CALCULATION SHEET

			CALC. NO.	G(PO17)-2	REV. NO. 0
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PROJECT	Private Fuel Storage Facility				JOB NO. 1101-000
SUBJECT	Storage Pad Analysis and Design				SHEET 241

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

Node Number	Maximum Displacement Zd ($\times 10^{-3}$ ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	6.046	13.58	-30.77	4.002	7.599	-50.25	1.945	5.852	-33.37
7	6.421	9.074	-29.91	3.341	5.761	-24.61	1.955	3.728	-20.64
13	9.799	14.73	-47.10	4.855	10.53	-27.68	2.379	6.073	-21.03
144	-12.78	-24.37	-30.63	-9.079	-22.41	-29.56	-5.715	-15.900	-23.99
150	-6.301	-12.57	-16.70	-5.213	-12.41	-15.66	-4.055	-10.450	-12.29
156	-10.13	-25.14	-21.34	-5.896	-13.95	-29.82	-3.801	-11.180	-19.07
287	-26.50	-35.51	-69.21	-23.57	-27.08	-25.68	-18.900	-16.760	-14.97
293	-21.77	-32.04	-61.38	-17.39	-22.58	-21.37	-14.010	-14.500	-15.10
299	-26.01	-37.77	-54.79	-29.69	-22.41	-26.55	-15.430	-16.340	-16.84
	Maximum Soil Bearing Pressure q_{zd} (Kips/ft^2)								
1	0.00	0.00	-1.20	0.00	0.00	-3.53	0.00	0.00	-3.92
7	0.00	0.00	-1.16	0.00	0.00	-1.73	0.00	0.00	-2.42
13	0.00	0.00	-1.83	0.00	0.00	-1.94	0.00	0.00	-2.47
144	-0.50	-0.95	-1.19	-0.64	-1.57	-2.08	-0.67	1.87	-2.82
150	-0.25	-0.49	-0.65	-0.37	-0.87	-1.11	-0.48	-1.23	-1.44
156	-0.39	-0.98	-0.83	-0.41	-0.98	-2.09	-0.45	-1.31	-2.24
287	-1.03	-1.38	-2.70	-1.66	-1.90	-1.80	-2.22	-1.97	-1.76
293	-0.85	-1.25	-2.39	-1.22	-1.59	-1.50	-1.64	-1.70	-1.77
299	-1.01	-1.47	-2.13	-2.09	-1.57	-1.87	-1.81	-1.92	-1.98

Notes:

1. q_{zd} = maximum soil bearing pressure = $(Kzd \times Z_d)/A$, where $A = 64' \times 30' = 1920$ ft^2.
2. Kzd for LB, BE, and UB soils are vertical-z dynamic soil spring stiffnesses given below:

(Kzd)LB =	(Kzd)BE =	(Kzd)UB =
6.23E+06 lb/in	1.12E+07 lb/in	1.88E+07 lb/in
7.48.E+04 Kips/ft	1.35.E+05 Kips/ft	2.25.E+05 Kips/ft

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

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR		DATE 9/20/99	CHECKED	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	311

6.2 Vertical Soil Bearing Pressures and Horizontal Soil Shear Stresses

Vertical soil bearing pressures for individual loadings and combined loadings are summarized as shown in Table 5.

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

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR		DATE 9/20/99	CHECKED	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	312

Table 5

Summary of Vertical Soil Bearing Pressures (ksf)

Node Number		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	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	*
	Cask LL	1.35	1.36	1.36	0.35	0.35	0.35	0	0	0	
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	
	Cask EQ	2.22	1.64	1.81	0.67	0.48	0.45	0	0	0	
	100% Ve	4.71	4.14	4.31	2.16	1.97	1.94	1.14	1.14	1.14	
4-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	
	Snow LL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	*
	Cask LL	1.77	1.77	1.77	0.80	0.80	0.80	0	0	0	
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	
	Cask EQ	1.97	1.70	1.92	1.87	1.23	1.31	0	0	0	
	100% Ve	4.88	4.61	4.83	3.81	3.17	3.25	1.14	1.14	1.14	
8-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	
	Snow LL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	*
	Cask LL	1.47	1.47	1.47	1.60	1.60	1.60	1.47	1.47	1.47	
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	
	Cask EQ	2.70	2.39	2.13	2.82	1.44	2.24	3.92	2.42	2.47	
	100% Ve	5.31	5.00	4.74	5.56	4.18	4.98	6.53	5.03	5.08	

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.) x a_v, where pad wt. = 864 kips, and a_v = 0.533g.
(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 1.

* SNOW LOAD SHOULD BE 0.045 KSF (i.e., 45 psf); ∴ ADJUST "100% Ve" LINES ACCORDINGLY.

CALCULATION SHEET

International Civil Engineering Consultants, Inc.

ORIGINATOR	[signature]	DATE	9-23-99	CALC. NO. G(PO17)-2	REV. NO. 0
				CHECKED DH	DATE 9/23/99
PROJECT	Private Fuel Storage Facility				JOB NO 1101-000
SUBJECT	Storage Pad Analysis and Design				SHEET NO. 23

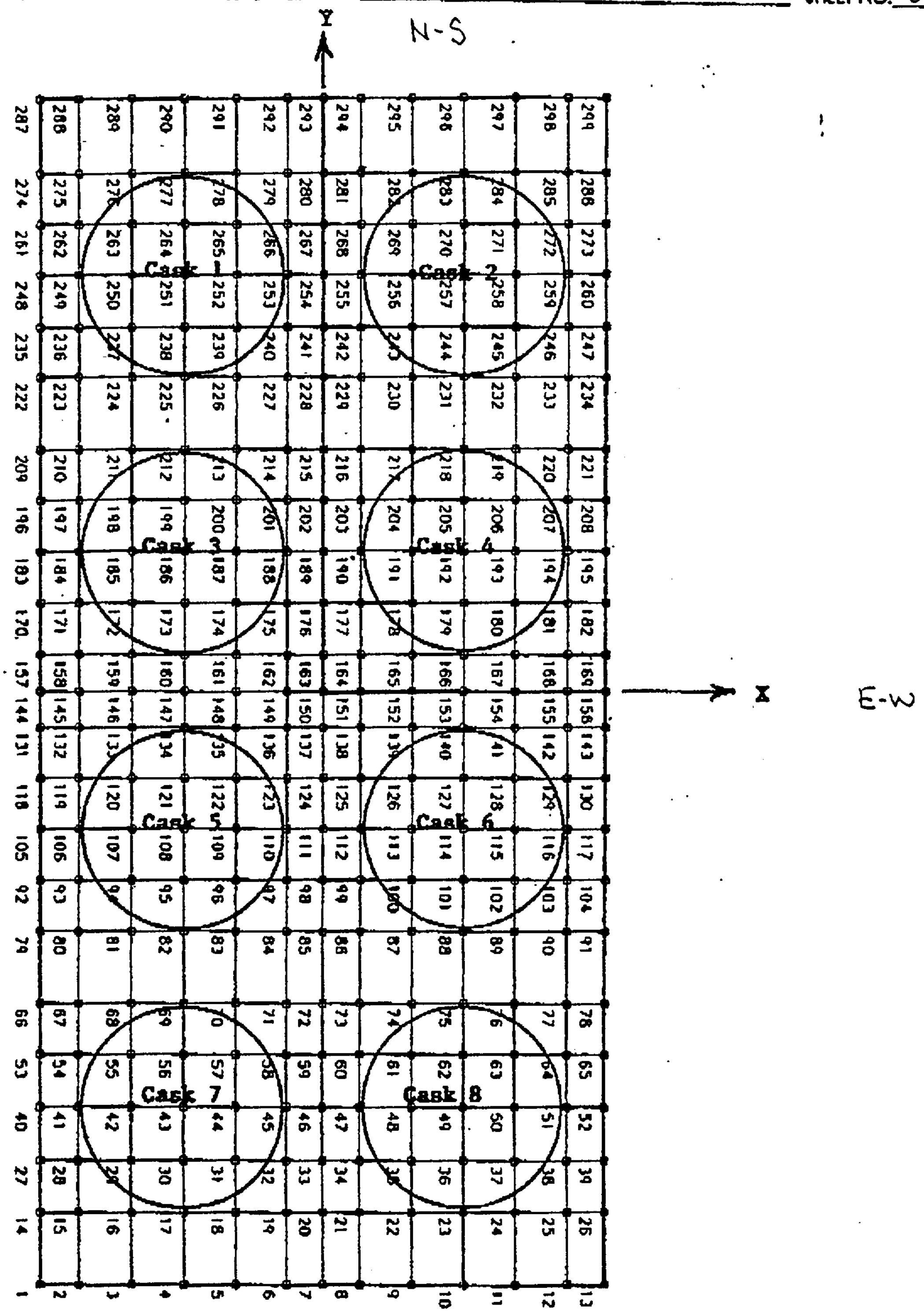

Figure 1 CECSAP Finite-Element Model with Node Numbers

TABLE 6

SUMMARY OF TRIAXIAL TEST RESULTS FOR SOILS WITHIN ~10 FT OF GROUND SURFACE AT THE SITE

Boring	Sample	Depth ft	Elev ft	w %	ATTERBERG LIMITS LL	PL	PI	USC Code	γ_m pcf	γ_d pcf	e_o	σ_c ksf	s_u ksf	ε_a %	Type	Date
B-1	**U-2C**	5.9	4453.9	47.1	66.1	33.4	32.7	MH	79.3	53.9	2.15	0.0	2.03	1.7	CU	Nov '99
B-1	**U-2B**	5.3	4454.5	52.9	80.6	40.9	39.7	MH	70.8	46.3	2.67	1.0	2.21	6.0	CU	Nov '99
B-4	**U-3D**	10.4	4462.1	27.4	42.5	24.7	17.8	CL	85.5	67.1	1.53	1.3	2.18	4.0	UU	Jan '97
C-2	**U-2D**	11.1	4453.4	35.6	See U-2C & E[1]			CL	78.5	57.9	1.93	1.3	2.39	11.0	UU	Jan '97
CTB-1	**U-3D**	8.7	4463.7	47.9	See U-3C[2]			CH	91.9	62.1	1.73	1.7	2.84	5.0	CU	June '99
CTB-4	**U-2D**	9.5	4465.5	45.2	See U-2E[2]			CH	87.7	60.4	1.81	1.7	3.11	6.0	CU	June '99
CTB-6	**U-3D**	8.3	4467.9	52.7				CH	85.7	56.2	2.02	1.7	2.70	7.0	CU	June '99
CTB-N	**U-1B**	5.7	4468.4	30.1	41.3	22.5	18.8	CL	100.6	77.3	1.20	1.7	3.00	8.0	CU	Nov '98
CTB-N	**U-2B**	7.7	4466.4	65.4	See U-2A[2]			MH	74.6	45.1	2.76	1.7	2.41	13.0	CU	June '99
CTB-N	**U-3D**	10.5	4463.6	52.2	61.1	30.8	30.3	CH	86.3	56.7	1.98	1.7	2.73	7.0	CU	June '99
CTB-S	**U-1B**	5.8	4468.7	73.6	66.2	40.9	25.3	MH	78.0	44.9	2.78	1.7	2.05	12.0	CU	Nov '98
CTB-S	**U-2D**	8.4	4466.1	54.6	57.9	28.9	29.0	CH	90.0	58.2	1.92	1.7	2.40	5.0	CU	June '99
B-1	**U-2D**	6.5	4453.3	45.2	59.8	34.7	25.1	MH	76.7	52.8	2.22	2.1	3.26	15.0	CU	Mar '99
B-3	**U-1B**	5.2	4463.0	33.5	52.4	25.2	27.2	MH	90.6	67.9	1.50	2.1	3.55	8.0	CU	Mar '99
C-2	**U-1D**	6.3	4458.2	50.5	70.3	41.3	29.0	MH	74.5	49.5	2.43	2.1	3.03	12.0	CU	Mar '99

NOTES 1 Attachment 2 of SAR Appendix 2A.
2 Attachment 6 of SAR Appendix 2A.

ATTACHMENT C p C1/3
CALC 05996.02-G(B)-04-6

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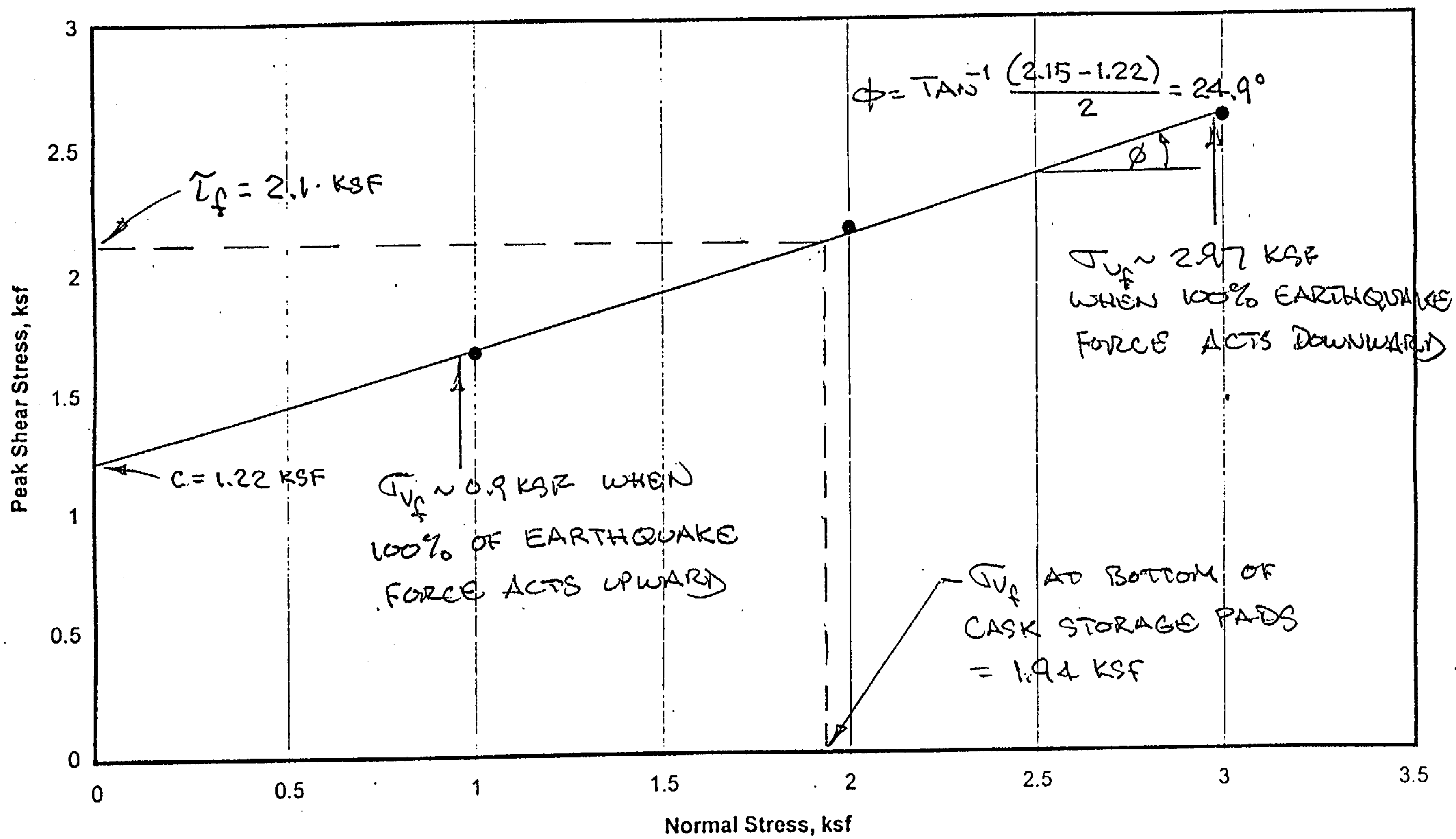
FIGURE 7

DIRECT SHEAR TEST

Boring C-2, Sample U-1C

PAD EMPLACEMENT AREA



CALC 05996.02-G(B)-05-2 p 32

ATTACHMENT C p C2
CALC 05996.02-G(B)-04-6

REF SAR APP 2A ATT 7

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

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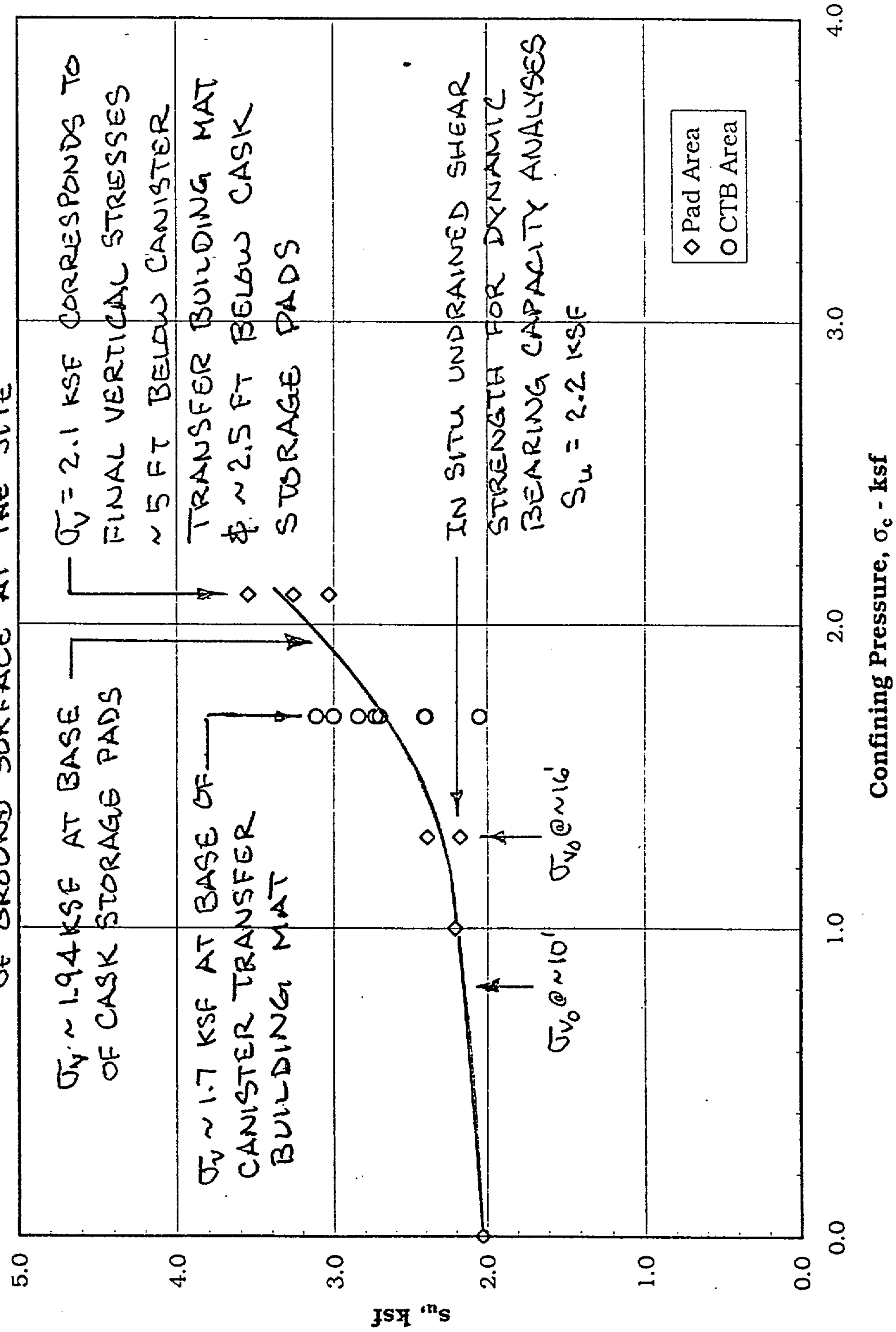
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 36
05996.02	G(B)	05-2		

Figure 11

Summary of Triaxial Test Results for Soils Within Depth of ~10 ft
OF GROUND SURFACE AT THE SITE



ATTACHMENT C P C3/3
CALC 05996.02-G(B)-04-6

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Private Fuel Storage Facility

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-04
Revision No. 6

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
Method			
Identify the method used to verify the "Method" of the calculation			
• By design review	✓		
• Compare the Method with another calculation			✓
• Alternate calculation			✓
If the compare method was used, is the statement identifying the other calculation identified in this calculation?			✓
If an alternate calculation was used for a QA Category I calculation, is it included with the calculation?			✓
Is the calculation method acceptable?	✓		
Assumptions			
Affirmative answers to the following questions are required:			
• Are all assumptions uniquely identified as assumptions and adequately described?	✓		
• Are all assumptions reasonable?	✓		
• Are all assumptions that require confirmation at a later date specifically identified as assumptions that must be confirmed?	✓		
For Revisions to the Calculation			
• Are changes clearly identified?	✓		
• For QA Category I calculations, is a reason for the revision given?	✓		
• Does the calculation identify the calculation, including revision, when applicable, which is superseded?	✓		

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-04
Revision No. 6

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
• Are affected pages identified with the new calculation number or revision number?	✓		
• When applicable, is an alternate calculation included as part of the calculation?			✓
• When applicable, is a statement identifying the calculation to which the method was compared included as part of the revision?			✓

Thomas Y. Chang	Thomas Y. Chang	6-16-2000
Printed Name	Signature	Date

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

ID'D ON TOC

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CLIENT & PROJECT	PAGE 1 OF
PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY	36 + 23 INSERTS + 57 pp OF ATT'S

CALCULATION TITLE (Indicative of the Objective):	QA CATEGORY (✓)
DOCUMENT BASES FOR GEOTECHNICAL PARAMETERS PROVIDED IN GEOTECHNICAL DESIGN CRITERIA	☑ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	G(B)	05		119

PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION *REQUIRED (✓) YES	NO
PAUL J. TRUDEAU 5/7/97 Paul J Trudeau TOM SPONSELLER 5/7/97 Tom Sponseller	ALAN F. BROWN 5/8/97 Alan F Brown	ALAN F. BROWN 5/8/97 Alan F Brown	0			✓
Alan C. Smith Alan C Smith PAUL J. TRUDEAU ~~12-16-99~~ Paul J Trudeau 1-21-2000	Thomas Y. Chang ~~12-20-99~~ TYC Thomas Y Chang 1-21-2000	Thomas Y. Chang ~~12-20-99~~ TYC Thomas Y Chang 1-21-2000	1	0 & 05996.01-G(B)-01, REV 3		✓
PAUL J. TRUDEAU 6-15-00 Paul J Trudeau	TY CHANG 6-15-00 Thomas Y Chang	TY CHANG 6-15-00 Thomas Y Chang	2	1 & 05996.01-G(B)-01, REV 3		✓

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CALCULATION IDENTIFICATION NUMBER				PAGE 2
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	05-2		

TABLE OF CONTENTS

RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1 – Description of / Reasons for Changes

p 1: Changed J. O. Number to 05996.02 from 05996.01 and updated number of pages.

p 2: Updated Table of Contents.

p 3: Added Record of Revisions.

pp 4, 4A-C, & 5: Changed soil properties to incorporate laboratory test results included in Attachments 3 to 7 of SAR Appendix 2A (added in SAR Amendment 6) and in Attachment 8 of SAR Appendix 2A (added in SAR Amendment 8)

p 5: Revised moist unit weights per laboratory test results presented in Tables 2 & 3 and revised earthquake coefficients to 2,000-yr return period design basis ground motion.

p 7: Added "/Compacted Aggregate" to title "Crushed Stone" and changed "structural fill" to "crushed stone" at bottom of page.

p 11: Changed Canister Transfer Building foundation from spread and strip footings to a mat.

p 14: Updated drawing numbers to current issue and revised differential settlement criteria for the Security & Health Physics Building to reflect the change in type of construction from one-story pre-engineered metal building to one-story reinforced-concrete masonry (SWEC, 1998).

p 16, 16A, 16B, & 17: Incorporated coefficients of subgrade reaction, which were originally in Calc 05996.01-G(B)-1, Rev 3, so that Calc G(B)-01, Rev 3 could be marked superseded by Calc 05996.02-G(PO18)-2, Rev 0 and this calc.

pp 22, 22A-22F: Incorporated low-strain moduli section, which was originally in Calc 05996.01-G(B)-1, Rev 3, so that Calc G(B)-01, Rev 3 could be marked superseded by Calc 05996.02-G(PO18)-2, Rev 0 and this calc.

pp 23 & 24: Added references to Reference section.

pp 25, 25A-25J: Added Tables 2 to 5.

pp 32-35: Added Figures 7 to 10.

p A1: Revised K_{AE}.

p C3: Revised cask weights.

p C5: Replaced "DRAFT" copy of Holtec drawing showing dimensions of casks with references to data available in SAR.

p D1: Added explanation for removal of "PRELIMINARY" drawings and reference to latest issue of applicable drawings.

REVISION 2 – Description of / Reasons for Changes

pp 4A - 4C: Revised discussion of results of direct shear tests and triaxial tests and added section titled "Undrained Shear Strength for Dynamic Bearing Capacity Analyses" to identify basis for undrained shear strength used in bearing capacity analyses for cask storage pads in Calc 05996.02-G(B)-04-6 and for the Canister Transfer Building in Calc 05996.02-G(B)-13-3.

pp 25A: Identified "UU" & "CU" tests in Triaxial Test heading in Table 2.

p 25K: Added Table 6 - "Summary of Triaxial Test Results for Soils Within ~10 Ft of Ground Surface at the Site".

p 32: Added annotations to Figure 7 to be consistent with annotations added to Figures 9 & 10.

p 34: Added annotations to Figure 9 to identify basis for shear strength used to resist sliding in Calc 05996.02-G(B)-13-3.

p 35: Added annotations to Figure 10 to identify basis for shear strength used to resist sliding in Calc 05996.02-G(B)-13-3.

p 36: Added Figure 11 - "Summary of Triaxial Test Results for Soils Within ~10 Ft of Ground Surface at the Site" to identify basis for undrained shear strength used in bearing capacity analyses for cask storage pads in Calc 05996.02-G(B)-04-6 and for the Canister Transfer Building in Calc 05996.02-G(B)-13-3.

OBJECTIVE

Document the bases for the recommended values of soil properties and geotechnical engineering parameters presented in the Geotechnical Design Criteria for the proposed Private Fuel Storage Facility (PFSF) at the Skull Valley, UT site.

CALCULATION METHOD/ASSUMPTIONS

As discussed below. No assumptions that require confirmation.

SOURCES OF DATA/EQUATIONS

As discussed below.

DISCUSSION

SOIL PROPERTIES

Geotechnical laboratory tests were performed on samples obtained from the boring programs. The results of these tests are summarized below.

Pad Emplacement Area

For the soils in the pad emplacement area, consisting of silt, clayey silt and silty clay, within the upper 25 to 30 ft of the profile, the soil properties, based on the test results shown in Table 2, are as follows:

Index Property:	**Minimum**	**Maximum**	**Average**
Water Content, %	8	58	32
Liquid Limit	25	77	44
Plastic Limit	20	46	30
Plasticity Index	0.5	38	14
Moist Unit Weight, pcf	64	91	78
Dry Unit Weight, pcf	40	71	56
Void Ratio	1.4	3.2	2.1
Saturation, %	28	64	53
Specific Gravity: 2.72			
Consolidation parameters:	**Low**	**High**	**Average**
Maximum past pressure, ksf:	5.6	7.2	6.2
Virgin compression ratio, CR:	0.25	0.34	0.29
Recompression ratio, RR:	0.008	0.017	0.012

Pad Emplacement Area (cont'd)

Direct shear tests were performed on Sample U-1C of Boring C-2, obtained from a depth of ~5.8 ft in the pad emplacement area. The results of these tests are presented in Table 5 and plotted in Figure 7 (from Attachment 7 of Appendix 2A of the SAR). Total-stress strength parameters based on these direct shear tests are c = 1.22 ksf and ϕ = 24.9°.

Unconsolidated-undrained and consolidated-undrained triaxial tests were performed on several samples obtained of the soils within the depth range of ~5 to ~10 ft in the pad emplacement area. The results of these tests are presented in Table 2 and plotted in Figure 8 (from Attachment 8 of Appendix 2A of the SAR). Total-stress strength parameters based on these triaxial tests are c = 1.4 ksf and ϕ = 21.3°.

The dotted line shown in this figure is tangent to the Mohr's circle for Sample U-2B of Boring B-1, and it indicates that the cohesion of this specimen is slightly less than that of the other specimens tested. This strength was lower because its natural water content (w_n) was higher than that of the other specimens. As indicated by the plots of water content vs depth presented in SAR Figure 2.6-20, most of the in situ soils in the upper ~25-ft layer at the site have w_n < 50%, which is more like Samples U-2C and U-2D; hence the recommendation that c = 1.4 ksf for these soils.

Canister Transfer Building Area

For the silt, clayey silt and silty clay soils in the Canister Transfer Building area, above the sand layer located at approximately 30 ft depth. (See Table 3)

Index Property:	**Minimum**	**Maximum**	**Average**
Water Content, %	7	86	40
Liquid Limit	28	83	51
Plastic Limit	18	48	30
Plasticity Index	4	38	20
Moist Unit Weight, pcf	73	118	92
Dry Unit Weight, pcf	40	98	65
Void Ratio	0.7	3.3	1.8
Saturation, %	40	88	71
Specific Gravity	2.71	2.73	2.72
Consolidation parameters:	**Low**	**High**	**Average**
Maximum past pressure, ksf:	6	26	13
Virgin compression ratio, CR:	0.13	0.37	0.31
Recompression ratio, RR:	0.014	0.020	0.018

Canister Transfer Building Area (cont'd)

Direct shear tests were performed on Sample U-3 of Boring CTB-6 and Sample U-1 of Boring CTB-S, obtained in the Canister Transfer Building area at depths corresponding approximately with the proposed depth of the foundation. The results of these tests are presented in Table 5 and plotted in Figures 9 and 10 (from Attachments 7 and 8 of Appendix 2A of the SAR). Total-stress strength parameters based on the average values from these direct shear tests are c = 1.13 ksf and ϕ = 21.1°.

The results of performing consolidated-undrained triaxial tests on samples obtained from beneath the Canister Transfer Building are presented in Table 3. These CU tests were performed at confining stresses of 1.7 ksf, which is approximately equal to the vertical stresses expected at the base of the Canister Transfer Building mat after completion of construction. As indicated at the bottom of the last page of Table 3, the undrained shear strengths (s_u) measured in the tests of samples obtained from beneath the Canister Transfer Building ranged from 1.66 to 3.15 ksf, with an average value of 2.64 ksf and a mean value of 2.73 ksf. These average and mean values are nearly equal to the results of averaging the s_u values measured at confining stresses of 1.3 ksf and 2.1 ksf on samples obtained in the pad emplacement area (on last page of Table 2). In addition, comparison of the index properties of samples obtained from both of these areas, presented in the tables above, indicate that these soils are similar, although those in the Canister Transfer Building area have slightly higher water contents, liquid limits, plasticity indices, and unit weights. Because the water contents of the clayey soils obtained from beneath the Canister Transfer building are slightly higher (average w_n = 40% vs 32% in the pad emplacement area), it is reasonable to expect the strength of these soils to be slightly lower than those in the pad emplacement area. Total-stress strength parameters applicable for the Canister Transfer Building area based on these triaxial tests are assumed to be the same as those described above based on the direct shear tests, namely c = 1.13 ksf and ϕ = 21.1°.

For the sand or sandy soils layer in the Canister Transfer Building area found in some of the borings located at a depth of 8 to 20 ft. (See Table 4)

Index Property:	**Minimum**	**Maximum**	**Average**
Water Content, %	3	15	6
Moist Unit Weight, pcf	85	105	98
Dry Unit Weight, pcf	77	102	93
Void Ratio	0.64	1.2	0.83
Saturation, %	11	32	19
% Fines	9	38	23
Specific Gravity: 2.69			

Undrained Shear Strength for Dynamic Bearing Capacity Analyses

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.

Table 6 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. 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, 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 unconsolidated-undrained (UU) triaxial tests that were performed at confining stresses of 1.3 ksf (SAR Appendix 2A, Attachment 2). 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 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.

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CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 05-2	OPTIONAL TASK CODE	PAGE 5

LATERAL EARTH PRESSURES

STATIC & DYNAMIC LATERAL EARTH PRESSURES SHOULD BE DETERMINED BASED ON THE DISTRIBUTIONS OF PRESSURES SHOWN IN GTG 6.15-1, FIGURE 1 *. (SWEC, 1982)

WHERE: z_w = DEPTH TO GROUNDWATER TABLE. GROUNDWATER IS > 100' DEEP; so z_w WILL GENERALLY BE > H_1 & H_2 WILL = 0.

K_o = AT-REST EARTH PRESSURE (SEE NEXT PAGE FOR K_a & K_p)

BASED ON EFFECTIVE STRESS STRENGTH PARAMETERS FOR UPPER LAYER OF SILT, SILTY CLAY, & CLAYEY SILT, $\phi \simeq 30° \Rightarrow K_o = 0.5 = \frac{1-\sin\phi}{1+\sin\phi}$

FOR γ_t, USE γ_m = 80 PCF FOR PAD EMPLACEMENT AREA - PER TABLE 2

FOR CTB AREA, PER TABLE 3, USE γ_m = 80 PCF FOR $0 < z < 5'$ & FOR $z > 5'$, USE γ_m = 90 PCF, MOIST UNIT WEIGHT OF IN SITU SILT, SILTY CLAY, & CLAYEY SILT, IF WALL IS POURED AGAINST IN SITU MATERIAL. $\boxed{1}$

γ_{SAT} = 105 PCF IF WALL IS BACKFILLED WITH THIS MATERIAL (COMPACTED)

γ = 125 PCF IF WALL IS BACKFILLED WITH COMPACTED GRANULAR BACKFILL.

$$\left.\begin{array}{l}\alpha_h = 0.528 = \text{HORIZ SEISMIC COEFF} \\ \alpha_v = 0.533 = \text{VERT} \quad " \quad "\end{array}\right\} \text{FROM GEOMATRIX (1999)}$$ $\boxed{1}$

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* COPY INCLUDED IN ATTACHMENT A

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 6
05996.02	G(B)	05 - 2		

ACTIVE EARTH PRESSURES (EFFECTIVE STRESS PARAMETERS)
LONG TERM

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$ EQ 13.1 LAMBE & WHITMAN (1969)

$\phi = 30°$ LONG TERM BASED ON PI = 23% ($5 \le PI \le 23$) & FIG 18.1 OF TERZAGHI & PECK (1967)

$$\therefore K_a = \frac{1 - \sin 30°}{1 + \sin 30°} = 0.33$$

$K_o = 1 - \sin\phi = 1 - \sin 30° = 0.50$ EQ 10.1 LAMBE & WHITMAN (1969)

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi}$$ EQ 13.2 LAMBE & WHITMAN (1969)

$$\therefore K_p = \frac{1 + \sin 30°}{1 - \sin 30°} = 3.0$$

$$\frac{K_p}{K_o} = \frac{3.0}{0.33} = 9.0$$

NOTE: AT-REST EARTH PRESSURES CAN BE REDUCED TO ACTIVE EARTH PRESSURES IF THE YIELD RATIO EXCEEDS 0.1%, WHERE YIELD RATIO = S/H, & S & H ARE DEFINED:

WALL ROTATION WALL TRANSLATION

BASED ON "MEDIUM DENSE SAND" CURVE IN FIG 6b* OF GTG 6.15-1
IN DETERMINING PASSIVE PRESSURES, ASSUME K INCREASES LINEARLY AS A FUNCTION OF YIELD RATIO & IT EQUALS K_o AT S/H = 0% TO A MAXIMUM OF ~~K_p AT~~ S/H = 2%.

* COPY INCLUDED IN ATTACHMENT A

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STRUCTURAL FILL

The in situ materials generally are not adequate for use as structural backfill; therefore, it is expected that structural fill materials will be obtained from an offsite source. Structural fill material should be granular material consisting of well graded sand and gravel, containing no more than 10% of material passing the #200 sieve and a maximum particle size not greater than 6 inches.

The following are recommended values for structural backfill:

Total unit weight = 125 pcf.

Friction angle = 35 degrees

Cohesion = 0.

Poisson's ratio = 0.33

California Bearing Ratio (CBR) = 40.

Coefficients of earth pressure for structural backfill are as follows:

At-rest,	K_o, is 0.43	1-sin 35°
Active,	K_a, is 0.27	(1-sin 35°)/(1+sin 35°)
Passive,	K_p, is 3.7.	(1+sin 35°)/(1-sin 35°)

Coefficient of friction for concrete placed on structural backfill is 0.70 (=tan 35°).

CRUSHED STONE/COMPACTED AGGREGATE

The following are recommended values for crushed stone:

Total unit weight = 125 to 140 pcf.

Friction angle = 40 degrees

Cohesion = 0.

Poisson's ratio = 0.33

California Bearing Ratio (CBR) = 80.

Coefficient of friction for concrete placed on crushed stone is 0.8 (=tan 40°).

STONE & WEBSTER ENGINEERING CORPORATION

NOTED FEB 21 1997 P. J. Trudeau

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 8
05996.02	G(B)	05 - 2		

GRADATIONS (MAY BE APPLICABLE FOR SPECIFYING FILL MATERIALS)

REVIEW 1992 STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION, UT DOT, SALT LAKE CITY, UT
TO DETERMINE IF THERE IS PERTINENT INFORMATION REGARDING GEOTECHNICAL CRITERIA OR PARAMETERS.

FINDINGS:

P.	TABLE	DESCRIPTION
140	220-1	GRADATION REQ'TS: GRANULAR BACKFILL BORROW
162	301-1	" " : AGGREGATE JOB-MIX GRADATION (BASE COURSE FOR ROADS)
173	304-1	" " AGGREGATE JOB-MIX GRADATION (LEAN CONCRETE BASE COURSE)
301	505-1	" " COARSE AGGREGATE FOR PORTLAND CEMENT CONC.
302	505-3	" " FINE " " " "
524	—	" " UNDERDRAIN GRANULAR BACKFILL
190	402-1	" " AGGREGATE FOR DENSE-GRADED ASPHALT CONC

p 161 BASE COURSE OF GRAVEL, CRUSHED ROCK, OR SLAG — DRY-RODDED UNIT WEIGHT ≥ 75 PCF.

p 163 OPTIMUM MOISTURE CONTENT ± 2% REQUIRED WHEN COMPACTING BASE COURSE
BASE COURSE COMPACTED TO 97% OF MAX LAB DENSITY, AASHTO T-180 METHOD D

p. 169 HYDRATED LIME TREATED ROADBED COMPACT TO 92% MAX LAB DENSITY, AASHTO T-99 METHOD D, WITHIN ±2% OPTIMUM

COPIES OF GRADATIONS INCLUDED IN ATT B.

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BEARING CAPACITY CRITERIA

The minimum factor of safety against a bearing capacity failure due to static loads is 3.0, based on typical geotechnical engineering practice (p 271, Peck, Hanson and Thornburn (1974).

The minimum factor of safety against a bearing capacity failure due to static loads + dynamic loads from the design earthquake is 1.1. This is consistent with the acceptance criteria specified NUREG-75/087, Section 3.8.5, "Foundations," II.5, "Structural Acceptance Criteria" for the factor of safety against overturning. It is also consistent with the with AASHTO, Standard Specifications for Highway Bridges, Section 6.4.2(B), Interim 1995, which states:

> "Because of the dynamic cyclic nature of seismic loading, the ultimate capacity of the foundation supporting medium should be used in conjunction with these load combinations."

and, thus, only requires a factor of safety of 1.0.

This recommendation is based on the fact that the accelerations from the design earthquake will equal the peak ground acceleration for only a very brief period of time for a limited number of cycles, and therefore, a low value of the factor of safety can be accepted.

> NOTE: See Calculation 05996.02-G(B)-04, Rev 6, for stability analyses of the storage pads and Calculation 05996.01-G(B)-07, Rev 0, for allowable bearing capacities of strip & square footings. Stability analyses of the Canister Transfer Building are performed in Calculation 05996.02-G(B)-13, Rev 3.

DEPTH OF FOOTINGS FOR PROTECTION AGAINST FROST

All exterior footings shall be founded at a depth of no less than 30 inches below finished grade to provide protection against frost, in accordance with local code requirements. Interior footings in heated areas may be founded at shallower depths, if desired.

Overturning, Sliding, and Flotation Criteria

The minimum factors of safety against these failures are based on acceptance criteria specified in NUREG-75/087, Section 3.8.5, "Foundations," Section II.5, "Structural Acceptance Criteria", which states:

"...the factors of safety against overturning, sliding, and floatation are acceptable if found in accordance with the following:

	Minimum Factors of Safety		
For Combination	Overturning	Sliding	Floatation
a. D+H+E	1.5	1.5	—
b. D+H+W	1.5	1.5	—
c. D+H+E'	1.1	1.1	—
d. D+H+W_t	1.1	1.1	—
e. D+F'	—	—	1.1

Where[1]: D = Dead load

H = Lateral earth pressure

E = Loads due to OBE[2]

E' = Loads due to SSE

W = Loads due to design wind

W_t = Loads due to tornado wind

F' = Bouyant force due to design basis flood."

Note 1: Based on Sect II.3 of SRP 3.8.4 & Section II.5 of 3.8.5.

Note 2: Based on ¶6.4.1 of SWEC(1997a), "Storage Facility Design Criteria", Rev 2,

"...the Operating Basis Earthquake (OBE) is not applicable for a PFSF."

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.

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STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 11
05996.02	G(B)	05-2		

SETTLEMENT CRITERIA

CANISTER TRANSFER BUILDING

BASED ON SWEC DRAWINGS 0599601-EA-8-D
EA-9-D $\boxed{1}$

DIMENSIONS OF THE CANISTER TRANSFER BUILDING ARE:
~165' WIDE* × ~260' LONG × ~90' HIGH. $\boxed{1}$
TOP OF CRANE RAIL ~60' ABOVE FLOOR LEVEL.
WALLS WILL BE REINFORCED CONCRETE, ~30" THICK.

TABLE 3 OF WAHLS (1981) INDICATES ALLOWABLE AVERAGE SETTLEMENT FOR "BUILDING WITH BRICK WALLS, REINFORCED WITH REINFORCED CONCRETE OR REINFORCED BRICK" IS 6 IN. AND FOR "SOLID REINFORCED CONCRETE FOUNDATIONS OF SMOKESTACKS, SILOS, TOWERS, ETC." IS 12 IN.

∴ ALLOWABLE SETTLEMENT OF THE EXTERIOR WALLS OF THE CANISTER TRANSFER BUILDING SHOULD BE AT LEAST AS GREAT AS THAT RECOMMENDED FOR BUILDING WITH BRICK WALLS (= 6"), SINCE REINFORCED CONCRETE WALLS ARE LESS SUSCEPTIBLE TO DAMAGE DUE TO DIFFERENTIAL SETTLEMENTS THAN ARE BRICK WALLS.

* NOTE: THE SHORTEST, CONTINUOUS WALLS ARE ~65' WIDE, ~~00005~~
SEE COL LINES 8, 10, & 11.

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STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 12
05996.02	G(B)	05-2		

SETTLEMENT CRITERIA -

DIFFERENTIAL SETTLEMENTS ARE GENERALLY ~ 3/4 OF MAXIMUM SETTLEMENT (SECT 3-6, TENG, 1962 & SECT 2-21, BOWLES, 1968); ∴, ASSUMING ALLOWABLE AVERAGE SETTLEMENT = 6", THE ALLOWABLE DIFFERENTIAL SETTLEMENT WOULD BE 0.75×6" = 4.5".

FOR THE SHORTEST, CONTINUOUS WALLS OF THE CANISTER TRANSFER BUILDING, THE MAXIMUM DIFFERENTIAL SETTLEMENT IS EXPECTED TO OCCUR AT THE CENTER OF THE STRIP FOOTING; ∴ $L = 65'/2 = 32.5'$. THIS RESULTS IN

$$\frac{\delta}{L} = \frac{4.5" \times \frac{1\,FT}{12"}}{32.5\,FT} = 0.0115 \quad OR \quad \frac{\delta}{L} = \frac{1}{87}$$

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STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 13
05996.02	G(B)	05-2		

SETTLEMENT CRITERIA

TABLE 14.1, "ALLOWABLE SETTLEMENT", IN LAMBE & WHITMAN (1969) INDICATES PROBABILITY OF NONUNIFORM SETTLEMENT FOR FRAMED STRUCTURES IS LIKELY IF THE MAXIMUM SETTLEMENT EXCEEDS 2" TO 4". IT ALSO INDICATES THAT THE MAXIMUM TILTING OF CRANE RAILS AND THE DIFFERENTIAL SETTLEMENT OF REINFORCED-CONCRETE BUILDING CURTAIN WALLS SHOULD BE LIMITED TO 0.003ℓ ($\delta/\ell \simeq 1/300$). FOR THE CONTINUOUS FOOTINGS SUPPORTING THE SHORTEST, CONTINUOUS WALLS, ~65' WIDE, OF THE CANISTER TRANSFER BUILDING (SEE DRAWING 0599601-EM-1), THIS RESULTS IN A δ = 1.3 IN. WHERE

$$\delta \leq 0.0033\ell = 0.0033 \times \frac{65'}{2} \times 12"/' = 1.3 \text{ IN.}$$

DIFFERENTIAL SETTLEMENTS ARE GENERALLY ~3/4 OF MAXIMUM SETTLEMENT (SECT 3-6 OF TENG, 1962 AND SECT 2-21, BOWLES, 1968); $\therefore$ TO LIMIT DIFFERENTIAL SETTLEMENT BETWEEN CENTER OF THE WEST & EAST WALLS TO 1.3", THE MAXIMUM ALLOWABLE SETTLEMENT IS $\sim \frac{1.3"}{0.75} = 1.73$ IN.

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SETTLEMENT CRITERIA

Other Buildings (QA Cat III): Administration, Operations & Maintenance, and Security & Health Physics Buildings

Based on ¶6.4.1 and 2 of SWEC(1998), "Balance of Facility Design Criteria", Rev 3: the Administration Building and the Operations and Maintenance Building will be one-story pre-engineered metal buildings. See SWEC Drawings 0599601-EA-1-C and EA-3-C for plan and elevation views of the Administration Building, and Drawings EA-4-C and EA-5-C for the Operations and Maintenance Building.

It is reasonable to characterize these as simple steel frame structures. Because of the inherent flexibility of steel structures, these structures are expected to be less susceptible to damage due to differential settlements than the Canister Transfer Building. Table 14.1 of Lambe & Whitman (1969) indicates that the differential settlement of "simple steel frame" structures should be limited to 0.005ℓ.

Based on ¶6.4.3 of SWEC (1998), the Security and Health Physics Building will be a one-story reinforced-concrete masonry structure. For increased conservatism and to limit the potential for wall cracking, assume this type of construction is similar to the "one-story brick mill building" for which Table 14.1 of Lambe & Whitman (1969) indicates that the differential settlement should be limited to 0.001ℓ to 0.002ℓ — use 0.0015ℓ.

Using one-half of the width of these buildings to determine maximum differential settlement, the allowable differential settlements are calculated as follows:

Building	Drawing 0599601	Width	Length	δ_{diff}	δ_{max}
Administration	EA-1-C	80	150	2.4	3.2
Op's & Maint'n	EA-4-C	80	200	2.4	3.2
For columns spaced at 20'		N/A	N/A	1.2	1.6
For columns spaced at 16'		N/A	N/A	1.0	1.3
Security & Health Physics	EA-6-D	76	120	0.7	0.9

where $\delta_{diff} = 0.005\ell = 0.005 \times ½ \text{ width} \times 12$ in./ft for the Administration and Op's & Maint'n Buildings.

$\delta_{diff} = 0.0015\ell = 0.0015 \times ½ \text{ width} \times 12$ in./ft for the Security & Health Physics Building.

$\delta_{max} = \delta_{diff}/¾$, since differential settlement is normally taken as ~3/4 of maximum settlement.

Conclusions Regarding Settlement Criteria

To limit the expected differential settlements to tolerable values, wall footings of the Administration Building and the Operations & Maintenance Building should be designed such that the maximum estimated settlement at the center of the minimum width of the buildings is ≤2* inches and spread footings supporting column loads spaced ~16 to 24 ft should be designed such that the maximum estimated settlement at the center of the footings is ≤1.5 inches. Because the type of construction used for the Security & Health Physics Building (one-story reinforced–concrete masonry) is more susceptible to cracking due to differential settlements, wall footings of that building should be designed such that the maximum estimated settlement at the center of the minimum width of the building is ≤1 inch.

* Note, the range of maximum settlement is 1.73" to 4.5" based on data presented on pp 11-14. Because of the consistent nature of the upper ~25 to 30 ft layer of silt, silty clay, and clayey silt, as evidenced by the N-values in Table 1, differential settlements are expected to be less of a problem than at most sites. Therefore, recommend using 2", which is slightly > than the minimum value of 1.73" calculated for the Canister transfer Building. Note also, the Canister transfer Building foundation has been changed to a mat foundation. Structures founded on mat foundations are more tolerant of differential settlements than are those constructed on spread footings. Limiting maximum settlements to 2" should for these structures should minimize settlement-related problems during the life of the facility.

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 16
05996.02	G(B)	05-2		

COEFFICIENT of VERTICAL SUBGRADE REACTION

for STORAGE PAD $\quad$ 30 ft x 64 ft x 3 ft thick
(per SWEC drawing 0599601-EY-2-H)

Terzaghi (1955) indicates $R \sim 7 \times$ slab thickness
(R = range of influence)

$\therefore R = 7 \times 3 = 21\text{ft}$ or $2R = 42\text{ ft}$

For clays, FIG 4c OF Terzaghi (1955) indicates k_s should BE BASED ON THE DISTANCE BETWEEN CONCENTRATED LOADS OR 2R, WHICHEVER IS SMALLER.

However, for the case of a single cask on the pad, k_s should be calculated based ON THE WIDTH OF THE PAD OR 2R, WHICHEVER IS SMALLER.

$\therefore B = 30'$ SINCE $2R \sim 42'$.

FOR clays, $k_s = k_{s1}\frac{1}{B}$ $\quad$ Eq. 7 Terzaghi (1955)

From Table 2 of Terzaghi (1955)

$\bar{k}_{s_1} = 50-100\ \text{T/ft}^3$ for stiff clay, $q_u = 1-2$ Tsf

UU tests on upper layer soils, $q_u = 2.2 - 2.4$ ksf or $1.1 - 1.2$ Tsf

$\therefore$ use $\bar{k}_{s_1} = 50\ \text{T/ft}^3 = \underline{\underline{100\ \text{K/ft}^3}}$

$k_{s_{(30\times00)}} = 50\ \text{T/ft}^3 \times \frac{1\text{ ft}}{30\text{ ft}} = 1.67\ \text{T/ft}^3 = 3.33\ \text{K/ft}^3$

or $1.93\ \text{lb/in}^3$

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 16A
05996.02	G(B)	05-2		

from pg 265 DAS (1995)

$\bar{K}_{s_1} = 44 - 92 \text{ lb/in}^3$ for stiff clay

for $\bar{K}_{s_1} = 50 \text{ T/ft}^3 \times \left(\frac{1 \text{ ft}}{12 \text{ in}}\right)^3 \times \frac{2000 \text{ lb}}{1 \text{ T}} = 58 \text{ lb/in}^3$

this is in the range recommeded by DAS

for rectangular footing, Terzaghi (1955) recommends:

$k_{s1} = \bar{K}_{s_1} \frac{l + 0.5}{1.5 l}$ where $l = \frac{\text{length}}{\text{width}}$, $k_s = K_{s1} \frac{1}{B}$

$$K_{s_{30\times64}} = \bar{K}_{s1} \frac{\frac{L}{B} + 0.5}{1.5 \frac{L}{B}} \cdot \frac{1}{B} = 1.67 \text{ T/ft}^3 \frac{\left(\frac{64}{30} + 0.5\right)}{1.5\left(\frac{64}{30}\right)}$$

$$K_{s_{30\times64}} = 1.37 \text{ T/ft}^3 = \underline{\underline{2.75 \text{ K/ft}^3}} = 1.59 \text{ lb/in}^3$$

---

for footings on footings on cohesionless silts, assume Terzaghi (1955) recommendations for sands are applicable.

$k_{1\times1} = 6N, \text{ T/ft}^3$ Eq. 4.49 DAS (1995)

for upper Layer $8 < N < 20$ typically

for $N = 8$ $k_{1\times1} = 48 \text{ T/ft}^3$

$N = 20$ $k_{1\times1} = 160 \text{ T/ft}^3$

assume $N = 10$, $k_{1\times1} = 60 \text{ T/ft}^3 = \underline{\underline{120 \text{ K/ft}^3}}$

Terzaghi (1955) proposes using the following

$k_{s_{1\times1}} = 40\ T/ft^3$ for loose ⎫
$= 130\ T/ft^3$ for medium ⎭ dry to moist sand

Use $k_{s_{1\times1}} = 60\ T/ft^3$ as recommended by Terzaghi (1955) for loose to medium dense sand

$$k_{s_{B\times B}} = k_s = \bar{k}_{s_1}\left(\frac{B+1}{2B}\right)^2 \qquad \text{Eq. 8 Terzaghi (1955)}$$

$$k_{s_{30\times30}} = 60\ T/ft^3\left(\frac{30+1}{2\times30}\right)^2 = 16.0\ T/ft^3 = 32.0\ K/ft^3$$

$$= 18.5\ lb/in^3$$

No need for this correction, however this is conservative

$$k_{s_{30\times64}} = k_{30\times30}\frac{\left(\frac{L}{B}+0.5\right)}{1.5\frac{L}{B}} = 16.0\ T/ft^3\ \frac{\left(\frac{64}{30}+0.5\right)}{1.5\left(\frac{64}{30}\right)}$$

O.K. TLC

$$k_{s_{30\times64}} = 13.2\ T/ft^3 = \underline{\underline{26.3\ K/ft^3}}$$

SUMMARY OF RESULTS
COEFFICIENT OF VERTICAL SUBGRADE REACTION

SOIL TYPE	$k_{s_{1'\times1'}}$	$k_{s_{30'\times64'}}$
CLAY	100 K/FT³	~2.75 K/FT³
COHESIONLESS SILT	120 K/FT³	~26 K/FT³

COEFFICIENT OF HORIZONTAL SUBGRADE REACTION

Terzaghi (1955) indicates (p 317) that k_{h1} for piles embedded in clay can be assumed to be roughly identical with values of k_{s1} for beams resting on the horizontal surface of the same clay. Therefore, $k_{h1} \sim k_{s1} = 50$ t/ft^3, for stiff clay, where $q_u \sim 1$ tsf.

The value for a pile of width B and L >> B is given by $k_h = k_{h1}/1.5B$. Therefore, for the clayey soils, $k_h \sim 100/1.5B$ k/ft^3, or 67/B k/ft^3.

For cohesionless soils, Terzaghi recommends (Table 3) that $n_h = 7$ t/ft^3 for dry or moist loose sands and 21 t/ft^3 for medium dense sands. To be conservative, for the cohesionless silts and sandy soils at the site, assume n_h is approximately equal to the average of these values, or ~15 t/ft^3, which = 30 k/ft^3.

Eq 19b indicates $k_h = p/y = n_h \times z/1.5B$ (assuming B>>L). Therefore, for the cohesionless soils, $k_h \sim 20z/B$ k/ft^3.

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 18
05996.02	G (B)	05-2		

CALIFORNIA BEARING RATIO (CBR) VALUES FOR SKULL VALLEY SITE

ESTIMATES OF THE CBR ARE GIVEN FOR THE FOLLOWING MATERIALS:

TYPE 1. EXISTING SITE MATERIAL (SILT, CLAYEY SILT, SILTY CLAY)

TYPE 2. BASE MATERIAL I (GRAVEL, - CLEAN)

TYPE 3. BASE MATERIAL II (CRUSHED STONE)

TYPE 4. SUB-BASE OTHER THAN SITE MATERIAL (i.e., SAND OR SILTY SAND)

SOURCES:

A. DM-7.2, DEPT. OF NAVY FACILITIES ENGINEERING COMMAND, 1986, TABLE 1, pg 39

B. YODER & WITCZAK (1975) Principles of Pavement Design, 2nd Ed, J.W WILEY & SONS, NY

DESIGN CBR VALUES

MATERIAL	CBR VALUE
TYPE 1	5-15
TYPE 2	60-80
TYPE 3	80
TYPE 4	20-40

THE RANGES GIVEN FOR CBR VALUES ALLOW FOR VARIABILITY IN MATERIAL COMPOSITION (I.E. TYPE 2 COULD BE ANYTHING FROM A SILTY GRAVEL TO A CLEAN, WELL-GRADED GRAVEL). ONCE THE MATERIAL COMPOSITIONS ARE KNOWN WITH SOME CERTAINTY, THE TABLES ON THE FOLLOWING 2 PAGES MAY BE USED TO NARROW DOWN THE CBR VALUES

~~01002~~

J.O. or W.O. # 05996.02

DIVISION & GROUP G(B)

DM-7.2, 1986

7.2-39

TABLE 1
Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression: At 1.4 tsf (20 psi)	Typical Value of Compression: At 3.6 tsf (50 psi)	Typical Strength Characteristics: Cohesion (as compacted) psf	Cohesion (saturated) psf	ϕ (Effective Stress Envelope Degrees)	Tan ϕ	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
				Percent of Original Height								
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	$5 \times >10^{-5}$	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	$2 \times >10^{-6}$	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	5 - 20	100 - 300
ML	Inorganic silts and clayey silts.	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay.	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	$5 \times >10^{-7}$	.....	
CL	Inorganic clays of low to medium plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	$5 \times >10^{-7}$	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50 - 150
OH	Organic clays and silty clays	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown.
(..) indicates insufficient data available for an estimate.

CALC No. 05-2

P 19

01003

J.O. or W.O. # 05996.02 | DIVISION & GROUP G(B) | CALC NO. 05-2 | P 20

TABLE 7.4. Characteristics Pertinent to Road and Airway Foundation*

Major Divisions (1)	(2)	Letter (3)	Name (4)	Value as Foundation When Not Subject to Frost Action (5)	Value as Base Directly under Wearing Surface (6)	Potential Frost Action (7)	Compressibility and Expansion (8)	Drainage Characteristics (9)	Compaction Equipment (10)	Unit Dry Weight (pcf) (11)	Field CBR (12)	Subgrade Modulus k (pci) (13)
Coarse-grained soils	Gravel and gravelly soils	GW	Gravel or sandy gravel, well graded	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	125–140	60–80	300 or more
		GP	Gravel or sandy gravel, poorly graded	Good to excellent	Poor to fair	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	120–130	35–60	300 or more
		GU	Gravel or sandy gravel, uniformly graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	115–125	25–50	300 or more
		GM	Silty gravel or silty sandy gravel	Good to excellent	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	130–145	40–80	300 or more
		GC	Clayey gravel or clayey sandy gravel	Good	Poor	Slight to medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120–140	20–40	200–300
	Sand and sandy soils	SW	Sand or gravelly sand, well graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	110–130	$\boxed{20–40}$	200–300
		SP	Sand or gravelly sand, poorly graded	Fair to good	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	105–120	15–25	200–300
		SU	Sand or gravelly sand, uniformly graded	Fair to good	Not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	100–115	10–20	200–300
		SM	Silty sand or silty gravelly sand	Good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	120–135	20–40	200–300
		SC	Clayey sand or clayey gravelly sand	Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105–130	10–20	200–300
Fine-grained soils	Low compressibility LL < 50	ML	Silts, sandy silts, gravelly silts, or diatomaceous soils	Fair to poor	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	100–125	5–15	100–200
		CL	Lean clays, sandy clays, or gravelly clays	Fair to poor	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired equipment, sheepsfoot roller	100–125	5–15	100–200
		OL	Organic silts or lean organic clays	Poor	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired equipment, sheepsfoot roller	90–105	4–8	100–200
	High compressibility LL > 50	MH	Micaceous clays or diatomaceous soils	Poor	Not suitable	Medium to very high	High	Fair to poor	Rubber-tired equipment, sheepsfoot roller	80–100	4–8	100–200
		CH	Fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	90–110	3–5	50–100
		OH	Fat organic clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	80–105	3–5	50–100
Peat and other fibrous organic soils		Pt	Peat, humus, and other	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical			

* From Corps of Engineers.

YODER & WITCZAK, 1975

~~01004~~

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 21
05996.02	G(B)	05 - 2		

CBR

ACCESS RD STA 0+00 @ ~ N 7,318,200
E 1,292,650
BASED ON SWEC DRWG 0599601-EY-6-A

BORING	Δ EAST FROM STA 0+00	STA
AR-1	8576.205	85+76.21
AR-2	6477.655	64+77.66
AR-3	4394.555	43+94.56
AR-4	2318.515	23+18.52
AR-5	339.765	3+39.77
STA	0.	

~ STA 4+20 USING DIST ALONG ℄, WHICH CURVES IN VICINITY OF AR-5

CONCLUSION: RE CBR ACCESS RD

NOTE: BORINGS IN THE VICINITY OF THE ACCESS RD INDICATE THAT THE EXISTING NEAR-SURFACE SOILS ARE SILT, SILTY CLAY, & CLAYEY SILT; ∴ LOW CBR VALUES ARE APPLICABLE FOR ACCESS ROAD CONSTRUCTED ON EXISTING SOILS.

Z FT	AR-1	AR-2	AR-3	AR-4	AR-5
0-1.5	CLAYEY SILT	SILT	SILT	SILTY CLAY	SILT
5-6.5	SAND	SAND	SILTY CLAY	CLAYEY SILT	SANDY SILT OVER SILTY GRAVEL
10-11.5	SAND	SAND	SILT	SANDY SILT	SILTY GRAVEL
15-16.5	SILT & CLAY	SANDY SILT	CLAYEY SILT	CLAYEY SILT	NR
20-21.5	SILTY CLAY	SAND	SILTY SAND	SANDY GRAVEL	
25-26.5	SILTY CLAY & SILT	SILT & SILTY CLAY	SILTY GRAVEL	NR	
30-31.5	SILT & CLAYEY SILT	SILTY CLAY	SILTY GRAVEL & SILTY CLAY		

~~01005~~

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 22
05996.02	G(B)	05-2		

Low-Strain Moduli

from Geomatrix calc 05996.02-G(P018)-2 Rev 0

Best Estimate Profile

	Depth to Layer Base (ft)	h (ft)	Vs (ft/sec)	Vp (ft/sec)	Computed Poisson's Ratio	Unit Weight (pcf)
Layer 1	10	10	540	1135	0.354	85
Layer 2	12	2	720	1250	0.252	92.5
Layer 3	25	13	865	1445	0.221	92.5
Layer 4	45	20	1015	1705	0.226	115
Layer 5	85	40	2000	3400	0.235	120
Layer 6	125	40	4511.155	7813.549	0.250	135
Layer 7	625	500	4511.155	7813.549	0.250	145
half space			6397.638	11154.856		170

Layer 1 (0-10 ft)

$$G = \rho (V_s)^2 = \frac{\left(\frac{85\ pcf}{1000\ lb/k}\right)}{32.2\ ft/sec^2} \times (540)^2 = \underline{\underline{769.8\ ksf}}$$

(say 770)

$E = 2(1+\mu)G$ from Eq. 12.4 Lambe & Whitman (1969)

$$E = 2(1+0.354)\ 769.8$$

$$E = \underline{\underline{2085\ ksf}}$$

Layer 2 (10-12 ft)

$$G = \frac{\frac{92.5}{1000}}{32.2} \times (720)^2 = \underline{\underline{1489\ ksf}}$$

(say 1490)

$$E = 2(1+0.252)\ 1489$$

$$E = \underline{\underline{3729}}\ ksf$$

(say 3730)

Estimate E

Layer 3 (12-25 ft) $G = \frac{92.5}{32.2} \times (865)^2 \frac{1}{1000} = \underline{\underline{2149}}$ Ksf (say 2150)

$E = 2(1+0.221)\,2149 = \underline{\underline{5249}}$ Ksf (say 5250)

Layer 4 (25-45 ft) $G = \frac{115}{32.2} \times (1015)^2 \frac{1}{1000} = \underline{\underline{3679}}$ Ksf (say 3680)

$E = 2(1+0.226)\,3679$

$E = \underline{\underline{9022}}$ Ksf (say 9020)

See next page for Layers 5→7 E calculat.

ESTIMATE LOW-STRAIN CONSTRAINED MODULI (Compression Modulus)

$$D = \frac{E(1-\mu)}{(1+\mu)(1-2\mu)}$$ Eq. 12.8 Lambe & Whitman (1969)

Layer 1 $\mu = 0.354$, $E = 2085$ KSF

$$D = \frac{2085(1-0.354)}{(1+0.354)(1-2\times0.354)} = \underline{\underline{3407}} \text{ Ksf}$$ (say 3410)

Layer 2 $\mu = 0.252$, $E = 3729$ Ksf

$$D = \frac{3729(1-0.252)}{(1+0.252)(1-2\times0.252)} = \underline{\underline{4492}} \text{ Ksf}$$ (say 4490)

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 22B
05996.02	G(B)	05-2		

Estimate, E

Layer 5 (45 - 85 ft depth)

$$G = \frac{120}{32.2}(2000)^2 \frac{1}{1000} = \underline{\underline{14,910}} \text{ Ksf}$$

$$E = 2(1+0.235)\,14,910 = \underline{\underline{36,830}} \text{ Ksf}$$

Layer 6 (85 - 125 ft depth)

$$G = \frac{135}{32.2}(4511)^2 \frac{1}{1000} = \underline{\underline{85,310}} \text{ Ksf}$$

$$E = 2(1+0.250)\,85310 = \underline{\underline{213,300 \text{ Ksf}}}$$

Layer 7 (125 - 625 ft. depth)

$$G = \frac{145}{32.2}(4511)^2 \frac{1}{1000} = \underline{\underline{91,634}} \text{ Ksf}$$

(say 91630)

$$E = 2(1+0.25)\,91634 = \underline{\underline{229,100}} \text{ Ksf}$$

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 22C
05996.02	G(B)	05-2		

Estimate D

Layer 3 $\mu = 0.221$, $E = 5249$ ksf

$$D = \frac{5249(1-0.221)}{(1+0.221)(1-2\times0.221)} = \underline{\underline{6002 \text{ Ksf}}}$$

(say 6000)

Layer 4 $\mu = 0.226$, $E = 9022$ Ksf

$$D = \frac{9022(1-0.226)}{(1+0.226)(1-2\times0.226)} = \underline{\underline{10394 \text{ ksf}}}$$

(say 10390)

see next page for Calc of D for layers 5→7

ESTIMATE LOW-STRAIN. BULK MODULI

$$B = \frac{\sigma_0}{\Delta V/V} = \frac{E}{3(1-2\mu)}$$

Eq 12.6 Lambe & Whitman (1969)

Layer 1

$$B = \frac{2085}{3(1-2\times0.354)} = \underline{\underline{2380 \text{ Ksf}}}$$

Layer 2

$$B = \frac{3729}{3(1-2\times0.252)} = \underline{\underline{2506 \text{ Ksf}}}$$

(say 2510)

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CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 22D
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	05-2		

Estimate, D

Layer 5

$$D = \frac{36830\,(1-0.235)}{(1+0.235)(1-2\times 0.235)} = \underline{\underline{43040\ \text{Ksf}}}$$

Layer 6

$$D = \frac{213300\,(1-0.25)}{(1+0.25)(1-2\times 0.25)} = \underline{\underline{256{,}000\ \text{Ksf}}}$$

Layer 7

$$D = \frac{229100\,(1-0.25)}{(1+0.25)(1-2\times 0.25)} = \underline{\underline{274{,}900\ \text{Ksf}}}$$

Estimate, B

Layer 3.

$$B = \frac{5249}{3(1-2\times 0.221)} = \underline{\underline{3136}} \text{ Ksf}$$

(say 3140)

Layer 4

$$B = \frac{9022}{3(1-2\times 0.226)} = \underline{\underline{5488}} \text{ Ksf}$$

(say 5490)

Layer 5

$$B = \frac{36830}{3(1-2\times 0.235)} = \underline{\underline{23,160}} \text{ Ksf}$$

Layer 6

$$B = \frac{213300}{3(1-2\times 0.25)} = \underline{\underline{142,200}} \text{ Ksf}$$

Layer 7

$$B = \frac{229,100}{3(1-2\times 0.25)} = \underline{\underline{152,700}} \text{ Ksf}$$

SUMMARY OF LOW-STRAIN MODULI

Layer	Shear Modulus (ksf)	Elastic Modulus (ksf)	Constrained Modulus (ksf)	Bulk Modulus (ksf)
[illegible]	770	2,085	3,410	2,386
2	1,490	3,730	4,490	2,510
3	2,150	5,250	6,000	3,140
4	3,680	9,020	10,390	5,490
5	14,910	36,830	43,040	23,160
6	85,310	213,300	256,000	142,200
7	91,630	229,100	274,900	152,700

CALCULATION IDENTIFICATION NUMBER				PAGE 23
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 05-2	OPTIONAL TASK CODE	

CONCLUSIONS

This calculation documents the bases for the recommended values of soil properties and geotechnical engineering parameters presented in the Geotechnical Design Criteria for the proposed Private Fuel Storage Facility (PFSF) at the Skull Valley, UT site.

REFERENCES

- AASHTO, "Standard Specifications for Highway Bridges," Section 6.4.2(B), Interim 1995, Washington, D. C.
- Bowles, J. E., Foundation Analysis and Design, McGraw-Hill, New York, NY, 1968.
- Das, B.M., Principles of Foundation Engineering, PWS Publishing Co., Boston, MA, 1995.
- ConeTec(1999) Cone penetration testing report, Private Fuel Storage Facility, prepared for Stone and Webster Engineering Corp., Denver, CO, 2 volumes.
- Geomatrix (1997A), PFSF Report 0559601-G(P05)-1 Rev 0, "Deterministic Earthquake Ground Motions Analysis, Private Fuel Storage Facility, Skull Valley, Utah," prepared by Geomatrix Consultants, Inc and William Lettis & Associates, San Francisco, CA, March 1997.
- Geomatrix (1997B), PFSF Calculation 05996.01-G(P05)-1, Rev 1, "Development of Soil and Foundation Parameters in Support of Dynamic Soil-structure Interaction Analyses," prepared by Geomatrix Consultants, Inc, San Francisco, CA, March 1997.
- Geomatrix (1999), PFSF Calculation 05996.02-G(P018)-2, Rev 1, "Soil and Foundation Parameters for Dynamic Soil-structure Interaction Analyses, 2,000-year Return Period Design Ground Motions," prepared by Geomatrix Consultants, Inc, San Francisco, CA, August 1999.
- Lambe, T. W., and Whitman, R. V., Soil Mechanics, John Wiley & Sons, New York, NY, 1969.
- NAVFAC (1986), DM 7.2, "Foundations and Earth Structures," Dept of the Navy, Naval Facilities Eng'g, Command, Alexandria, VA.
- Newmark, N.M., 1965. Fifth Rankine Lecture: "Effects of Earthquakes on Dams and Embankments". Geotechnique, Institution of Civil Engineers, London. pp. 139-60.
- NUREG-75/087, USNRC Standard Review Plan, Section 3.8.5, "Foundations," Washington, D. C., March 1979.
- Peck, R. W., Hanson, W. E., and Thornburn, T. H., Foundation Engineering, John Wiley & Sons, New York, NY, 1974.

- Richardt, F.E., Jr., Hall, J.R., Jr., and Woods, R.D., Vibrations of Soils and Foundations, Prentice-Hall, Englewood Cliffs, NJ, 1970.
- Seed, H. B., and Whitman, R. V., "Design of Earth Retaining Structures for Dynamic Loads," Proc ASCE, 1970 Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures," 1970, pp 103-147.
- SWEC (1982), Geotechnical Technical Guideline, "Determination of Lateral Pressures on Buried Structures in Granular Soils," Stone & Webster Engineering Corporation, Boston, MA, 1982.
- SWEC (1997), PFSF Report No. 05996.01-G(B)-2 Rev 0, "Geotechnical Data Report," Stone & Webster Engineering Corporation, Boston, MA, 1997.
- SWEC (1997a), "Storage Facility Design Criteria," Rev 2, Stone & Webster Engineering Corporation, Denver, CO.
- SWEC (1998), "Balance of Facility Design Criteria," Rev 3, Stone & Webster Engineering Corporation, Denver, CO.
- Teng, W. C., Foundation Design, Prentice-Hall, Englewood Cliffs, NJ, 1962.
- Terzaghi, K, "Evaluation of Coefficients of Subgrade Reaction," Geotechnique, Vol 5, 1955, pp 297-325.
- Terzaghi, K., and Peck, R. B., Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, NY, 1967
- UT DOT (1992), "1992 Standard Specifications for Road and Bridge Construction," UT DOT, Salt Lake City, UT.
- Wahls, H. E., "Tolerable Settlement of Buildings," Proceedings of ASCE, JGED, Vol 107, No. GT11, Nov 1981, pp 1489-1504.
- Yoder & Witczak, Principles of Pavement Design, Second Edition, John Wiley & Sons, New York, NY, 1975.

TABLE 1

SUMMARY OF BLOW COUNTS IN LAYER 1 IN STORAGE PAD AREA

ELEVATION		BORING							
TOP	BOTTOM	A-1	A-2	A-3	A-4	B-1	B-2	B-3	B-4
4475	4470				14				22
4470	4465			4	18			9	9
4465	4460		1	9	13		4	U	U
4460	4455	23	U	15	18	13	5	U	15
4455	4450	13	11	15	12	U	13	18	21
4450	4445	22	14	20	20	15	16	12	21
4445	4440	19	17	30	50	20	12	24	34
4440	4435	13	16	34		12	15	28	
4435	4430	36							

ELEVATION		BORING							
TOP	BOTTOM	C-1	C-2	C-3	C-4	D-1	D-2	D-3	D-4
4475	4470				15				8
4470	4465			11	7		6	6	4
4465	4460	3	18	6	11		6	14	24
4460	4455	8	U	8	14	40	11	11	22
4455	4450	U	U	10	15	12	15	9	9
4450	4445	16	13	9	20	14	18	11	16
4445	4440	8	11	22	21	13	17	39	
4440	4435		34			16			
4435	4430								

ELEVATION		N_{AVG}	N_{MEDIAN}
TOP	BOTTOM	BLOWS/FT	
4475	4470	15	15
4470	4465	8	7
4465	4460	10	9
4460	4455	16	14
4455	4450	13	13
4450	4445	16	16
4445	4440	22	20
4440	4435	21	16
4435	4430	36	36

FOR ENTIRE LAYER:

N_{AVG} = 15.7 BLOWS/FT

N_{MEDIAN} = 14.0 BLOWS/FT

U = UNDISTURBED SAMPLE

TABLE 2 - Sheet 1 of 4
Laboratory Test Results on Clays and Silts in the Pad Emplacement Area

												Consolidation Tests			Triaxial Tests					
															UU			CU		
Boring No.	Sample	Z_{avg}	Natural Water Content	Liquid Limit	Plastic Limit	Plastic Index	Liquidity Index	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Saturation	σ_{mpp} (ksf)	CR	RR	σ_c (ksf)	s_u (ksf)	ε_a (%)	σ_c (ksf)	s_u (ksf)	ε_a (%)
A-1	S 2	5.8	34.7	54.8	30.9	23.9	0.16													
A-1	S 3	10.8	19.8	28.8	25.8	3.0	-2.00													
A-1	S 4	15.8	22.3	30.2	27.6	2.6	-2.04													
A-1	S 5	20.8	55.4	58.6	43.0	15.6	0.79													
A-2	S 1	0.8	15.6	28.9	23.3	5.6	-1.38													
A-2	U 2B	5.6	40.1					85.9	61.3	1.70	0.64									
A-2	U 2C	6.2	52.8	70.2	42.9	27.3	0.36	70.7	46.2	2.58	0.56									
A-2	U 2D	6.7	48.8					80.4	54.1	2.06	0.64									
A-2	U 2E	7.0	45.4	61.8	36.7	25.1	0.35													
A-2	S 3	10.8	18.4	27.0	24.5	2.5	-2.44													
A-2	S 4	15.8	29.7	36.5	26.5	10.0	0.32													
A-2	S 5	20.8	28.2	38.0	26.8	11.2	0.13													
A-2	S 6	25.8	27.9	41.4	30.4	11.0	-0.23													
A-3	S 2	5.8	36.0	49.8	23.3	26.5	0.48													
A-3	S 3	10.8	43.3	60.1	35.1	25.0	0.33													
A-3	S 4	15.8	25.9	35.8	27.7	8.1	-0.22													
A-4	S 2	5.8	44.2	69.0	42.4	26.6	0.07													
A-4	S 3	10.8	10.8		Nonplastic															
A-4	S 4	15.8	19.3	29.9	22.4	7.5	-0.41													
A-4	S 5	20.8	37.8	56.5	41.6	14.9	-0.26													
A-4	S 6	25.8	15.2	29.1	19.8	9.3	-0.49													
B-1	U 2B	5.3	52.9	80.6	40.9	39.7	0.30	70.8	46.3	2.67	0.54							1	2.21	6.0
B-1	U 2C	5.9	47.1	66.1	33.4	32.7	0.42	79.3	53.9	2.15	0.60				0	2.03	1.7			
B-1	U 2D	6.5	45.2	59.8	34.7	25.1	0.42	76.7	52.8	2.22	0.55							2.1	3.26	15.0
B-1	S 3	10.8	23.0	39.4	29.0	10.4	-0.58													
B-1	S 4	15.8	23.0	35.2	25.9	9.3	-0.31													
B-1	S 5	20.8	45.9	50.3	35.8	14.5	0.70													
B-2	S 2	5.8	32.0	47.4	25.6	21.8	0.29													
B-2	U 1A	8.0	45.7																	
B-2	U 1F	10.0	45.1																	

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TABLE 2 - Sheet 2 of 4
Laboratory Test Results on Clays and Silts in the Pad Emplacement Area

Boring No.	Sample	z_{avg}	Natural Water Content	Liquid Limit	Plastic Limit	Plastic Index	Liquidity Index	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Saturation	Consolidation Tests σ_{mpp} (ksf)	Consolidation Tests CR	Consolidation Tests RR	Triaxial Tests UU σ_c (ksf)	Triaxial Tests UU s_u (ksf)	Triaxial Tests UU ε_a (%)	Triaxial Tests CU σ_c (ksf)	Triaxial Tests CU s_u (ksf)	Triaxial Tests CU ε_a (%)
B-2	S 3	10.8	18.9	29.8	25.8	4.0	-1.73													
B-2	S 4	15.8	12.6		Nonplastic															
B-2	S 5	20.8	43.9	55.1	46.2	8.9	-0.26													
B-2	S 6	25.8	20.1	31.8	20.0	11.8	0.01													
B-3	S 1	0.8	8.9	26.6	19.7	6.9	-1.57													
B-3	U 1B	5.5	33.5	52.4	25.2	27.2	0.31	90.7	67.9	1.50	0.61							2.1	3.55	8.0
B-3	U 1D	6.5	47.2																	
B-3	U 1E	6.7	45.7																	
B-3	U 1F	6.9	45.6																	
B-3	U 2D	10.5	15.2																	
B-3	U 2H	11.6	18.1																	
B-3	U 2J	12.0	22.2																	
B-3	S 3	20.8	44.6	54.3	41.6	12.7	0.24													
B-4	S 2	5.8	48.4	56.5	27.8	28.7	0.72													
B-4	U 3D	10.7	27.4	42.5	24.7	17.8	0.15	85.5	67.1	1.531	0.49				1.3	2.18	4.0			
B-4	U 3J	12.1	14.0																	
B-4	S 4	15.8	19.9	30.7	24.6	6.1	-0.77													
B-4	S 5	20.8	24.2	35.4	29.9	5.5	-1.04													
B-4	S 6	25.8	24.5	32.6	24.3	8.3	0.02													
C-1	S 2	5.8	53.0	67.4	39.3	28.1	0.49													
C-1	U 3B	10.9	30.3	33.0	28.1	4.9	0.45	84.3	64.7	1.63	0.51	7.2	0.252	0.011						
C-1	U 3C	11.1	38.9	47.8	34.6	13.2	0.33	77.5	55.8	2.04	0.52	5.6	0.310	0.008						
C-1	U 3D	11.5	46.7	61.1	44.1	17.0	0.15	75.8	51.7	2.29	0.56	6.0	0.339	0.017						
C-1	U 3E	11.7	43.2																	
C-1	U 3F	11.9	32.1																	
C-1	S 4	15.8	27.4	34.2	24.4	9.8	0.31													
C-1	S 5	20.8	42.7	49.7	38.7	11.0	0.36													
C-2	U 1A1	5.1	39.0																	
C-2	U 1A2	5.3	37.8																	
C-2	U 1C	6.0		76.9	39.1	37.8	-1.03													

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STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO. 05996.02

DIVISION & GROUP G(B)

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STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 25C
05996.02	G(B)	05-2		

5010.65

TABLE 2 - Sheet 3 of 4
Laboratory Test Results on Clays and Silts in the Pad Emplacement Area

												Consolidation Tests			Triaxial Tests					
															UU			CU		
Boring No.	Sample	z_{avg}	Natural Water Content	Liquid Limit	Plastic Limit	Plastic Index	Liquidity Index	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Saturation	σ_{mpp} (ksf)	CR	RR	σ_c (ksf)	s_u (ksf)	ε_a (%)	σ_c (ksf)	s_u (ksf)	ε_a (%)
C-2	U 1C1	5.8	55.7					69.4	44.5	2.81	0.54									
C-2	U 1C2	6.0	58.2					63.7	40.2	3.22	0.49									
C-2	U 1C3	6.1	52.7					75.1	49.2	2.45	0.59									
C-2	U 1D	6.5	50.5	70.3	41.3	29.0	0.32	74.5	49.5	2.43	0.57							2.1	3.03	12.0
C-2	U 1E	6.9	47.9																	
C-2	U 2B	10.8	14.3					81.6	71.4	1.378	0.28									
C-2	U 2C	11.0	27.6	34.6	26.9	7.7	0.09	82.8	64.9	1.62	0.46	6.0	0.273	0.010						
C-2	U 2D	11.4	35.6					78.5	57.9	1.933	0.50				1.3	2.39	11.0			
C-2	U 2E	11.8	39.7	41.2	28.5	12.7	0.88	80.3	57.5	1.95	0.55									
C-2	U 2F	12.0	34.1																	
C-2	S 2	15.8	30.3	40.0	24.4	15.6	0.38													
C-2	S 3	20.8	41.8	48.8	37.2	11.6	0.40													
C-3	S 2	5.8	26.8	43.1	22.4	20.7	0.21													
C-3	S 3	10.8	32.6	48.8	29.4	19.4	0.16													
C-3	S 4	15.8	27.9	32.9	23.1	9.8	0.49													
C-3	S 5	20.8	39.5	50.8	35.8	15.0	0.25													
C-3	S 6	25.8	18.1	26.2	19.5	6.7	-0.21													
C-4	S 2A	5.2	28.6	46.1	22.9	23.2	0.25													
C-4	S 2B	6.0	50.6	69.5	44.2	25.3	0.25													
C-4	S 3	10.8	18.2	26.5	26.0	0.5	-15.6													
C-4	S 4	15.8	26.5	36.6	26.9	9.7	-0.04													
C-4	S 5	20.8	40.7	52.5	41.5	11.0	-0.07													
C-4	S 6	25.8	18.7	29.2	20.1	9.1	-0.15													
D-1	S 2	5.8	36.3	54.6	29.4	25.2	0.27													
D-1	S 3	10.8	28.6	40.5	25.2	15.3	0.22													
D-1	S 4	15.8	32.2	47.3	33.1	14.2	-0.06													
D-1	S 5	20.8	20.7	30.0	19.5	10.5	0.11													
D-2	S 2	5.8	36.9	46.4	31.1	15.3	0.38													
D-2	S 3	10.8	34.2	54.0	28.6	25.4	0.22													
D-2	S 4	15.8	22.6	44.3	29.9	14.4	-0.51													
D-2	S 5	20.8	12.2	37.7	31.6	6.1	-3.18													

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TABLE 2 - Sheet 4 of 4
Laboratory Test Results on Clays and Silts in the Pad Emplacement Area

Boring No.	Sample	z_{avg}	Natural Water Content	Liquid Limit	Plastic Limit	Plastic Index	Liquidity Index	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Saturation	Consolidation Tests σ_{mpp} (ksf)	Consolidation Tests CR	Consolidation Tests RR	Triaxial Tests UU σ_c (ksf)	Triaxial Tests UU s_u (ksf)	Triaxial Tests UU ε_a (%)	Triaxial Tests CU σ_c (ksf)	Triaxial Tests CU s_u (ksf)	Triaxial Tests CU ε_a (%)
D-2	S 6	25.8	13.9	31.4	19.5	11.9	-0.47													
D-3	S 2	5.8	23.5	43.4	27.3	16.1	-0.24													
D-3	S 3	10.8	25.0		Nonplastic															
D-3	S 4	15.8	36.8	40.6	28.0	12.6	0.70													
D-3	S 5	20.8	42.0	47.7	34.2	13.5	0.58													
D-4	S 2	5.8	38.0	49.3	27.7	21.6	0.48													
D-4	S 3A	10.3	16.8	24.7	23.3	1.4	-4.64													
D-4	S 4A	15.4	8.3		Nonplastic															
D-4	S 4B	16.2	32.8	42.8	25.7	17.1	0.42													
D-4	S 5	20.8	43.4	56.8	41.2	15.6	0.14													
D-4	S 6	25.8	18.0	27.0	21.6	5.4	-0.67													
Count		**102**	**101**	**76**	**76**	**76**	**76**	**19**	**19**	**19**	**19**	**4**	**4**	**4**	**3**	**3**	**3**	**4**	**4**	**4**
Max		25.8	58.2	80.6	46.2	39.7	0.88	90.7	71.4	3.22	0.64	7.2	0.339	0.017	1.3	2.39	11.0	2.1	3.55	15.0
Min		0.8	8.3	24.7	19.5	0.5	-15.6	63.7	40.2	1.38	0.28	5.6	0.252	0.008	0.0	2.03	1.7	1.0	2.21	6.0
Avg		**12.5**	**32.6**	**45.0**	**30.0**	**15.0**	**-0.35**	**78.1**	**55.6**	**2.11**	**0.54**	**6.2**	**0.294**	**0.012**	**0.9**	**2.20**	**5.6**	**1.8**	**3.01**	**10.3**
Mean		10.9	32.6	43.3	27.9	13.0	0.16	78.5	54.1	2.06	0.55	6.0	0.292	0.011	1.3	2.18	4.0	2.1	3.15	10.0

TABLE 3

CTB Borings - Laboratory Test Results on Silts And Clays in Upper Layer

Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits			Saturation	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test					CU Triaxial Test		
					LL	PL	PI							σ_{mpp} (ksf)	CR	RR	C_c	C_r	σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-1	S-1	1.0	4471.4	25.3																	
CTB-1	S-2 (top)	5.1	4467.3	30.1	40.1	22.3	17.8														
CTB-1	S-2 (bot)	6.1	4466.3	65.6																	
CTB-1	U-3C	8.1	4464.3	50.6	56.0	28.9	27.1	0.70			86.4	57.4	1.96								
CTB-1	U-3D	8.7	4463.7	47.9				0.75			91.9	62.1	1.73						1.7	2.84	5.0
CTB-1	U-3E	9.1	4463.3	48.8																	
CTB-1	S-4 (top)	9.5	4462.9	37.4	41.2	23.2	18.0														
CTB-1	S-6	16.0	4456.4	10.7					56.8												
CTB-1	U-7C	21.1	4451.3	51.9	56.5	42.4	14.1	0.68			83.8	55.2	2.08								
CTB-1	U-7D	21.7	4450.7	45.1				0.72			91.2	62.9	1.70						1.7	2.73	5.0
CTB-1	U-7E	22.1	4450.3	43.0																	
CTB-1	S-8	26.0	4446.4	20.9																	
CTB-2	S-2 (bot)	6.3	4467.7	29.4	40.8	21.1	19.7														
CTB-2	S-3	8.0	4466.0	60.1																	
CTB-2	S-4	10.0	4464.0	45.8	56.2	29.9	26.3														
CTB-2	S-5	12.0	4462.0	26.0																	
CTB-2	S-6	16.0	4458.0	27.8	34.3	21.9	12.4														
CTB-2	S-7	21.0	4453.0	28.6																	
CTB-2	S-8	26.0	4448.0	30.0																	
CTB-2	S-9 (top)	30.1	4443.9	26.8																	
CTB-3	S-1	1.0	4471.9	18.7																	
CTB-3	S-2	6.0	4466.9	55.2	58.7	32.3	26.4														
CTB-3	S-3	8.0	4464.9	53.7																	
CTB-3	S-5	12.0	4460.9	39.5																	
CTB-3	S-6 (bot)	16.4	4456.5	24.0																	
CTB-3	S-7 (bot)	21.2	4451.7	53.1																	
CTB-3	S-8	26.0	4446.9	28.3	32.0	22.1	9.9														
CTB-4	S-2 (top)	2.2	4472.8	22.6																	
CTB-4	S-2 (bot)	3.2	4471.8	41.1																	
CTB-4	S-3	5.0	4470.0	27.9	39.9	22.4	17.5														
CTB-4	U-1A	6.0	4469.0	28.9																	

5010.65

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J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE
05996.02	G(B)	05-~~1~~ 2		25F

TABLE 3 CONTINUED

CTB Borings - Laboratory Test Results on Silts And Clays in Upper Layer

Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits LL	PL	PI	Saturation	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test σ_{mpp} (ksf)	CR	RR	C_c	C_r	CU Triaxial Test σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-4	U-1C	7.0	4468.0	34.5				0.68	97.6		95.7	71.2	1.38								
CTB-4	U-1D	7.5	4467.5	60.3	67.9	39.3	28.6	0.62		2.73	74.9	46.7	2.65								
CTB-4	U-1E	7.9	4467.1	64.2																	
CTB-4	U-2D	9.5	4465.5	45.2				0.68			87.7	60.4	1.81						1.7	3.11	6.0
CTB-4	U-2E	9.9	4465.1	48.9	58.1	28.6	29.5	0.79			94.1	63.2	1.69	12.6	0.35	0.020	0.93	0.05			
CTB-4	U-2F	10.1	4464.9	53.0																	
CTB-4	S-6	11.0	4464.0	28.5	34.3	24.8	9.5														
CTB-4	U-7D	13.0	4462.0	22.6				0.60	69.2		101.3	82.7	1.03								
CTB-4	S-8 (top)	14.3	4460.7	20.4																	
CTB-4	S-10	19.0	4456.0	32.7	41.4	24.1	17.3														
CTB-4	U-11D	21.2	4453.8	31.5	37.2	33.5	3.7	0.58	97.2		89.8	68.4	1.48						1.7	3.15	8.0
CTB-4	U-11E	21.6	4453.4	25.0																	
CTB-4	S-12	23.0	4452.0	52.0	57.8	48.1	9.7														
CTB-4	U-13D	25.2	4449.8	37.4	43.2	26.7	16.5	0.78		2.72	101.4	73.8	1.30								
CTB-4	U-13E	25.5	4449.5	40.3																	
CTB-4	S-14	27.0	4448.0	14.8	28.3	18.5	9.8														
CTB-4	U-15C	28.0	4447.0	18.3				0.69			115.5	97.6	0.721								
CTB-4	U-15D	29.2	4445.8	14.4																	
CTB-5	S-2	3.0	4471.8	32.7																	
CTB-5	S-3	5.0	4469.8	72.6	75.3	43.5	31.8														
CTB-5	S-4 (bot)	7.2	4467.6	51.2																	
CTB-5	S-5	9.0	4465.8	48.8	51.5	27.3	24.2														
CTB-5	U-6A	10.0	4464.8	31.7																	
CTB-5	U-6C	10.8	4464.0	12.7				0.40			101.8	90.3	0.860								
CTB-5	U-6D	11.1	4463.7	18.6				0.64			111.3	93.8	0.790								
CTB-5	U-6E	11.3	4463.5	20.0				0.77	79.8		118.0	98.3	0.708								
CTB-5	U-6F	11.5	4463.3	16.4																	
CTB-5	S-9	17.0	4457.8	12.2					63.3												
CTB-5	U-10D	19.4	4455.4	27.7				0.58			94.5	74.0	1.29						1.7	2.93	8.0
CTB-5	U-10E	19.8	4455.0	33.3																	
CTB-5	S-11	21.0	4453.8	47.6	51.5	47.2	4.3														

TABLE 3 CONTINUED

CTB Borings - Laboratory Test Results on Silts And Clays in Upper Layer

Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits LL	PL	PI	Saturation	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test σ_{mpp} (ksf)	CR	RR	C_c	C_r	CU Triaxial Test σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-5	U-12B	23.2	4451.6	42.3				0.73			93.6	65.8	1.58								
CTB-5	U-12C	23.6	4451.2	52.4	51.5	32.8	18.7	0.85			96.4	63.3	1.68	12.3	0.33	0.014	0.89	0.04			
CTB-5	U-12D	23.9	4450.9	45.1				0.75			93.7	64.6	1.63								
CTB-5	U-12E	24.1	4450.7	50.8																	
CTB-5	S-13	25.0	4449.8	33.6	39.8	24.2	15.6														
CTB-5	U-14D	27.0	4447.8	30.5				0.88			113.9	87.2	0.947						1.7	1.66	12.0
CTB-5	U-14E	27.4	4447.4	26.2	30.0	19.5	10.5	0.82			114.7	90.9	0.868	25.5	0.13	0.014	0.25	0.03			
CTB-5	U-14F	27.6	4447.2	27.1																	
CTB-5	S-15 (top)	28.2	4446.6	17.6																	
CTB-5	S-15 (bot)	29.2	4445.6	9.0																	
CTB-6	S-1	1.0	4475.2	20.3																	
CTB-6	S-2	6.0	4470.2	31.0	42.9	21.5	21.4														
CTB-6	U-3A	7.1	4469.1	61.4																	
CTB-6	U-3B	7.6	4468.6	61.1	65.3	32.5	32.8	0.70			81.2	50.4	2.36								
CTB-6	U-3C	7.9	4468.3	56.6				0.77			88.5	56.4	2.01								
CTB-6	U-3D	8.3	4467.9	52.7				0.71			85.7	56.2	2.02						1.7	2.70	7.0
CTB-6	U-3E	8.7	4467.5	55.5																	
CTB-6	S-4 (top)	10.5	4465.7	52.9	56.9	27.9	29.0														
CTB-6	S-4 (bot)	11.5	4464.7	42.1																	
CTB-6	S-5 (top)	15.2	4461.0	10.2																	
CTB-6	S-6	21.0	4455.2	30.7																	
CTB-6	S-7	26.0	4450.2	37.8	41.5	33.9	7.6														
CTB-7	S-1	1.0	4472.1	21.1																	
CTB-7	S-2	6.0	4467.1	52.8	58.1	29.9	28.2														
CTB-7	S-5 (top)	15.2	4457.9	7.4																	
CTB-7	S-5 (bot)	16.2	4456.9	33.6																	
CTB-7	S-6	21.0	4452.1	46.9	51.6	33.5	18.1														
CTB-7	S-7	26.0	4447.1	20.9																	
CTB-8	S-1 (bot)	1.1	4472.8	31.8																	
CTB-8	S-2	6.0	4467.9	53.3	55.3	28.5	26.8														
CTB-8	S-3	8.0	4465.9	24.1																	

CTB Borings - Laboratory Test Results on Silts And Clays in Upper Layer

Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Atterberg Limits			Saturation	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio	Consolidation Test					CU Triaxial Test		
					LL	PL	PI							σ_{mpp} (ksf)	CR	RR	C_c	C_r	σ_c (ksf)	S_u (ksf)	ε_a (%)
CTB-8	S-7 (bot)	21.1	4452.8	57.0																	
CTB-8	S-8	26.0	4447.9	26.7	30.5	18.3	12.2														
CTB-N	U-1A	5.1	4469.0	30.6	38.4	23.1	15.3														
CTB-N	U-1B	5.7	4468.4	30.1	41.3	22.5	18.8	0.68			100.6	77.3	1.20						1.7	3.00	8.0
CTB-N	U-1D	6.7	4467.4	46.6	50.8	23.1	27.7														
CTB-N	U-1E	6.9	4467.2	67.7																	
CTB-N	U-2A	7.1	4467.0	69.0	74.2	45.4	28.8														
CTB-N	U-2B	7.7	4466.4	65.4				0.64			74.6	45.1	2.76						1.7	2.41	13.0
CTB-N	U-2C	8.3	4465.8	52.6				0.71			86.3	56.5	2.01								
CTB-N	U-2D	8.7	4465.4	63.0	60.6	36.8	23.8	0.68			78.8	48.4	2.51	6.1	0.37	0.020	1.31	0.07			
CTB-N	U-2E	8.8	4465.3	52.1																	
CTB-N	U-3A	9.0	4465.1	53.7																	
CTB-N	U-3C	9.9	4464.2	47.1				0.67			86.1	58.5	1.90								
CTB-N	U-3D	10.5	4463.6	52.2	61.1	30.8	30.3	0.72		2.71	86.3	56.7	1.98						1.7	2.73	7.0
CTB-N	U-3E	10.9	4463.2	53.1																	
CTB-S	U-1A	5.1	4469.4	85.5																	
CTB-S	U-1AA	5.3	4469.2	84.1	82.7	44.8	37.9	0.70			73.2	39.8	3.28								
CTB-S	U-1B	5.8	4468.7	73.6	66.2	40.9	25.3	0.72			78.0	44.9	2.78						1.7	2.05	12.0
CTB-S	U-1D	6.6	4467.9	60.7				0.74			84.8	52.8	2.22								
CTB-S	U-1E	6.9	4467.6	56.4																	
CTB-S	U-2D	8.4	4466.1	54.6	57.9	28.9	29.0	0.77			90.0	58.2	1.92						1.7	2.40	5.0
CTB-S	U-2E	8.8	4465.7	56.7																	
CTB-S	U-3C	10.1	4464.4	72.2	66.0	37.8	28.2	0.87	99.2	2.72	89.5	51.9	2.27	8.4	0.36	0.020	1.17	0.07			
CTB-S	U-3F	10.9	4463.6	31.2																	
		117	count	117	42	42	42	35		4	35	35	35	5	5	5	5	5	12	12	12
		30.1	max	85.5	82.7	48.1	37.9	0.88		2.73	118.0	98.3	3.28	25.5	0.37	0.020	1.31	0.07	1.7	3.15	13.0
		1.0	min	7.4	28.3	18.3	3.7	0.40		2.71	73.2	39.8	0.71	6.1	0.13	0.014	0.25	0.03	1.7	1.66	5.0
		13.4	avg	40.1	50.6	30.1	20.5	0.71		2.72	92.4	65.2	1.75	13.0	0.31	0.018	0.91	0.05	1.7	2.64	8.0
		10.1	mean	39.5	51.5	28.8	19.3	0.71		2.72	90.0	62.1	1.73	12.3	0.35	0.020	0.93	0.05	1.7	2.73	7.5

TABLE 3 CONTINUED

TABLE 4

CTB Borings - Laboratory Test Results on Sands in 8 - 20 ft Depth

Boring	Sample	Average Depth (ft)	Elevation (ft)	Water Content (%)	Saturation	USC Code	% Fines	Specific Gravity	Wet Density (pcf)	Dry Density (pcf)	Void Ratio
CTB-1	**S-6**	16.0	4456.4	10.7		ML					
CTB-3	**S-6 (top)**	15.4	4457.5	14.6		SM					
CTB-4	**U-7E**	13.2	4461.8	10.2		SP					
CTB-4	**S-8 (bot)**	15.4	4459.6	5.4		SM	37.5				
CTB-4	**U-9A**	16.0	4459.0	4.6		ML					
CTB-4	**U-9D**	16.7	4458.3	4.5		SM		2.69			
CTB-4	**U-9E**	16.9	4458.1	5.2	0.18	SM	16.7		98.4	93.5	0.80
CTB-4	**U-9F**	17.1	4457.9	9.7	0.32	SM	34.2		101.0	92.1	0.82
CTB-4	**U-9H**	17.5	4457.5	6.6		SM					
CTB-5	**S-7**	13.0	4461.8	4.1		SM	21.6				
CTB-5	**U-8A**	14.0	4460.8	3.7		SM					
CTB-5	**U-8D**	15.4	4459.4	3.4	0.14	SM			105.8	102.4	0.64
CTB-5	**U-8E**	15.6	4459.2	6.5		SM					
CTB-6	**S-5 (bot)**	16.2	4460.0	5.6		SM					
CTB-7	**U-3D**	8.3	4464.8	2.7	0.11	SP	8.7	2.69	102.3	99.6	0.69
CTB-7	**U-3E**	8.5	4464.6	2.6		SP					
CTB-7	**S-4**	11.0	4462.1	6.4		SM					
CTB-7	**S-5 (top)**	15.2	4457.9	7.4		ML					
CTB-8	**S-4**	10.0	4463.9	3.6		SM	14.8				
CTB-8	**S-5**	12.0	4461.9	3.0		SM					
CTB-8	**S-6**	16.0	4457.9	5.5		SM	34.8				
CTB-S	**U-3D**	10.4	4464.1	10.0	0.23	SM	18.9		84.7	77.0	1.18
		22	count	22	5		8	2	5	5	5
		17.5	max	14.6	0.32		37.5	2.69	105.8	102.4	1.18
		8.3	min	2.6	0.11		8.7	2.69	84.7	77.0	0.64
		14.1	avg	6.2	0.19		23.4	2.69	98.4	92.9	0.83
		15.4	mean	5.5	0.18		20.3	2.69	101.0	93.5	0.80

TABLE 5
Direct Shear Test Results

Boring	Sample	Depth (ft)	Elevation (ft)	Atterberg Limits			USC Code	Water Content (%)	Initial			After Consolid.		Normal Stress (ksf)	Peak Shear (ksf)	Cohesion (ksf)	Tan ϕ	ϕ (deg)
				LL	PL	PI			γ_m (pcf)	γ_d (pcf)	Void Ratio	γ_d (pcf)	Void Ratio					
C-2	U-1C1	5.7	4458.5	76.9	39.1	37.8	MH	55.7	69.4	44.50	2.81	45.1	2.76	3.0	2.60	1.22	0.465	24.9
C-2	U-1C2	5.9	4458.3					58.2	63.7	40.20	3.22	40.5	3.19	2.0	2.17			
C-2	U-1C3	6.0	4458.2					52.7	75.1	49.2	2.45	49.3	2.44	1.0	1.67			
CTB-6	U-3B1	7.2	4469.0	65.3	32.5	32.8	MH	61.7	74.7	46.2	2.68	46.5	2.65	1.0	1.01	1.26	0.375	20.6
CTB-6	U-3B3	7.5	4468.7					61.3	81.9	50.7	2.35	51.2	2.32	2.0	2.15			
CTB-6	U-3B4	7.7	4468.5					60.3	80.5	50.2	2.38	50.9	2.34	3.0	2.32			
CTB-6	U-3C	7.8	4468.4					56.6	88.5	56.4	2.01	56.7	2.00	1.0	1.57			
CTB-S	U-1AA3	5.1	4469.4	82.7	44.8	37.9	MH	80.9	75.7	41.8	3.06	42.6	2.98	3.0	2.24	1.00	0.397	21.6
CTB-S	U-1AA2	5.3	4469.2					84.6	73.1	39.6	3.29	39.9	3.25	2.0	1.75			
CTB-S	U-1AA1	5.4	4469.1					86.8	70.9	37.9	3.48	38.1	3.45	1.0	1.42			
CTB-S	U-1C	6.1	4468.4	79.0	44.8	34.2	MH	69.2	78.8	46.5	2.65	46.6	2.64	0.25	1.05			

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE
05996.02	G(B)	05-2		25K

5010.65

TABLE 6
SUMMARY OF TRIAXIAL TEST RESULTS FOR SOILS WITHIN ~10 FT OF GROUND SURFACE AT THE SITE

Boring	Sample	Depth ft	Elev ft	w %	ATTERBERG LIMITS LL	PL	PI	USC Code	γ_m pcf	γ_d pcf	e_o	σ_c ksf	s_u ksf	ε_a %	Type	Date
B-1	U-2C	5.9	4453.9	47.1	66.1	33.4	32.7	MH	79.3	53.9	2.15	0.0	2.03	1.7	CU	Nov '99
B-1	U-2B	5.3	4454.5	52.9	80.6	40.9	39.7	MH	70.8	46.3	2.67	1.0	2.21	6.0	CU	Nov '99
B-4	U-3D	10.4	4462.1	27.4	42.5	24.7	17.8	CL	85.5	67.1	1.53	1.3	2.18	4.0	UU	Jan '97
C-2	U-2D	11.1	4453.4	35.6	See U-2C & E[1]			CL	78.5	57.9	1.93	1.3	2.39	11.0	UU	Jan '97
CTB-1	U-3D	8.7	4463.7	47.9	See U-3C[2]			CH	91.9	62.1	1.73	1.7	2.84	5.0	CU	June '99
CTB-4	U-2D	9.5	4465.5	45.2	See U-2E[2]			CH	87.7	60.4	1.81	1.7	3.11	6.0	CU	June '99
CTB-6	U-3D	8.3	4467.9	52.7				CH	85.7	56.2	2.02	1.7	2.70	7.0	CU	June '99
CTB-N	U-1B	5.7	4468.4	30.1	41.3	22.5	18.8	CL	100.6	77.3	1.20	1.7	3.00	8.0	CU	Nov '98
CTB-N	U-2B	7.7	4466.4	65.4	See U-2A[2]			MH	74.6	45.1	2.76	1.7	2.41	13.0	CU	June '99
CTB-N	U-3D	10.5	4463.6	52.2	61.1	30.8	30.3	CH	86.3	56.7	1.98	1.7	2.73	7.0	CU	June '99
CTB-S	U-1B	5.8	4468.7	73.6	66.2	40.9	25.3	MH	78.0	44.9	2.78	1.7	2.05	12.0	CU	Nov '98
CTB-S	U-2D	8.4	4466.1	54.6	57.9	28.9	29.0	CH	90.0	58.2	1.92	1.7	2.40	5.0	CU	June '99
B-1	U-2D	6.5	4453.3	45.2	59.8	34.7	25.1	MH	76.7	52.8	2.22	2.1	3.26	15.0	CU	Mar '99
B-3	U-1B	5.2	4463.0	33.5	52.4	25.2	27.2	MH	90.6	67.9	1.50	2.1	3.55	8.0	CU	Mar '99
C-2	U-1D	6.3	4458.2	50.5	70.3	41.3	29.0	MH	74.5	49.5	2.43	2.1	3.03	12.0	CU	Mar '99

NOTES
1 Attachment 2 of SAR Appendix 2A.
2 Attachment 6 of SAR Appendix 2A.

[geotl]\05996\calc\G(B)\05-2\Table_6.xls

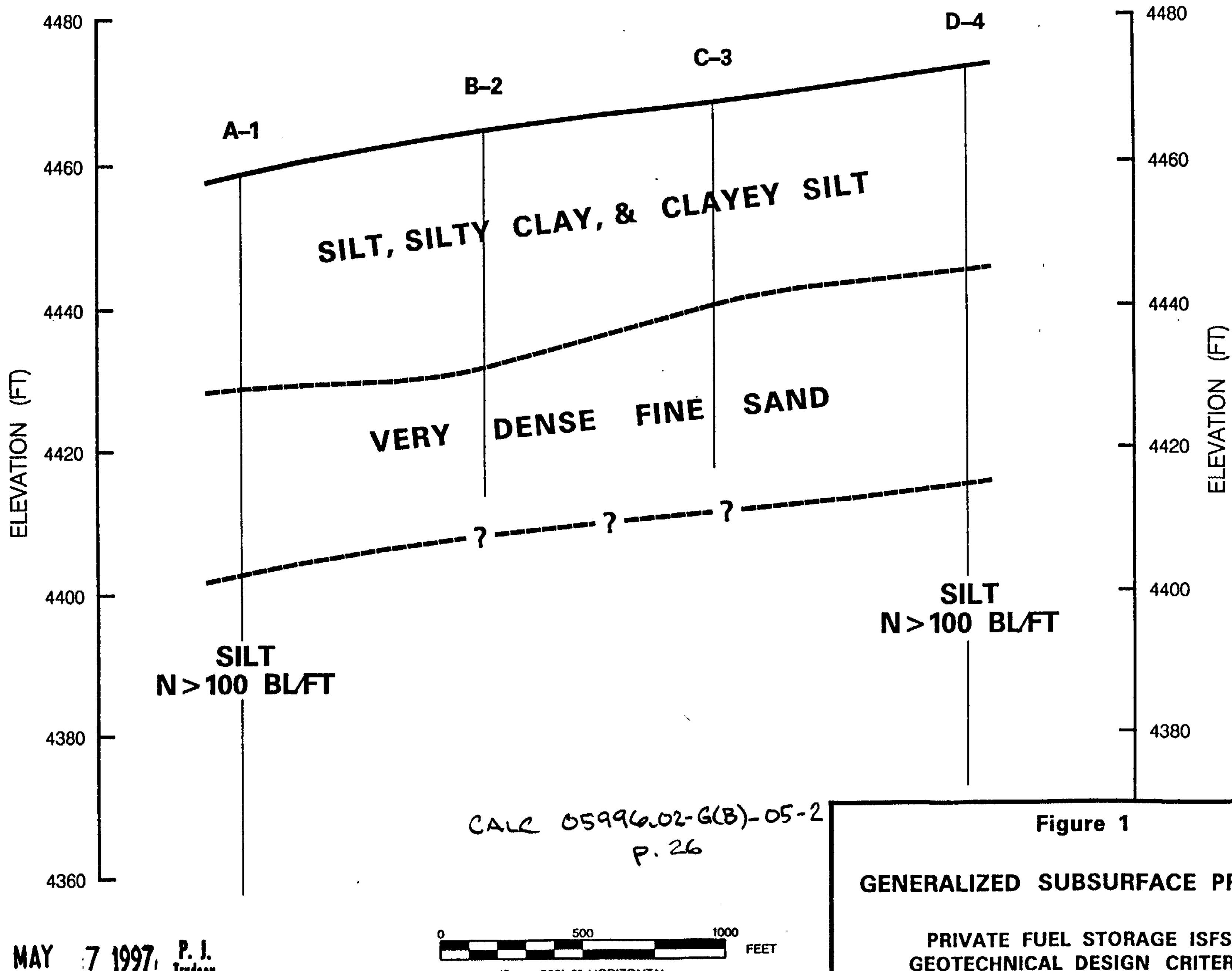


Figure 1

GENERALIZED SUBSURFACE PROFILE

PRIVATE FUEL STORAGE ISFSI
GEOTECHNICAL DESIGN CRITERIA

NOTED MAY 7 1997 P. J. Trudeau

Revision 0

01010

NOTED FEB 28 1997 P. J. Trudeau

STONE & WEBSTER ENGINEERING CORPORATION

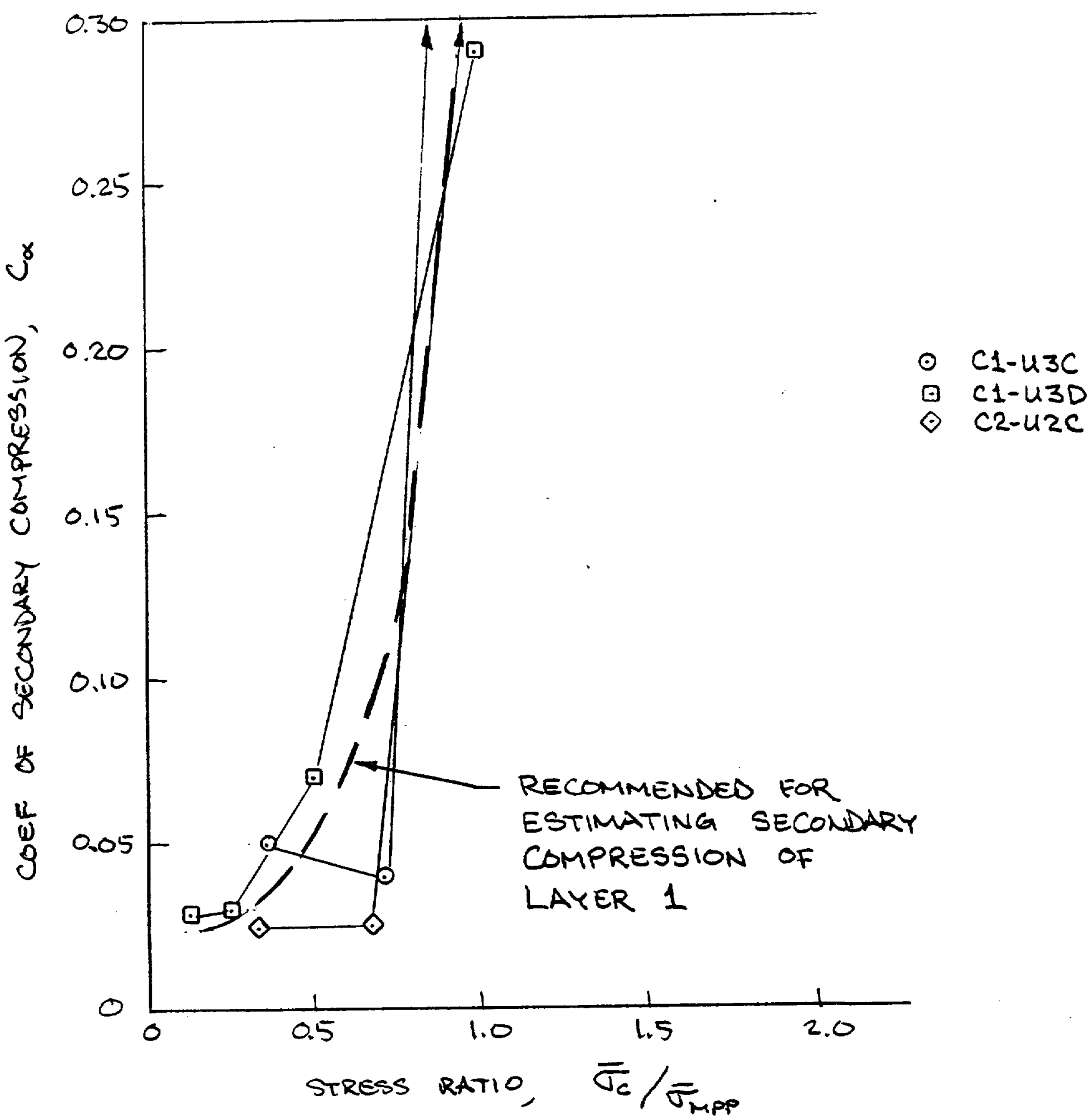
CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 05-2	OPTIONAL TASK CODE	PAGE 27

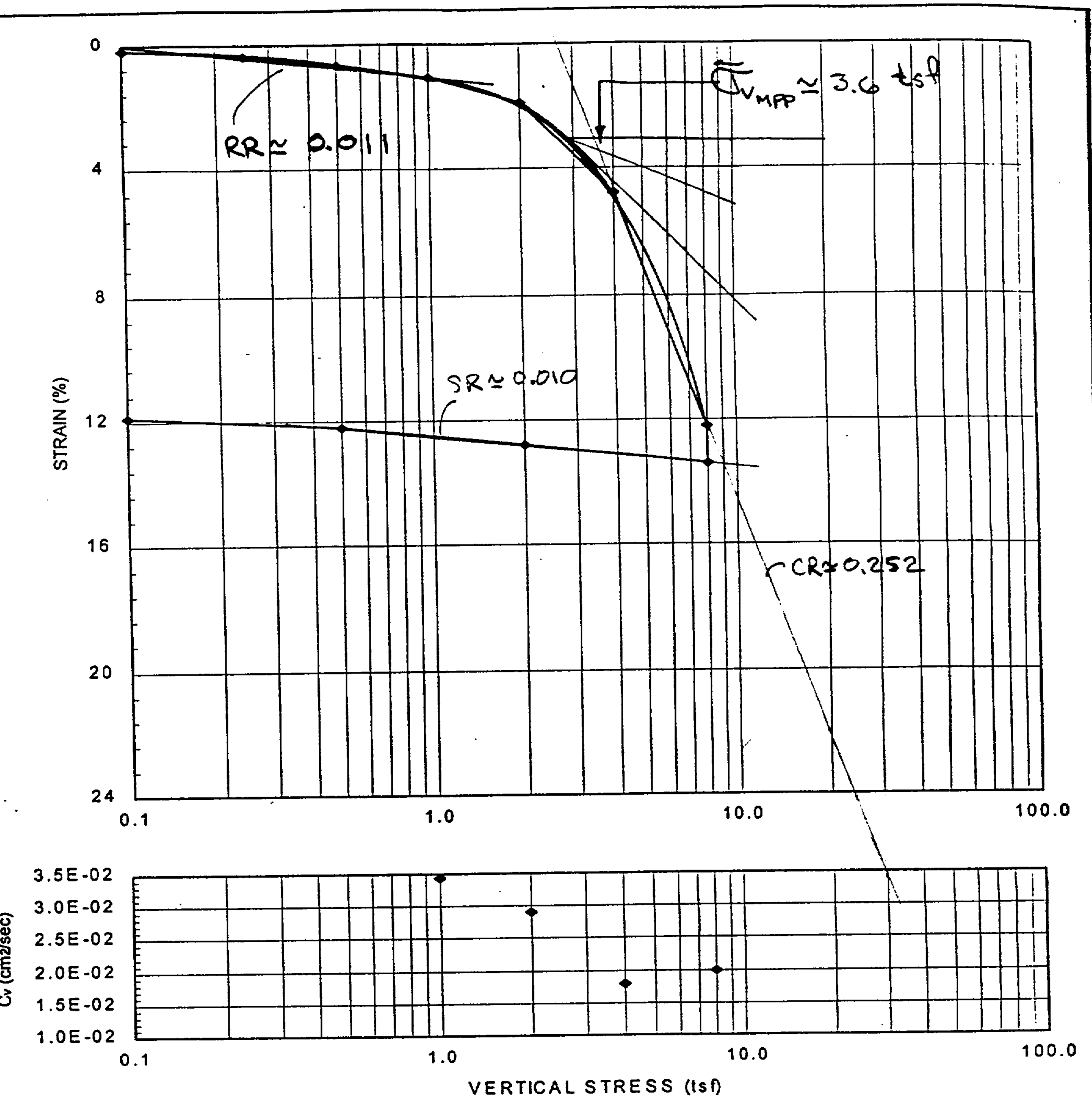
FIGURE 2

RATE OF SECONDARY COMPRESSION VS STRESS RATIO



BASED ON RELOADING PORTIONS OF CONSOLIDATION TESTS - SEE pp 14-18, TABLES 2A, 2B, 2C, AND FIGURES 5, 6A, 6B, & 6C IN CALC 05996.01-G(B)-03-1

~~01011~~



SAMPLE INFORMATION:

BORING:	C-1	DATE:	1/9/97
SAMPLE:	U-3B	TESTED BY:	ACS
DEPTH:	10.8 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	30.3 %	28.7 %
DRY UNIT WEIGHT:	64.7 pcf	73.4 pcf
VOID RATIO:	1.625	1.315
SATURATION:	50.7 %	59.3 %

SPECIFIC GRAVITY: 2.72 (est)

NOTE: Sample was not inundated and porous stones were dry

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FIGURE 3
CALC 05996.02-G(B)-05-2 p 28

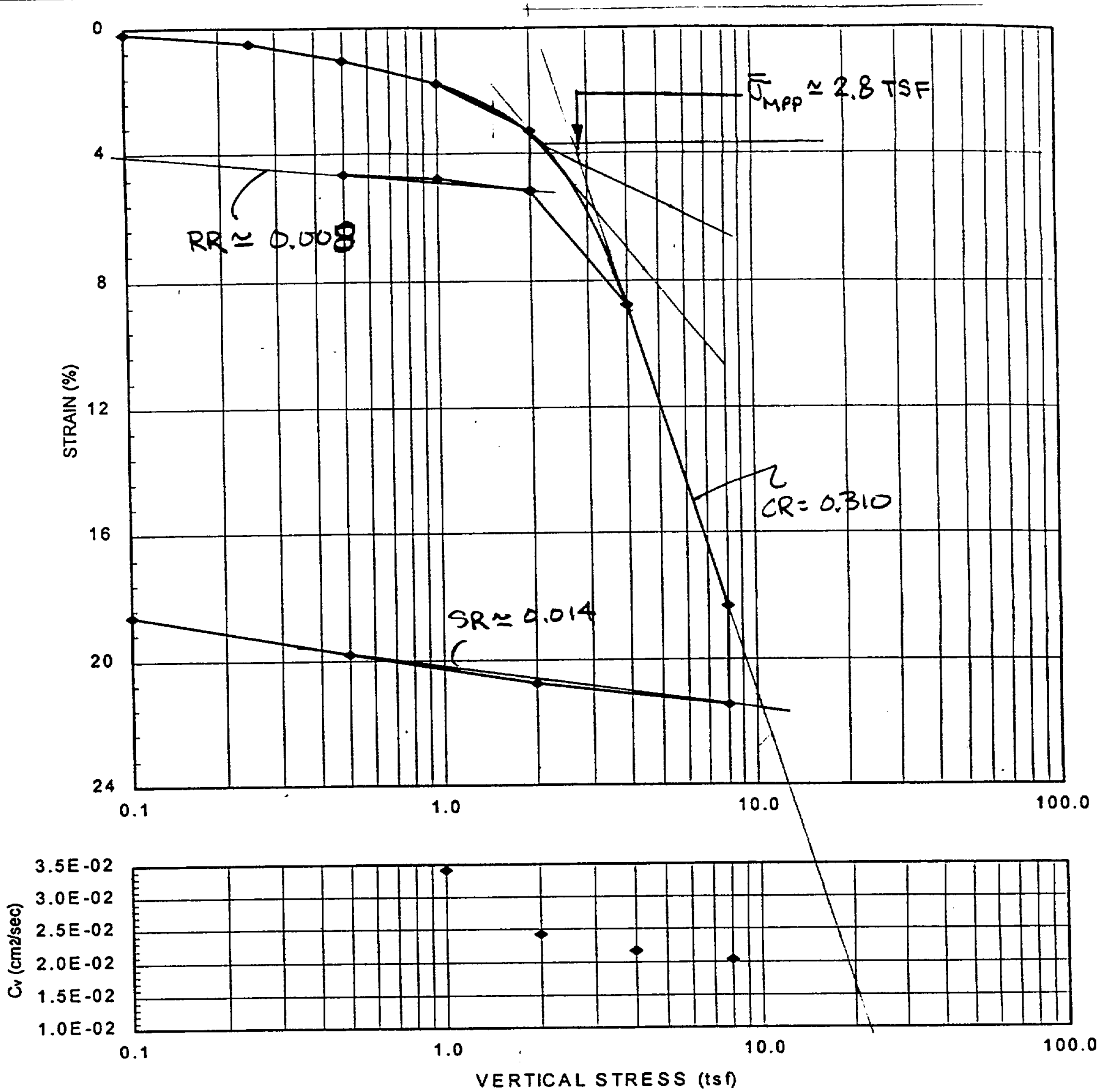
PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP.
BOSTON, MASSACHUSETTS

JO 05996.01

CONSOLIDATION TEST RESULTS
BORING C-1, SAMPLE U-3B

~~01012~~



SAMPLE INFORMATION:

BORING:	C-1	DATE:	12/20/96
SAMPLE:	U-3C	TESTED BY:	ACS
DEPTH:	11.2 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:	INITIAL	FINAL	SPECIFIC GRAVITY:
WATER CONTENT:	38.9 %	51.9 %	2.72
DRY UNIT WEIGHT:	55.8 pcf	68.4 pcf	
VOID RATIO:	2.041	1.484	
SATURATION:	51.8 %	95.2 %	

NOTE: Sample was not inundated and porous stones were moist

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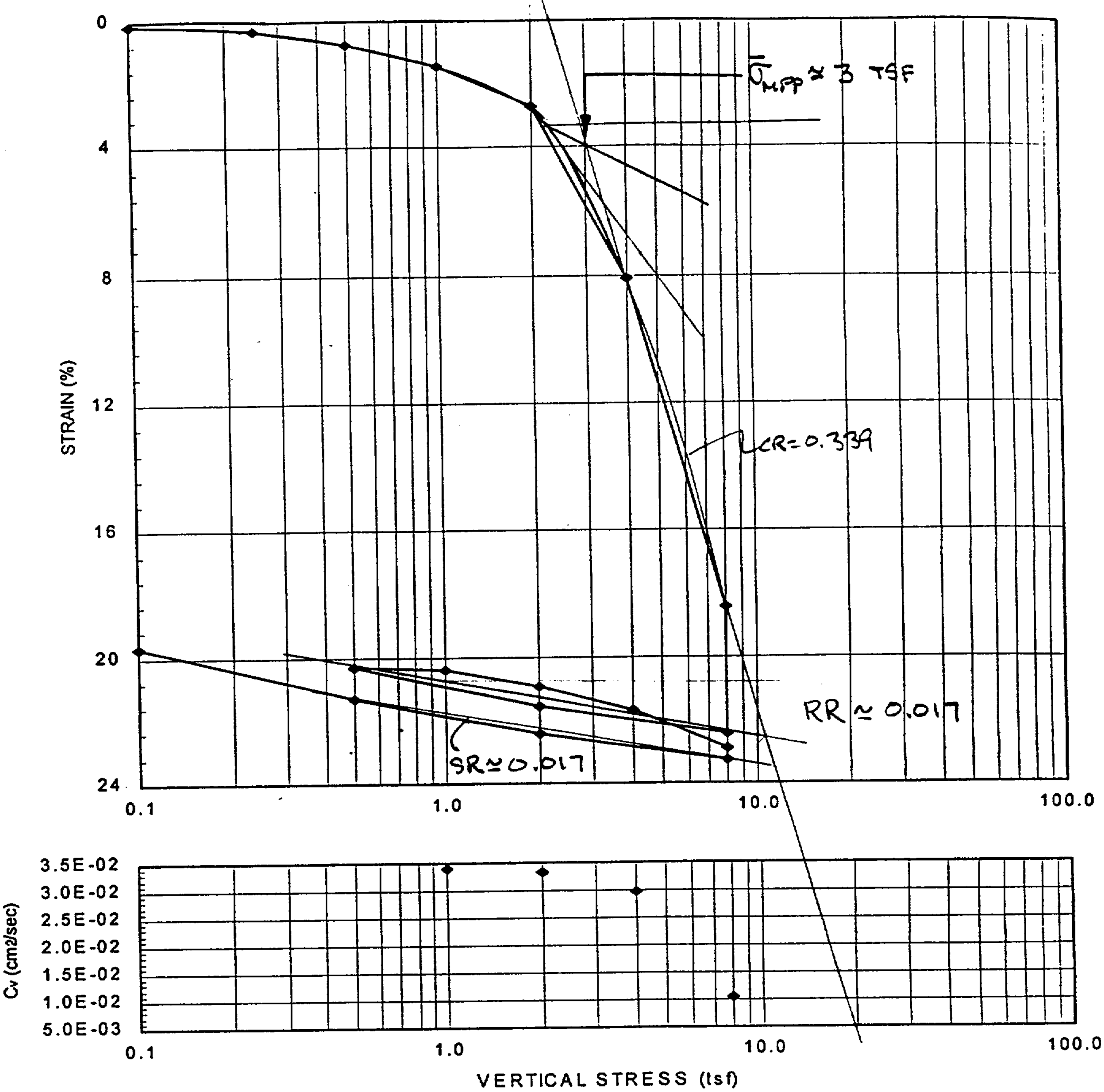
FIGURE 4

CALC 05996.02-G(B)-05-2 p29

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP.
BOSTON, MASSACHUSETTS

CONSOLIDATION TEST RESULTS ~~01013~~
BORING C-1, SAMPLE U-3C



SAMPLE INFORMATION:

BORING:	C-1	DATE:	12/12/96
SAMPLE:	U-3D	TESTED BY:	ACS
DEPTH:	11.4 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL
WATER CONTENT:	46.7 %	62.4 %
DRY UNIT WEIGHT:	51.7 pcf	64.1 pcf
VOID RATIO:	2.285	1.649
SATURATION:	55.6 %	103.0 %

SPECIFIC GRAVITY: 2.72

NOTE: Sample was inundated when the applied pressure was 0.5 tsf.

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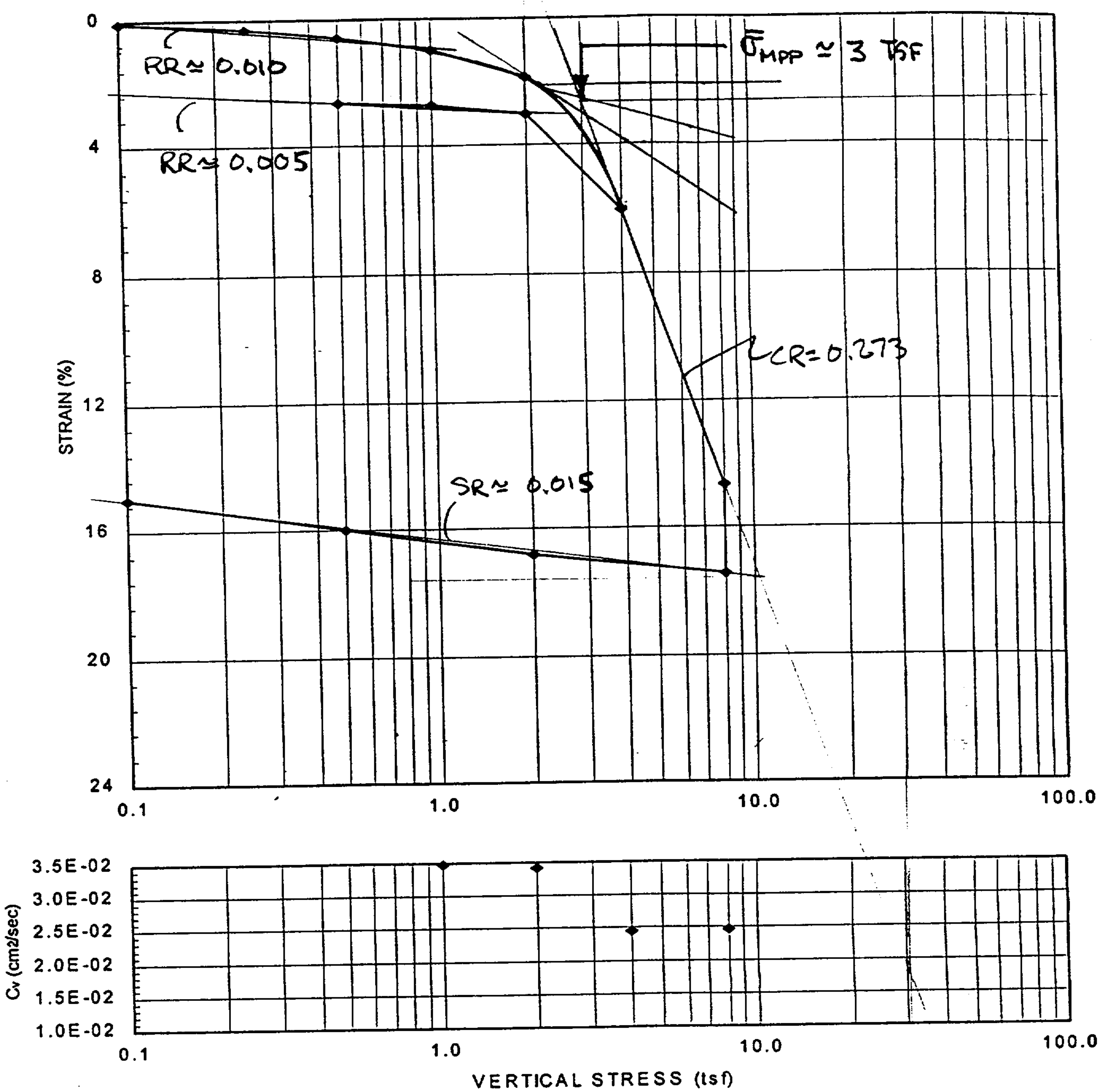
PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

FIGURE 5
CALC 05996.02-G(B)-05-2 p.30

STONE & WEBSTER ENGINEERING CORP.
BOSTON, MASSACHUSETTS

CONSOLIDATION TEST RESULTS
BORING C-1; SAMPLE U-3D

~~01014~~



SAMPLE INFORMATION:

BORING:	C-2	DATE:	12/17/96
SAMPLE:	U-2C	TESTED BY:	ACS
DEPTH:	10.9 ft	CHECKED:	PJT
DESCRIPTION:	Clayey SILT		

SPECIMEN INFORMATION:

	INITIAL	FINAL	SPECIFIC GRAVITY
WATER CONTENT:	27.6 %	44.2 %	2.72 (est)
DRY UNIT WEIGHT:	64.9 pcf	76.2 pcf	
VOID RATIO:	1.615	1.230	
SATURATION:	46.4 %	97.7 %	

NOTE: Sample was inundated when the applied pressure was 0.5 tsf.

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FIGURE 6
CALC 05996.02-G(B)-05-2 p 31

PRIVATE FUEL STORAGE FACILITY
SKULL VALLEY
PRIVATE FUEL STORAGE, LLC

STONE & WEBSTER ENGINEERING CORP.
BOSTON, MASSACHUSETTS

CONSOLIDATION TEST RESULTS
BORING C-2, SAMPLE U-2C

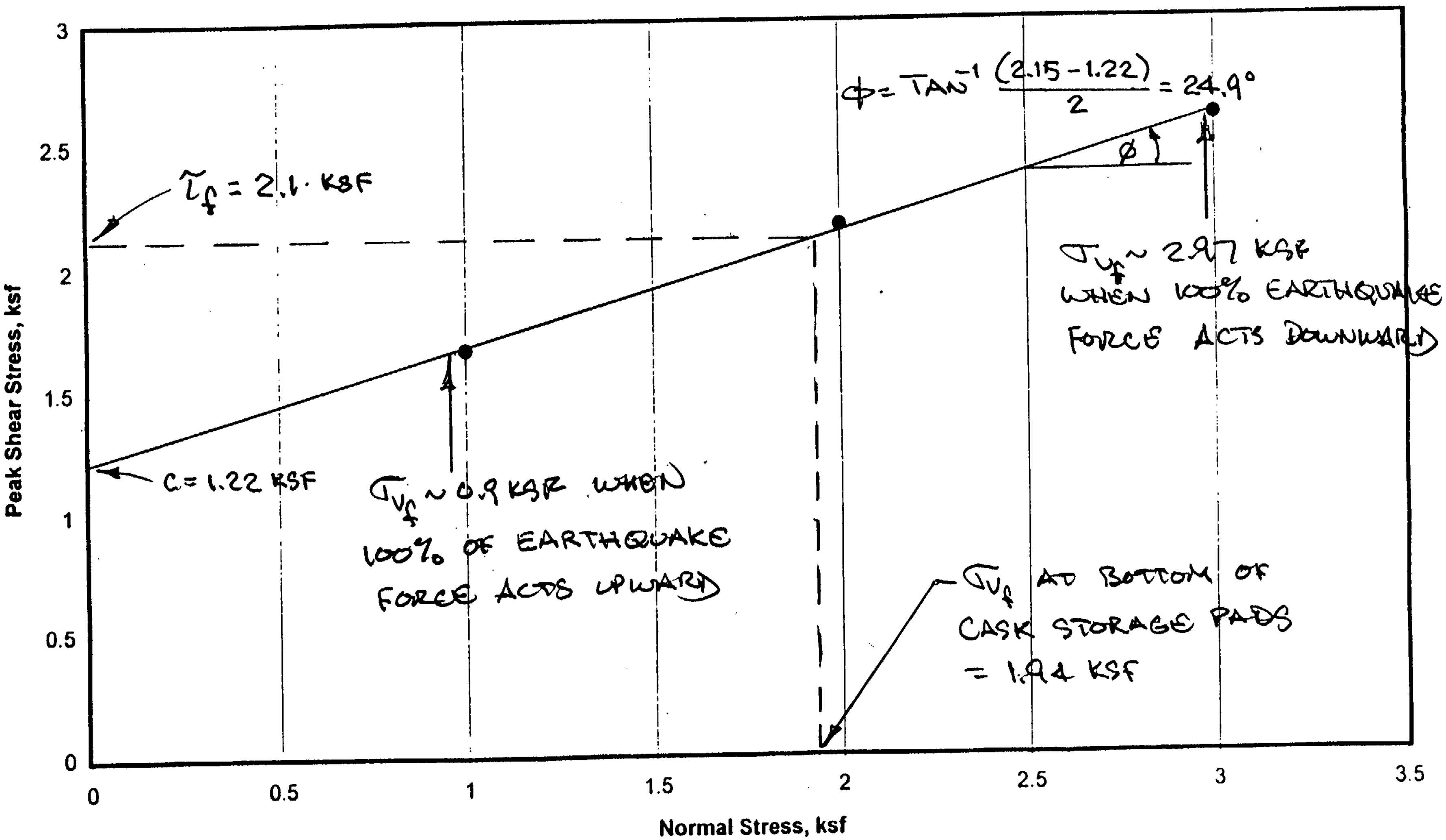
~~0101~~

FIGURE 7

DIRECT SHEAR TEST

Boring C-2, Sample U-1C

PAD EMPLACEMENT AREA



CALC 05996.02-G(B)-05-~~1~~ 2

p 32

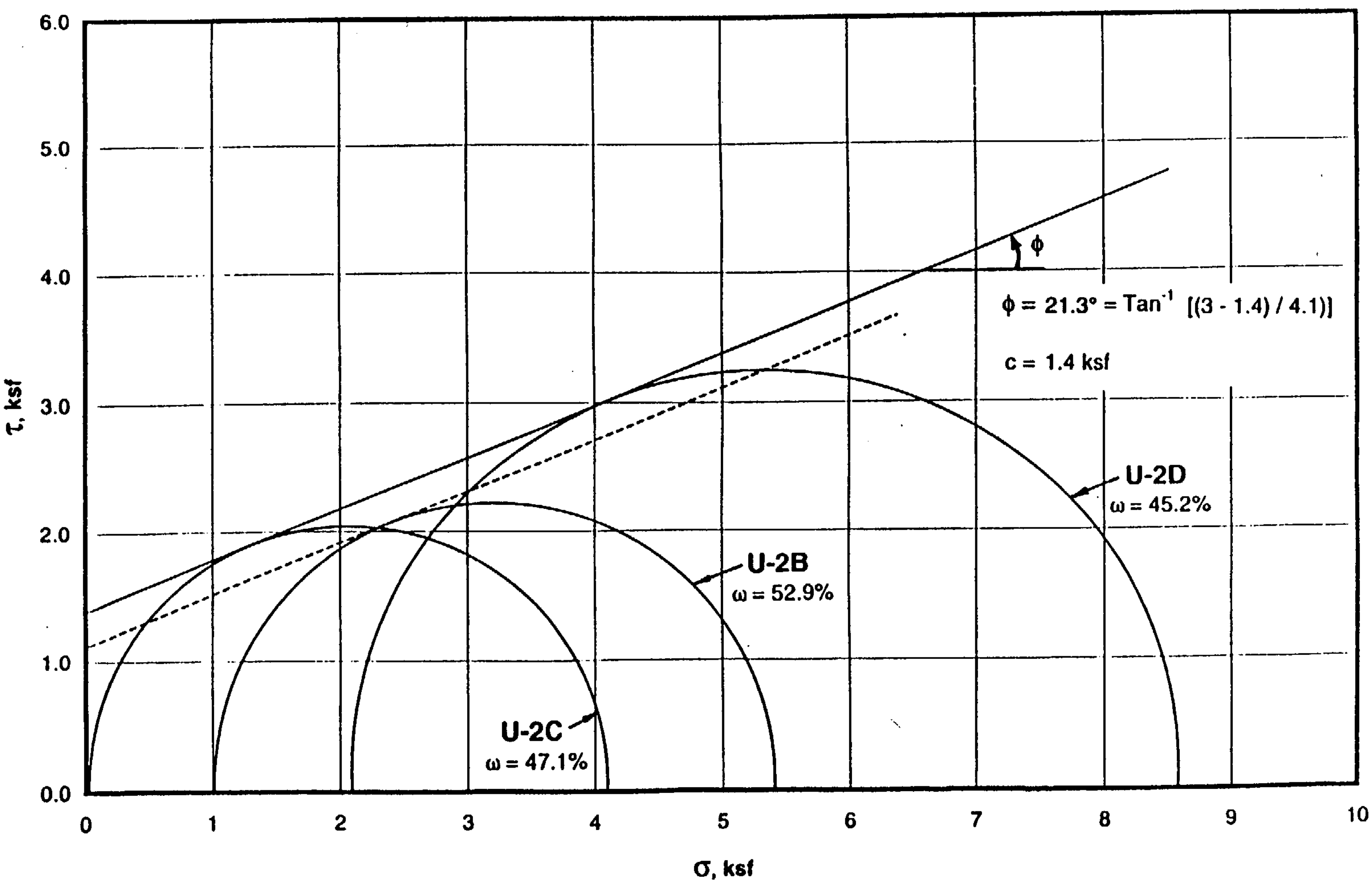
REF SAR APP 2A ATT 7

STONE & WEBSTER ENGINEERING CORPORATION

FIGURE 8

Total Stress Mohr's Circles

Boring B-1, Sample U-2



CALC 05996.02-G(B)-05-2 P 33

Private Fuel Storage, LLC
PFSF, Skull Valley, UT

JO 05996.02
November 1999

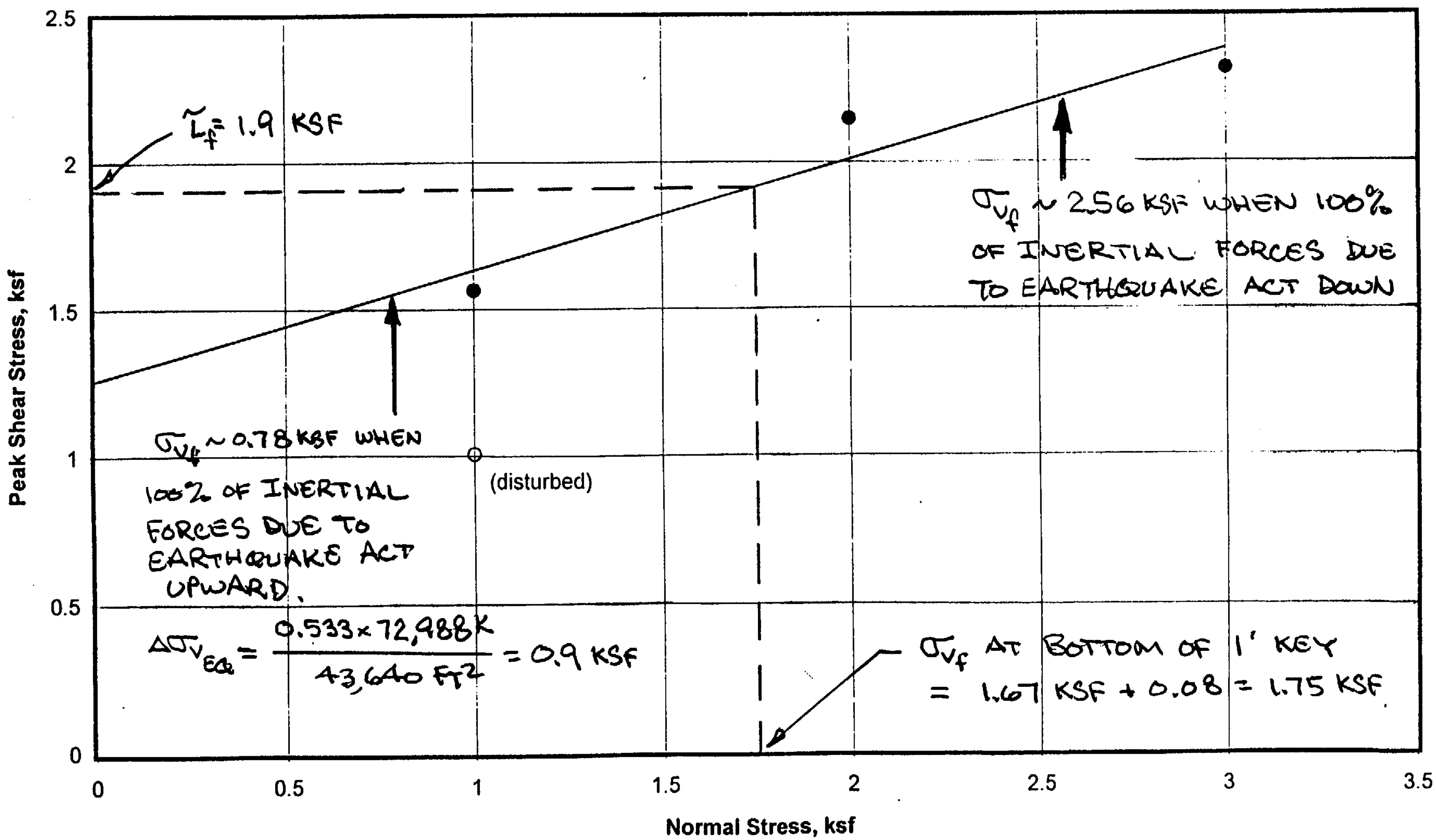
REF: SAR APP 2A ATT 8

FIGURE 9

DIRECT SHEAR TEST

Boring CTB-6, Sample U-3B&C

CANISTER TRANSFER BUILDING AREA



CALC 05996.02-G(B)-05-2

P 34

PFSF SKULL VALLEY, UT

SAR APP 2A ATT 7

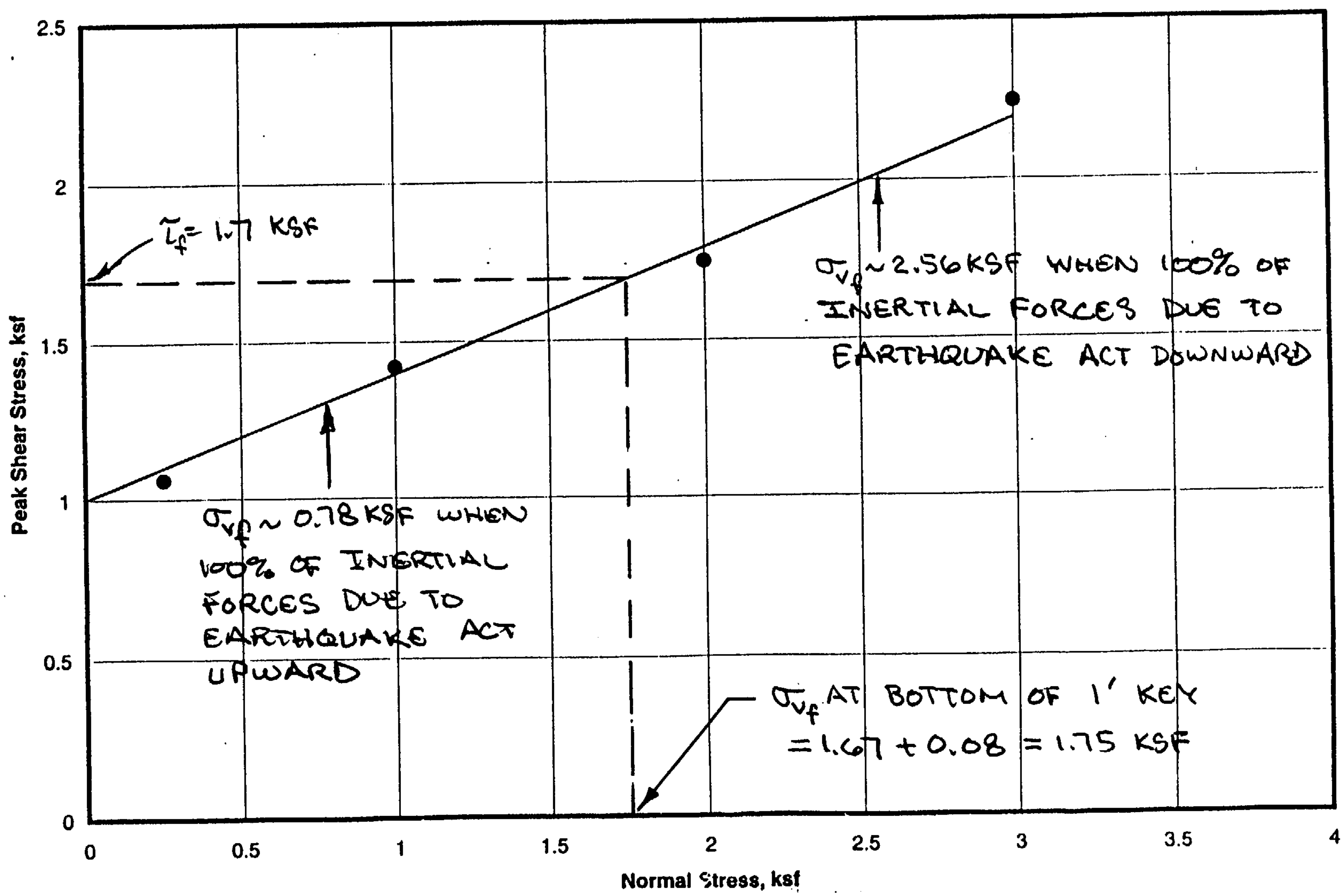
STONE & WEBSTER ENGINEERING CORPORATION

FIGURE 10

DIRECT SHEAR TEST

Boring CTB-S, Sample U-1AA&C

CANISTER TRANSFER BUILDING AREA

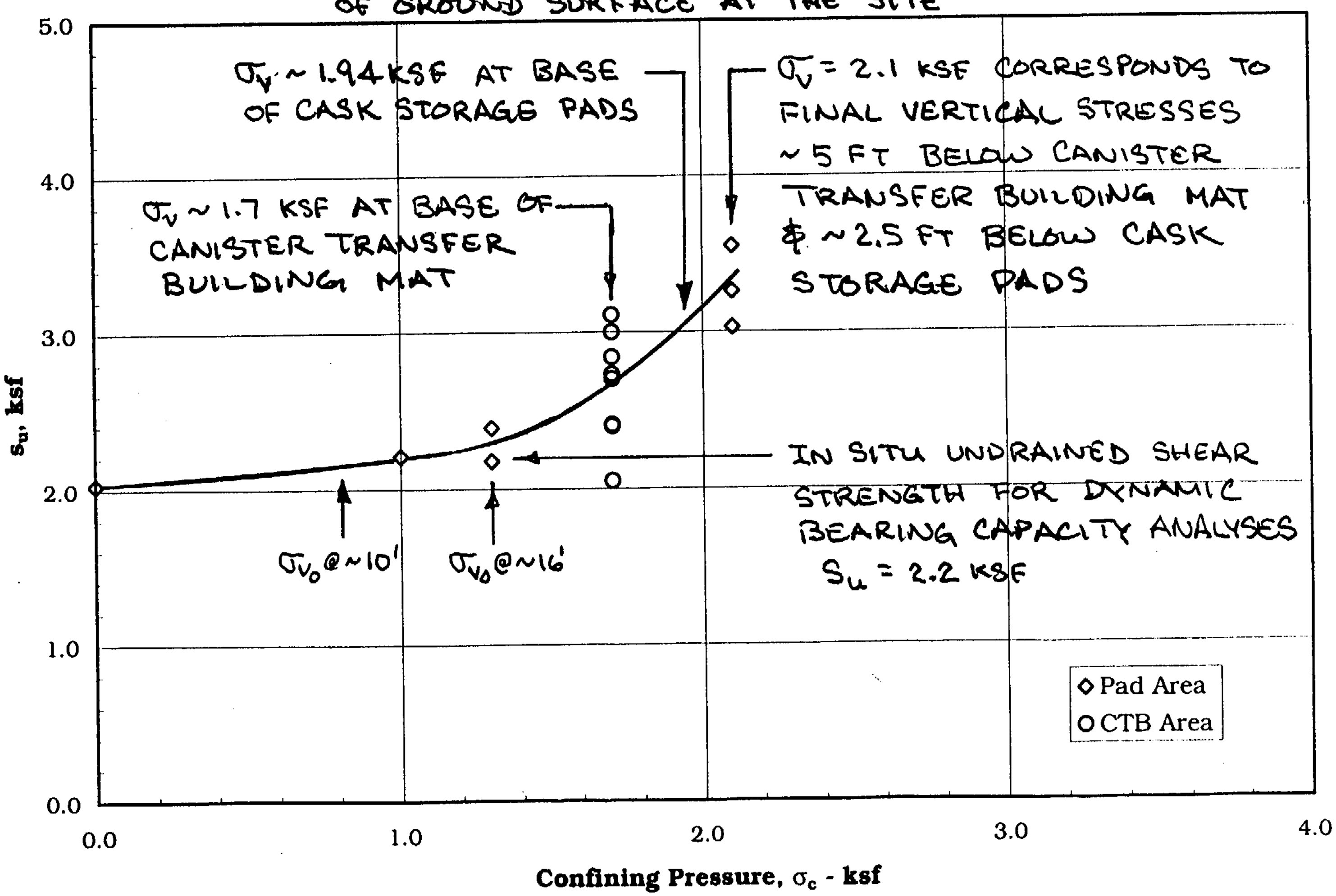


CALC 05996.02-G(B)-05-2 P 35

Private Fuel Storage, LLC
PFSF, Skull Valley, UT
SAR APP 2A ATT 8

JO 05996.02
November 1999

Figure 11
Summary of Triaxial Test Results for Soils Within Depth of ~10 ft OF GROUND SURFACE AT THE SITE



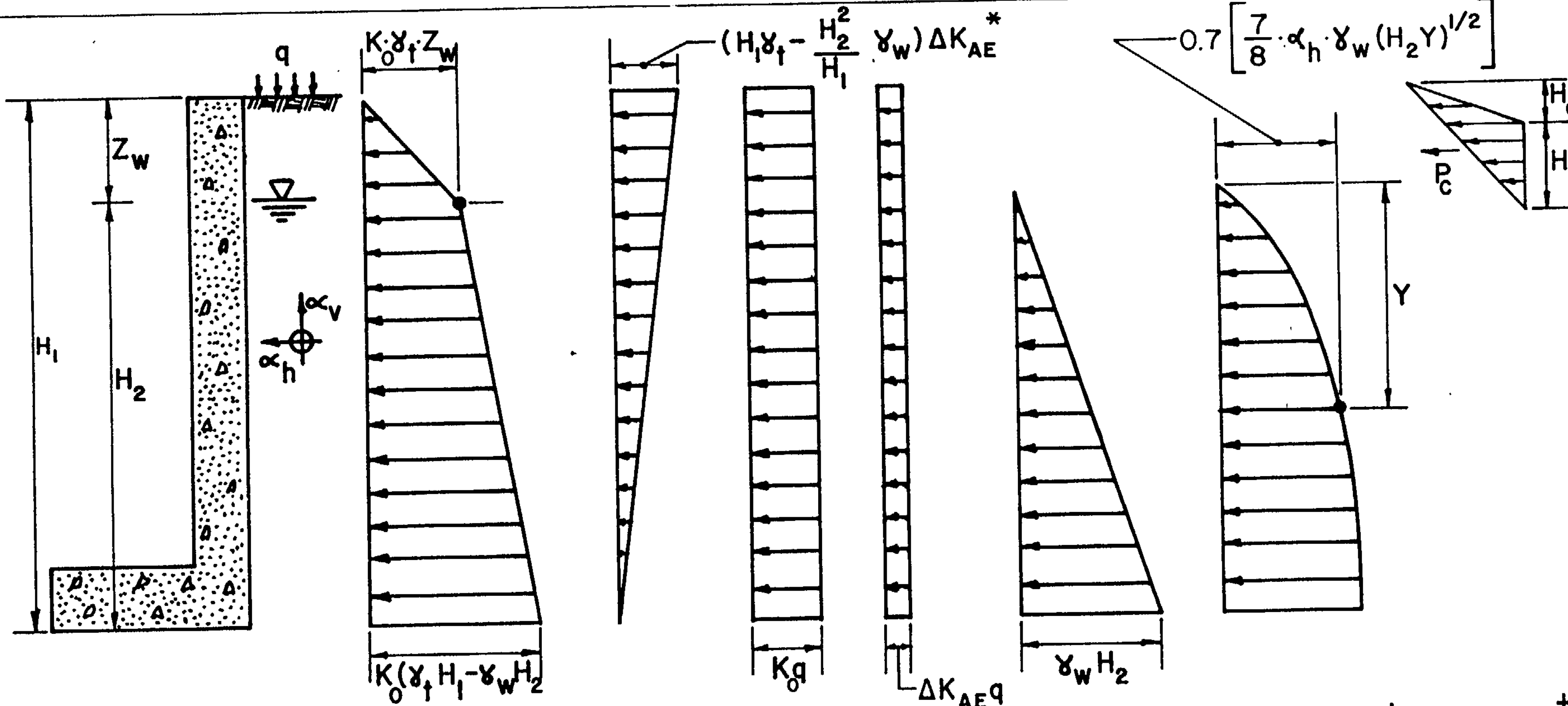
[geot]\05996\lab\triax-1.xls on 6/15/2000

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE
05996.02	G(B)	05-2		36

5010.65



LOAD/LIN. FT. OF WALL	SOIL	SURCHARGE**	WATER	COMPACTION+
STATIC	$(H_1^2\gamma_t - H_2^2\gamma_w)\frac{K_o}{2}$	$K_o q H_1$	$\frac{1}{2}H_2^2\gamma_w$	$\frac{1}{2}\sigma_c H_{c_2}$
DYNAMIC	$(H_1^2\gamma_t - H_2^2\gamma_w)\frac{1}{2}\Delta K_{AE}$*	$\Delta K_{AE} q H_1$	$0.7\left[\frac{7}{12}\alpha_h\gamma_w H_2^2\right]$	—
COMBINED	$\frac{1}{2}(H_1^2\gamma_t - H_2^2\gamma_w)(K_o+\Delta K_{AE})$	$q(K_o+\Delta K_{AE})H_1$	$\frac{\gamma_w H_2}{2}(1+0.82\alpha_h)$	$\frac{1}{2}\sigma_c H_{c_2}$

*APPLICABLE FOR $H_2 \le H_1$; IF $H_2 > H_1$, H_2 SHOULD BE TAKEN TO BE EQUAL TO H_1 SINCE STANDING WATER DOES NOT EFFECT THE MAGNITUDE OF EFFECTIVE STRESS. FOR $\alpha_h > 0.4$, USE EQ 1 OF SEED & WHITMAN (1970) TO CALCULATE KAE.

[1]

**FOR UNIFORM SURCHARGE ONLY.

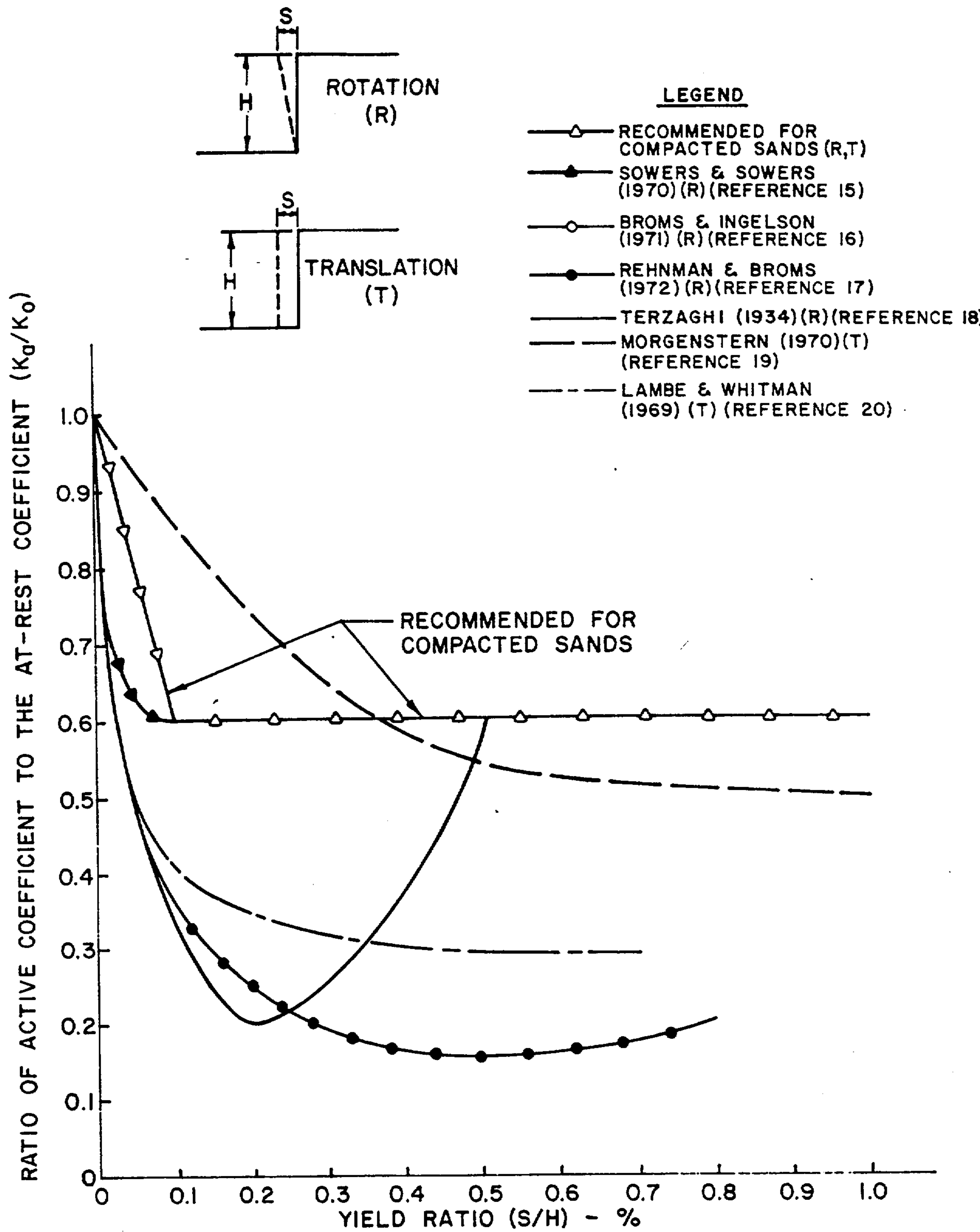
+ REFER TO FIGURE 8 FOR DETERMINATION OF σ_c.

FIGURE 1
STATIC AND DYNAMIC LATERAL EARTH PRESSURES

ATTACHMENT A CALC 0596.02-G(B)-05-2 p. A1/4

01016

ATT A CALC 05996.02-G(B)-05-2 p A2



~~01017~~

FIGURE 6a
ACTIVE EARTH PRESSURE
COEFFICIENTS vs WALL MOVEMENT

Att A Calc 05996.02-G(B)-05-2 p A3

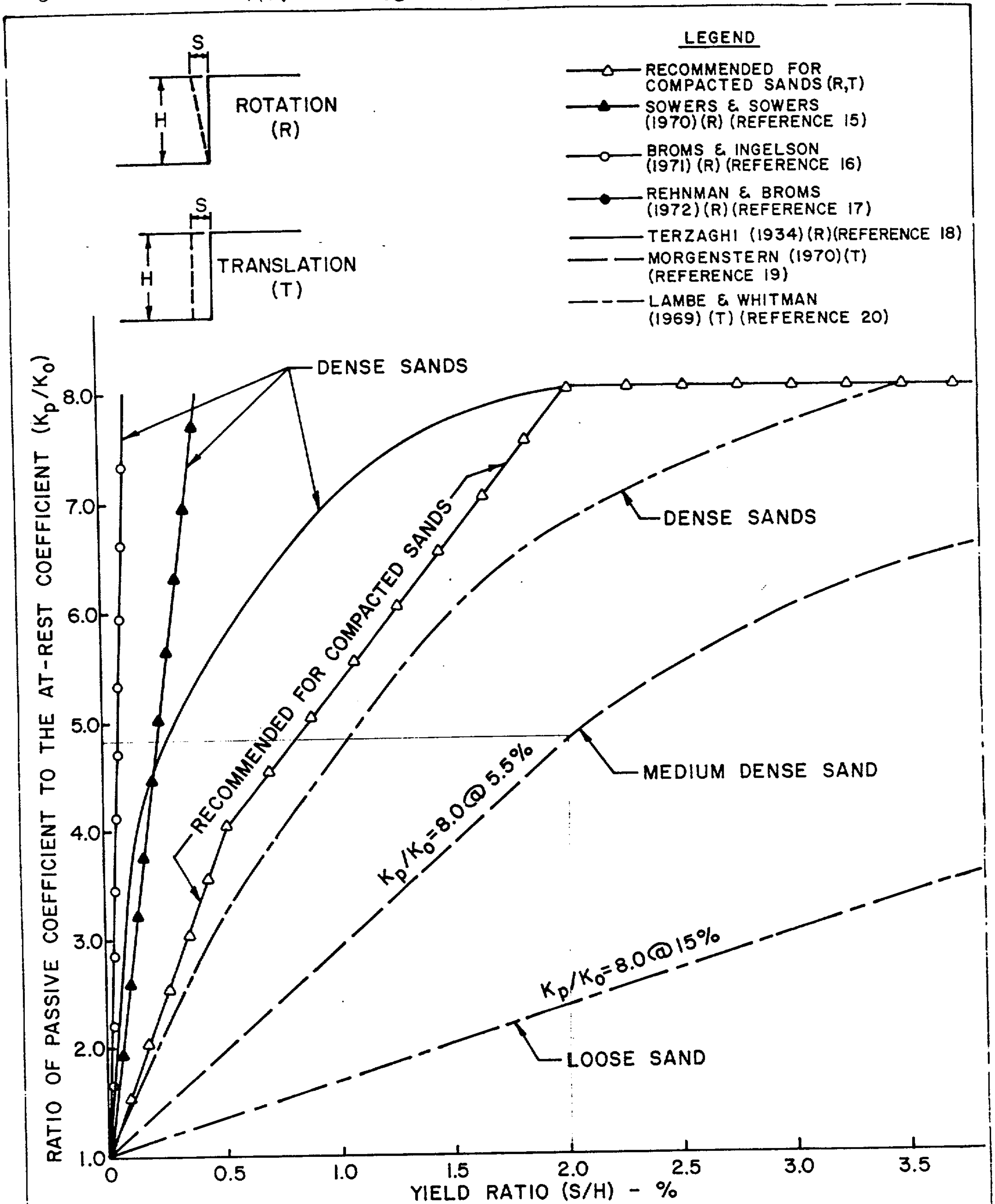
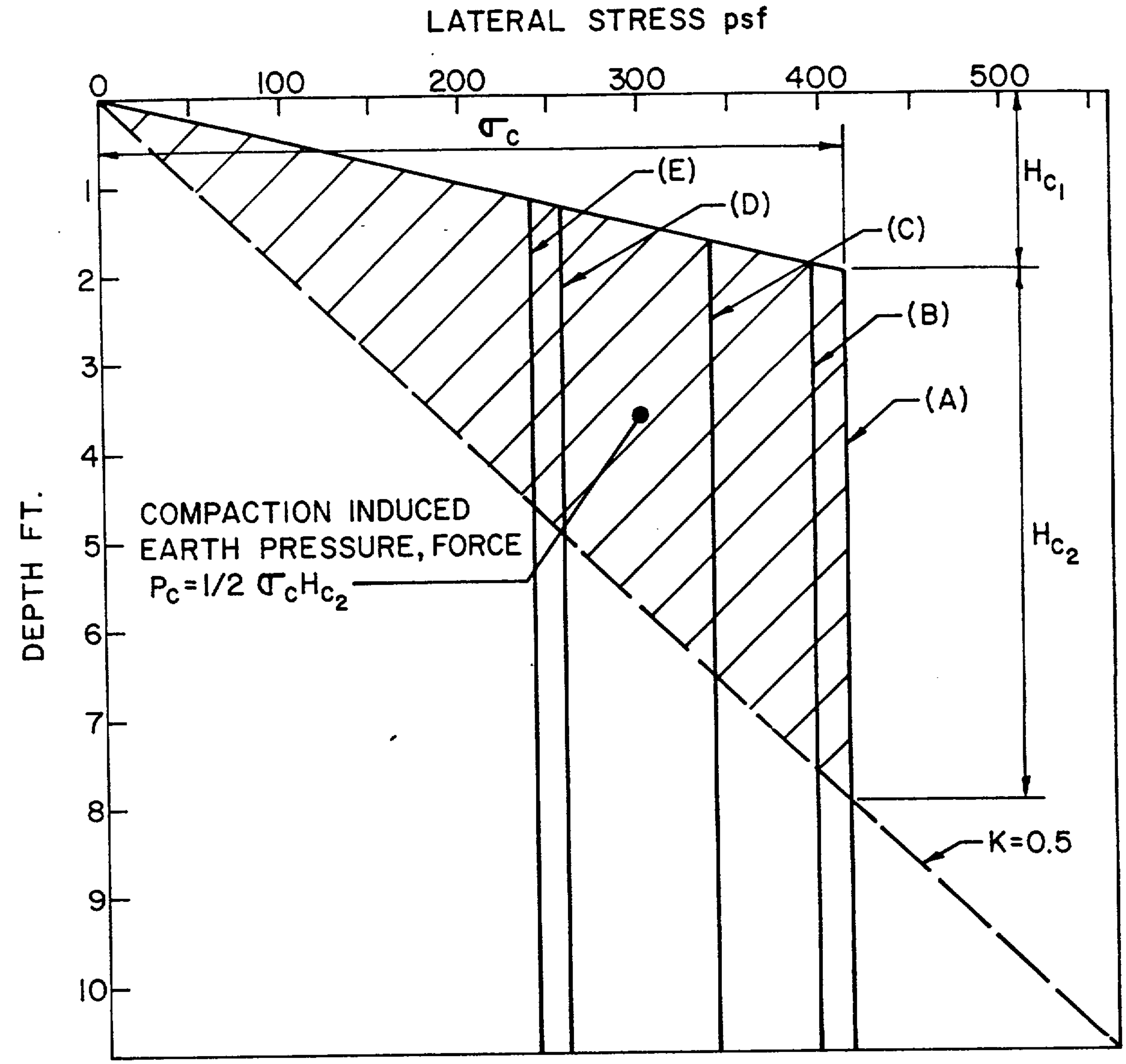


FIGURE 6b
PASSIVE EARTH PRESSURE
COEFFICIENTS vs WALL MOVEMENT

- ~~01018~~

ATT A CALC 05996.02-G(B)-05-2 p A4



NOTES:

1) ENVELOPES OF LATERAL PRESSURE FOR:
   - (A) 11.2 t SMOOTH WHEEL ROLLER
   - (B) 3.6 t VIBRATORY ROLLER
   - (C) 1.5 t VIBRATORY ROLLER
   - (D) 880 lb. VIBRATORY PLATE COMPACTOR
   - (E) 265 lb. VIBRATORY PLATE COMPACTOR
2) 1t = 2000 lbs.
3) ASSUME UNIT WEIGHT OF SOIL = 112 pcf

REFERENCE:

BROMS, B. (1971) (REFERENCE 7)

FIGURE 8
COMPACTION INDUCED
LATERAL STRESSES

~~01019~~

Subject:

BORROW, GRANULAR BORROW, AND GRANULAR BACKFILL BORROW

Section: **220**

220.1 DESCRIPTION

220.1.1 Obtain material, excavate, haul, place, and compact, as shown.

220.1.2 Related Work

Section 211—Excavate for Structure
Section 221—Embankment

220.2 MATERIALS

AASHTO M-145 A-1a—A-4

220.2.1 Borrow—Conform to the material standard.

AASHTO M-145 A-1, A-2-4, or A-3

220.2.2 Granular Borrow—Conform to suitability of source requirements. The suitability of source will be determined using the material standard and the design CBR or R value. These parameters will not be used for project control testing.

220.2.3 Granular Backfill Borrow

AASHTO M-145 A-1

220.2.3.1 Conform to the material standard modified to 2-inch maximum size and well graded.

220.2.3.2 Free draining granular backfill material—Natural aggregate or crushed slag to meet the following gradation:

Table 220-1

Sieve Size	Percent Passing
1 - 1/2 inch	100
1 inch	95 - 100
1/2 inch	25 - 60
No. 4	0 - 10

~~01020~~

AASHTO T-27
AASHTO T-11

301.2.1.1 Aggregate Job-Mix Gradation

Table 301-1
Gradation Limits—Single Value Job-Mix Formula

Sieve Size	Percent Passing of Total Aggregate (Dry Weight)		
	1 1/2 inch	1 inch	3/4 inch
1 1/2 inch	100	--	--
1 inch	--	100	--
3/4 inch	81 - 91	--	100
1/2 inch	67 - 77	79 - 91	--
3/8 inch	--	--	78 - 92
No. 4	43 - 53	49 - 61	55 - 67
No. 16	23 - 29	27 - 35	28 - 38
No. 200	6 - 10	7 - 11	7 - 11

301.3 CONSTRUCTION REQUIREMENTS

301.3.1 Job-Mix Gradation

301.3.1.1 Submit a written job-mix gradation for approval before production, including single values for each sieve size based on the dry weight of the aggregate.

301.3.1.2 Dry weight values shall fall within the bands shown in Table 301-1.

301.3.1.3 Procedures for Changing the Job-Mix Gradation

- All changes must fall within bands of Table 301-1.
- Changes shall be submitted in writing before a day's production starts.
- Changes are subject to approval.
- For each construction season, retroactive changes are allowed only for the first day's production.

~~C1021~~

AASHTO T-27
AASHTO T-11

304.2.2.1 Aggregate Job-Mix Gradation

Table 304-1		
	Job-Mix Gradation Band	Allowable Variation From Job-Mix Gradation
Sieve Size	Percent Passing	Percent
1 1/2 inch	100	-
1 inch	85 - 100	-
3/4 inch	50 - 100	±8
3/8 inch	30 - 75	±8
No. 4	25 - 60	±8
No. 40	8 - 25	±4
No. 200	0 - 9	±3

304.2.3 Water—Refer to Subsection 408.2.4

304.2.4 Admixtures

AASHTO M-154

304.2.4.1 Air-entraining agents.

AASHTO M-194 Type A

304.2.4.2 Water-reducing admixtures—except:

- The relative durability factor shall be at least 90.
- The chlorides content (as Cl^-) shall not exceed 1 percent by weight of the admixture.

304.2.4.3 Do not use calcium chloride.

AASHTO M-148 Type II

304.2.5 Curing Compound—As specified for white, pigmented material with wax base.

304.2.6 Bond Breaker—Use curing compound per Subsection 304.2.5.

~~01022~~

AASHTO T-30

402.2.2.3 **Aggregate Gradation**

Table 402-1
Gradation Limits for Single-Value Job-Mix Formula

Sieve Size	Percent of Total Aggregate (dry weight)			
	1-inch (1)	3/4-inch (2) (non-rutting)	3/4-inch (3)	1/2-inch (4)
1 inch	100	------	-----	-----
3/4 inch	-----	100	100	-----
1/2 inch	75-91	74-99	-----	100
3/8 inch	-----	69-91	75-91	-----
No. 4	47-61	49-65	46-62	60-80
No. 8	-----	33-47	-----	-----
No. 16	23-33	21-35	22-34	28-42
No. 50	12-22	6-18	11-23	11-23
No. 200	5-9	2-6	5-9	5-9

402.2.3 Hydrated Lime—Refer to Section 711—Hydrated Lime.

402.3 CONSTRUCTION REQUIREMENTS

402.3.1 Stockpiles

402.3.1.1 Separate the aggregate into two or more sizes and store separately. One stockpile shall contain a minimum of 80 percent passing the No. 4 sieve. The other shall contain a minimum of 80 percent retained on the No. 4 sieve. If a natural fine material is to be used, separate it into another stockpile, and protect it from moisture.

402.3.1.2 Prevent all segregation, degradation, or combining of materials of different gradings when moving the aggregate to or from stockpiles. **Re-screen or waste** all segregated or degraded material.

402.3.1.3 Do not build conical stockpiles by free-fall of aggregate from a chute or belt conveyor. Crush and stockpile at least 10,000 tons or 25 percent of the estimated quantity (whichever is less) before paving.

~~01023~~

505.2.2 Coarse Aggregate

AASHTO M-80

505.2.2.1 As specified and as modified, using one of the gradations per Table 505-1.

Table 505-1

Percent Passing (by weight)

Aggregate Size	2-1/2"	2"	1-1/2"	1"	3/4"	1/2"	3/8"	No. 4
2" to No. 4	100	95-100		35-70		10-30		0-5
1-1/2" to No. 4		100	95-100		35-70		10-30	0-5
1" to No. 4			100	95-100		25-60		0-10
3/4" to No. 4				100	90-100		20-55	0-10

AASHTO M-92

505.2.2.2 Use sieve screens with square openings as specified.

505.2.2.3 Deleterious Substances: Do not exceed percentages per Table 505-2.

Table 505-2

	Percent (by weight)
Soft Fragments	2.0
Coal and Lignite	0.3
Clay Lumps	0.3
Other Substances	2.0

~~01024~~

AASHTO M-80

505.2.2.4 Use the requirements for soundness, percentage of wear, and potential reactivity, as specified, to determine the suitability of coarse aggregate sources, but not for routine control testing.

505.2.3 Fine Aggregate

AASHTO M-6

505.2.3.1 As specified using one of the gradations per Table 505-3.

Table 505-3

Sieve Size	Percent Passing (by weight)
3/8 inch	100
No. 4	95 to 100
No. 16	45 to 80
No. 50	10 to 30
No. 100	2 to 10

505.2.3.2 Deleterious Substances: Do not exceed percentages per Table 505-4.

Table 505-4

	Percent (by weight)
Coal and Lignite	0.3
Clay Lumps	0.5
Other Substances	2.0

ATT B CALC 05996.02-G(B)-05-2

P. B6

~~01025~~

ATT B CALC 05996.02-G(B)-05-2 p 87

Subject:	Section:
UNDERDRAIN	**916**

916.1 DESCRIPTION

916.1.1 Furnish and place pipe underdrains of the class, type, and size shown.

916.1.2 Related Work

Section 901—Pipe, Pipe-Arch; Structural Plate Pipe and Plate Pipe-Arch Culvert

916.2 MATERIALS

AASHTO M-252

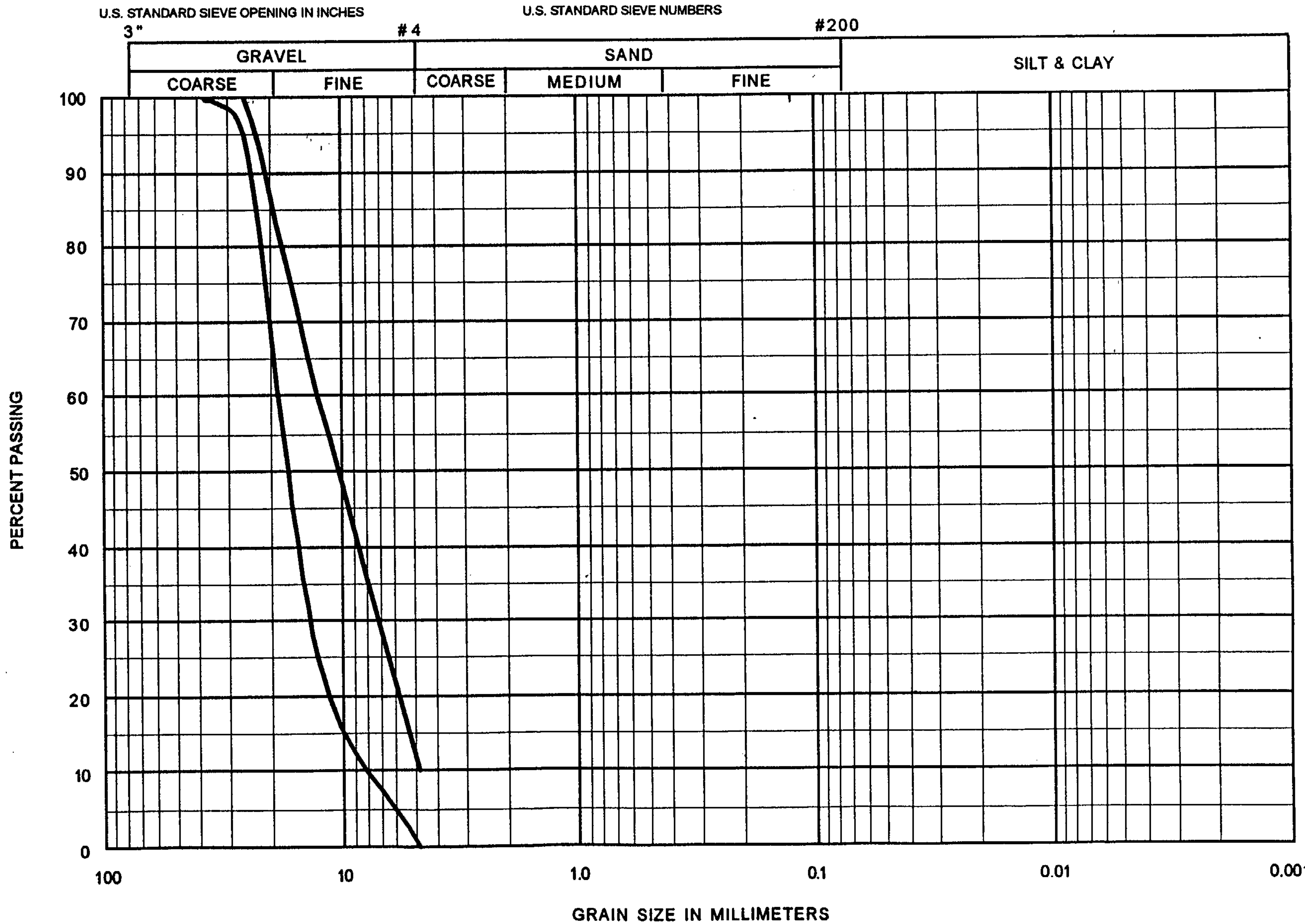
916.2.1 Pipe—Refer to Subsection 901.2

916.2.2 Underdrain Granular Backfill—Use the following gradations:

Sieve Size	Type A Percent Passing	Type B Percent Passing
2 1/2 inch	100	
1 1/2 inch	80 - 100	100
1/2 inch	55 - 75	50 - 80
No. 4	30 - 60	30 - 65
No. 40	10 - 25	10 - 30
No. 200	0 - 3	0 - 3

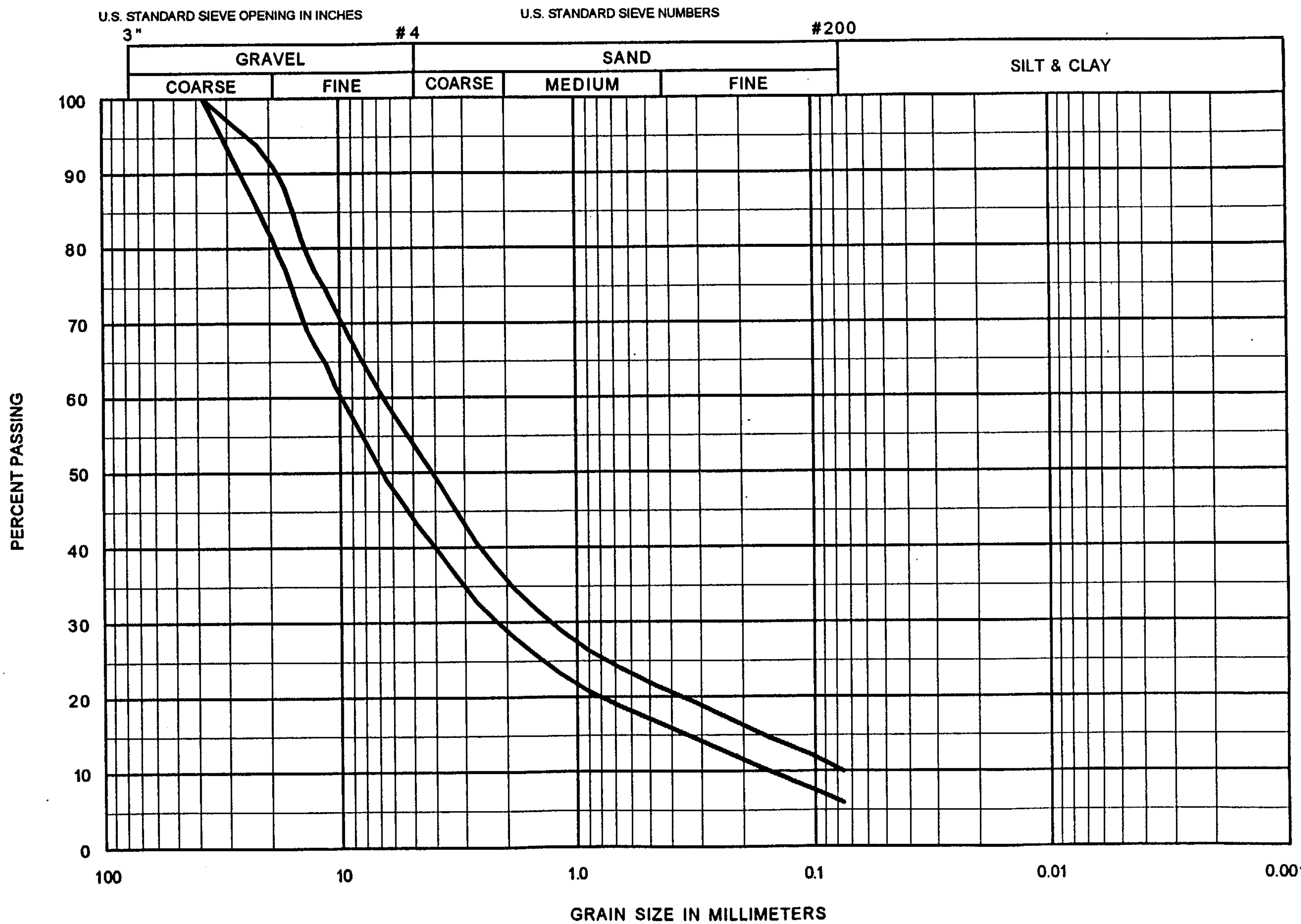
~~01026~~

ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B8



STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS GRANULAR BACKFILL BORROW UTAH DOT TABLE 220-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

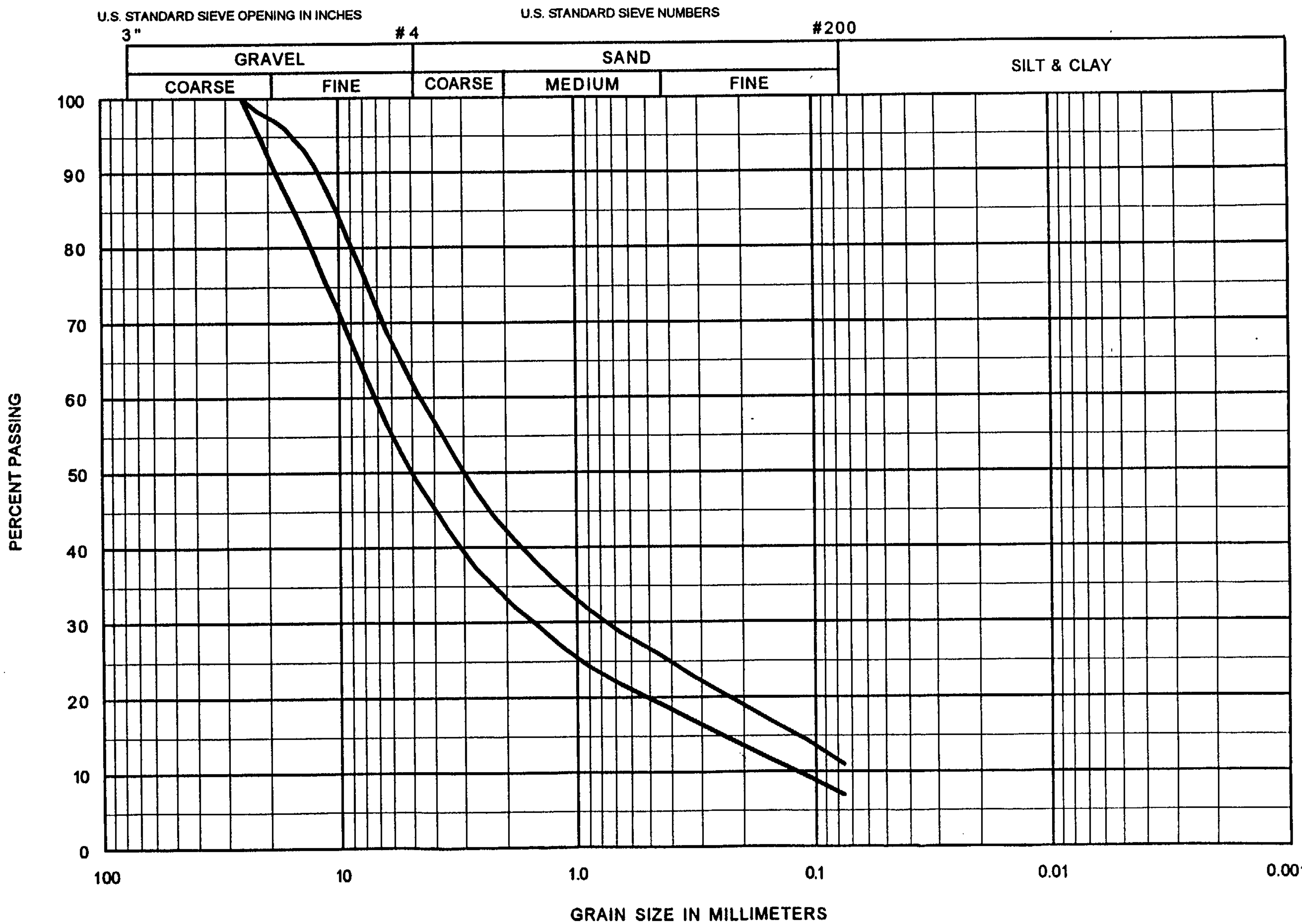
ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B9



01028

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - SINGLE VALUE JOB-MIX FORMULA 1 1/2 inch AGGREGATE - UNTREATED BASE COURSE UTAH DOT TABLE 301-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

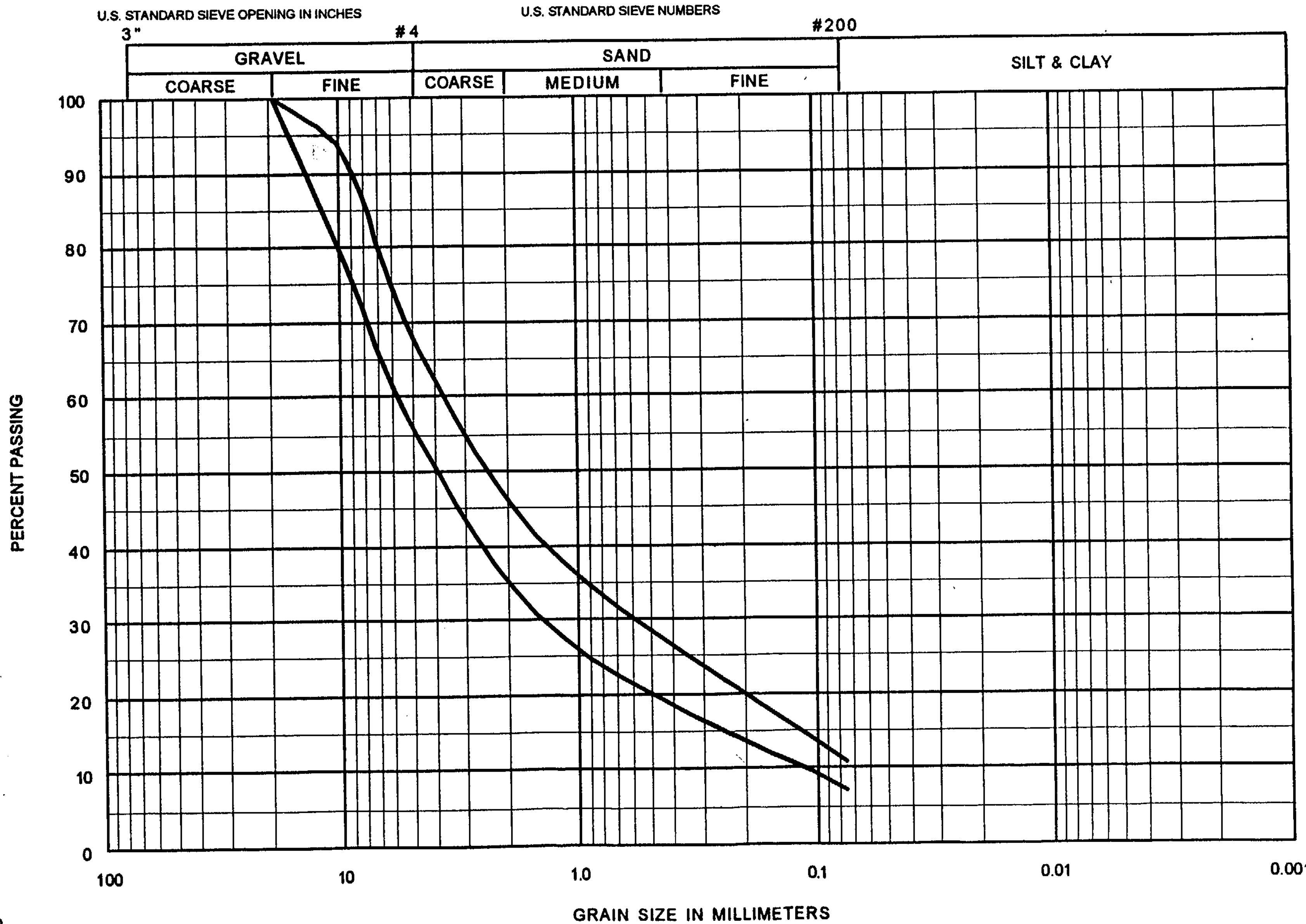
ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B10



~~01029~~

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - SINGLE VALUE JOB-MIX FORMULA 1 inch AGGREGATE - UNTREATED BASE COURSE UTAH DOT TABLE 301-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

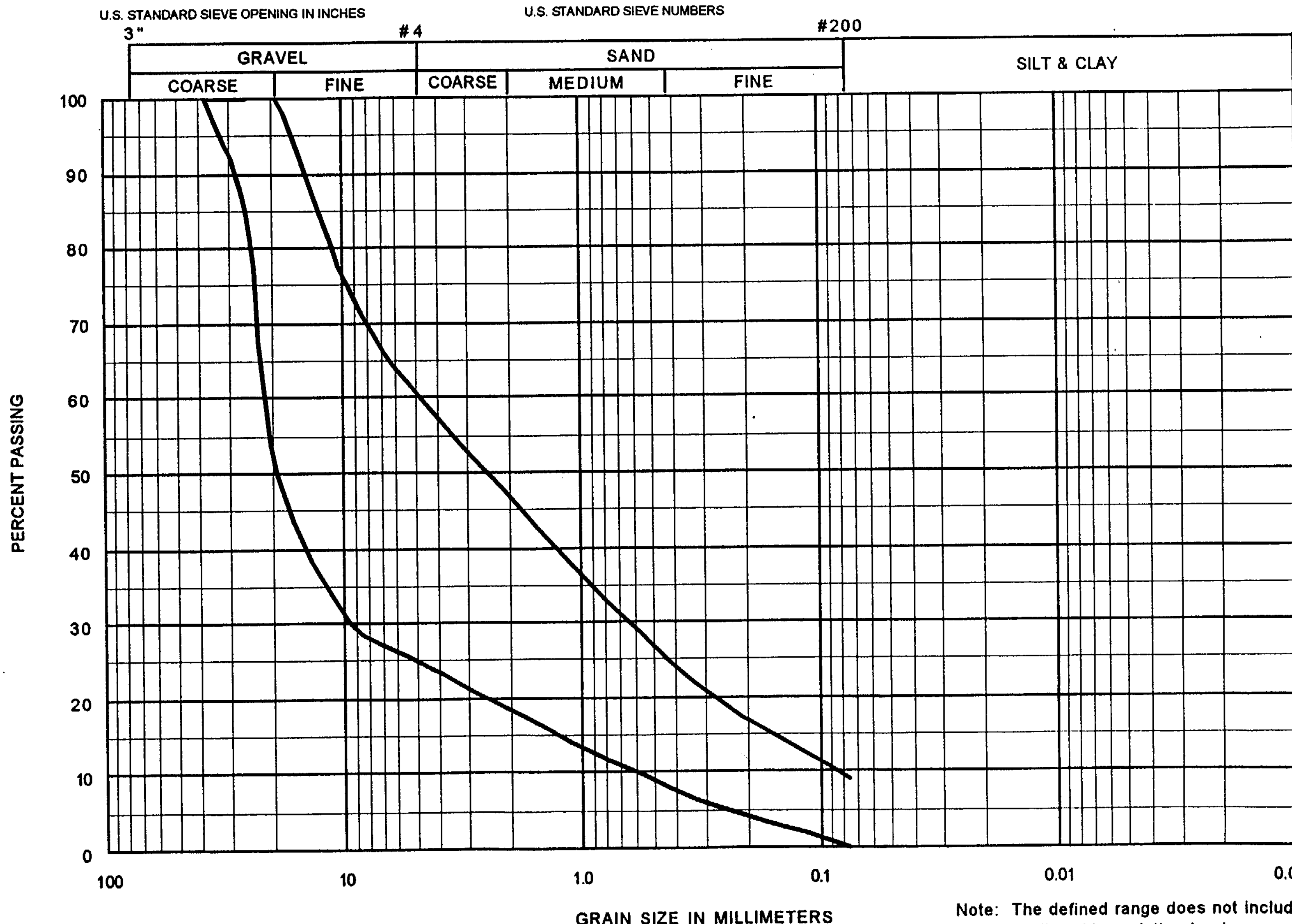
ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B11



01030

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - SINGLE VALUE JOB-MIX FORMULA 3/4 inch AGGREGATE - UNTREATED BASE COURSE UTAH DOT TABLE 301-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B12

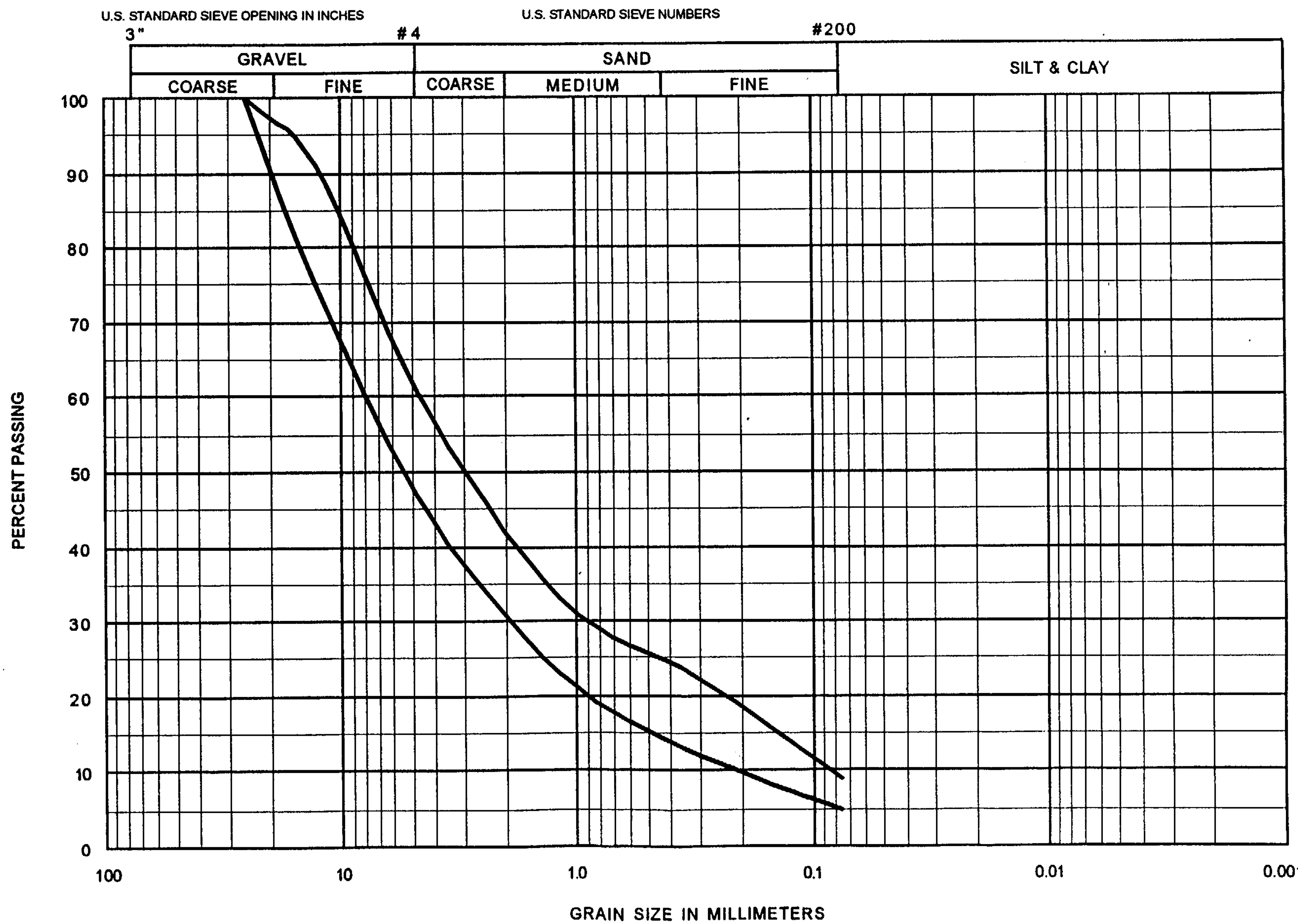


Note: The defined range does not include the allowable variation in sizes.

~~01031~~

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	AGGREGATE JOB-MIX GRADATION LIMITS LEAN CONCRETE BASE COURSE UTAH DOT TABLE 304-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B13



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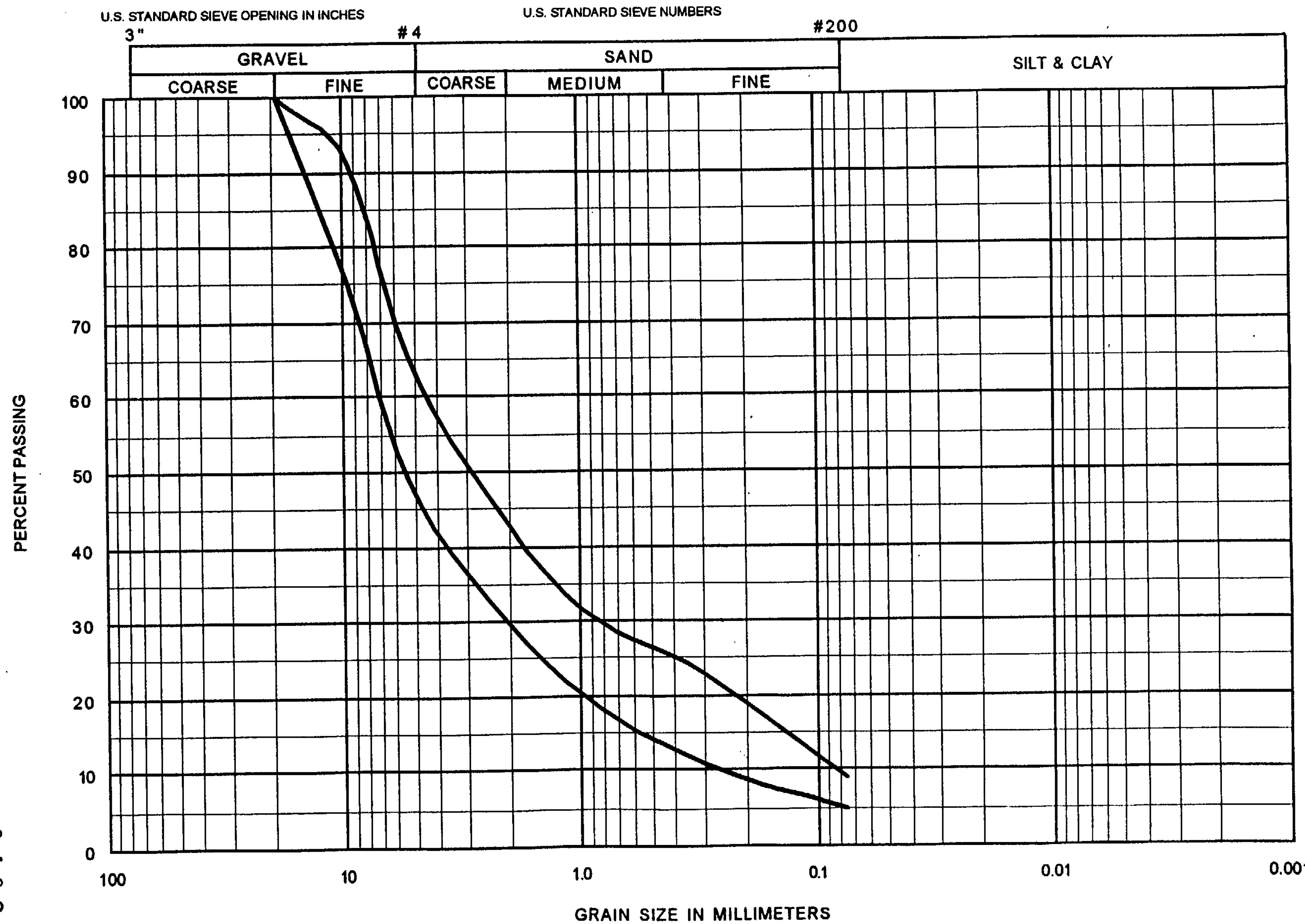
~~01032~~

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - SINGLE VALUE JOB-MIX FORMULA 1" Aggregate ASPHALT CONCRETE PAVEMENT UTAH DOT TABLE 402-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

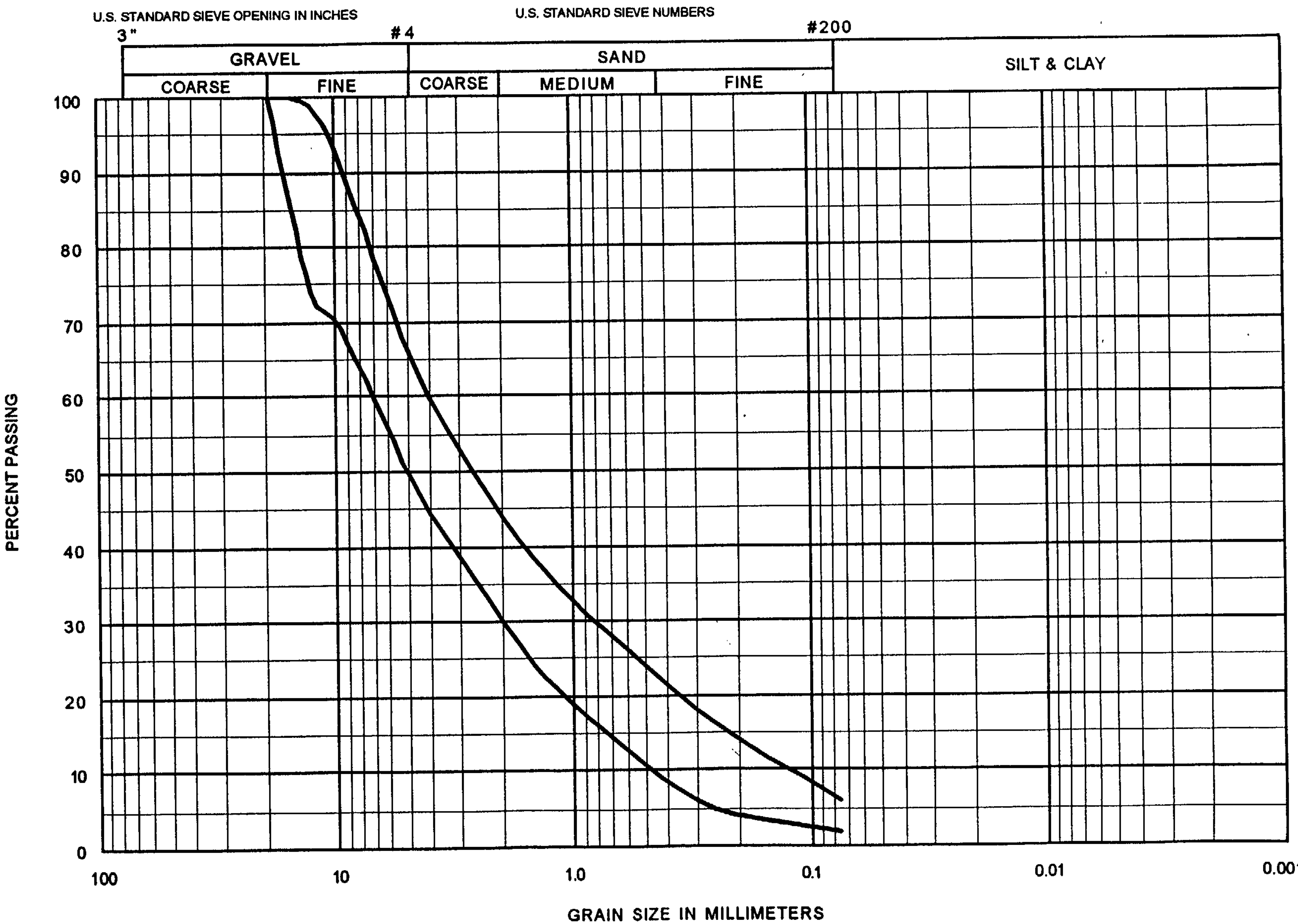
ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B14



01033

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - SINGLE VALUE JOB-MIX FORMULA 3/4" Aggregate ASPHALT CONCRETE PAVEMENT UTAH DOT TABLE 402-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

ATTACHMENT B CALC 05996.02-G(B)-05-2 p B15

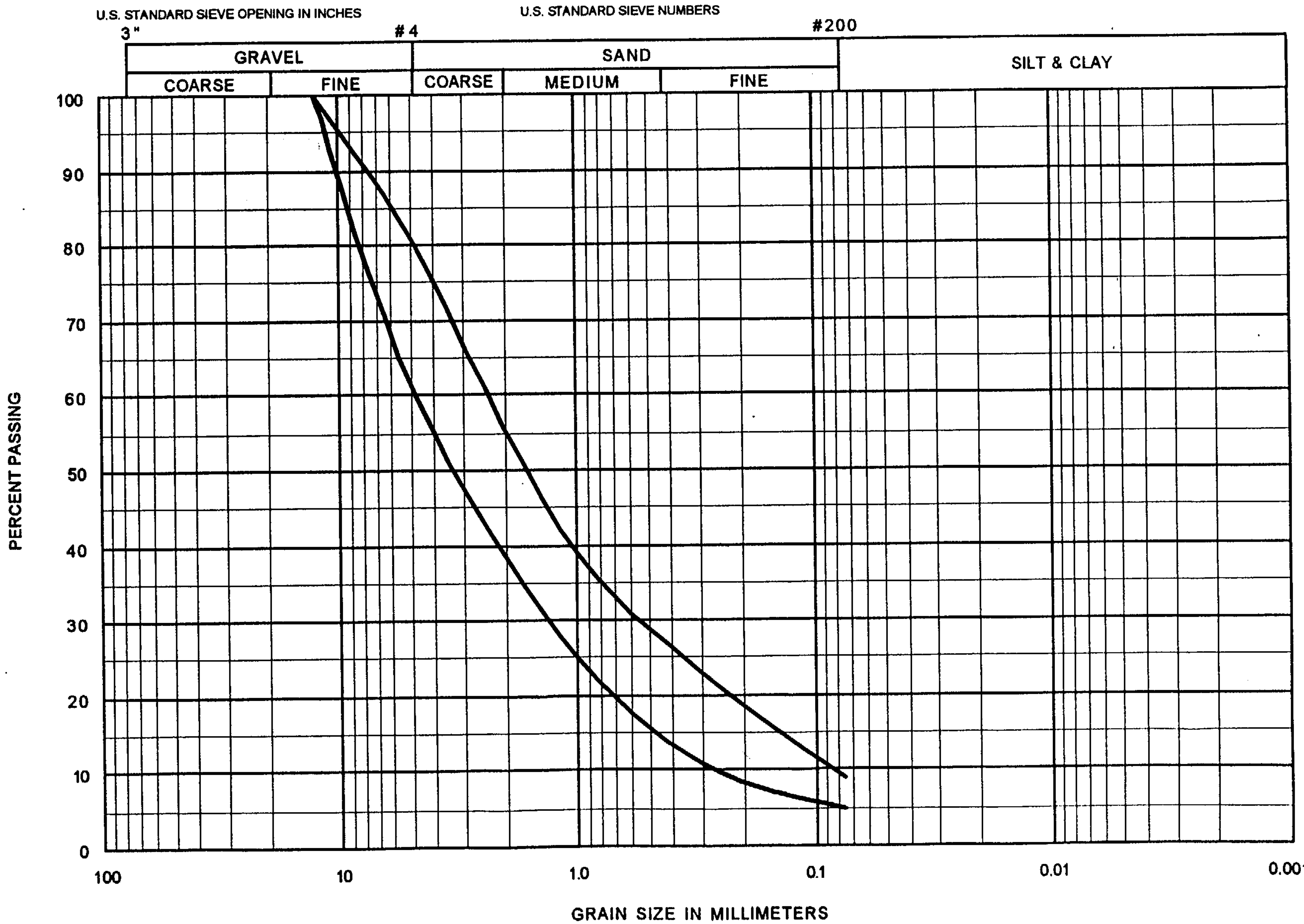


01034

STONE & WEBSTER ENGINEERING CORP
BOSTON, MASSACHUSETTS

GRADATION LIMITS - SINGLE VALUE JOB-MIX FORMULA
3/4" (non-rutting) Aggregate ASPHALT CONCRETE PAVEMENT
UTAH DOT TABLE 402-1

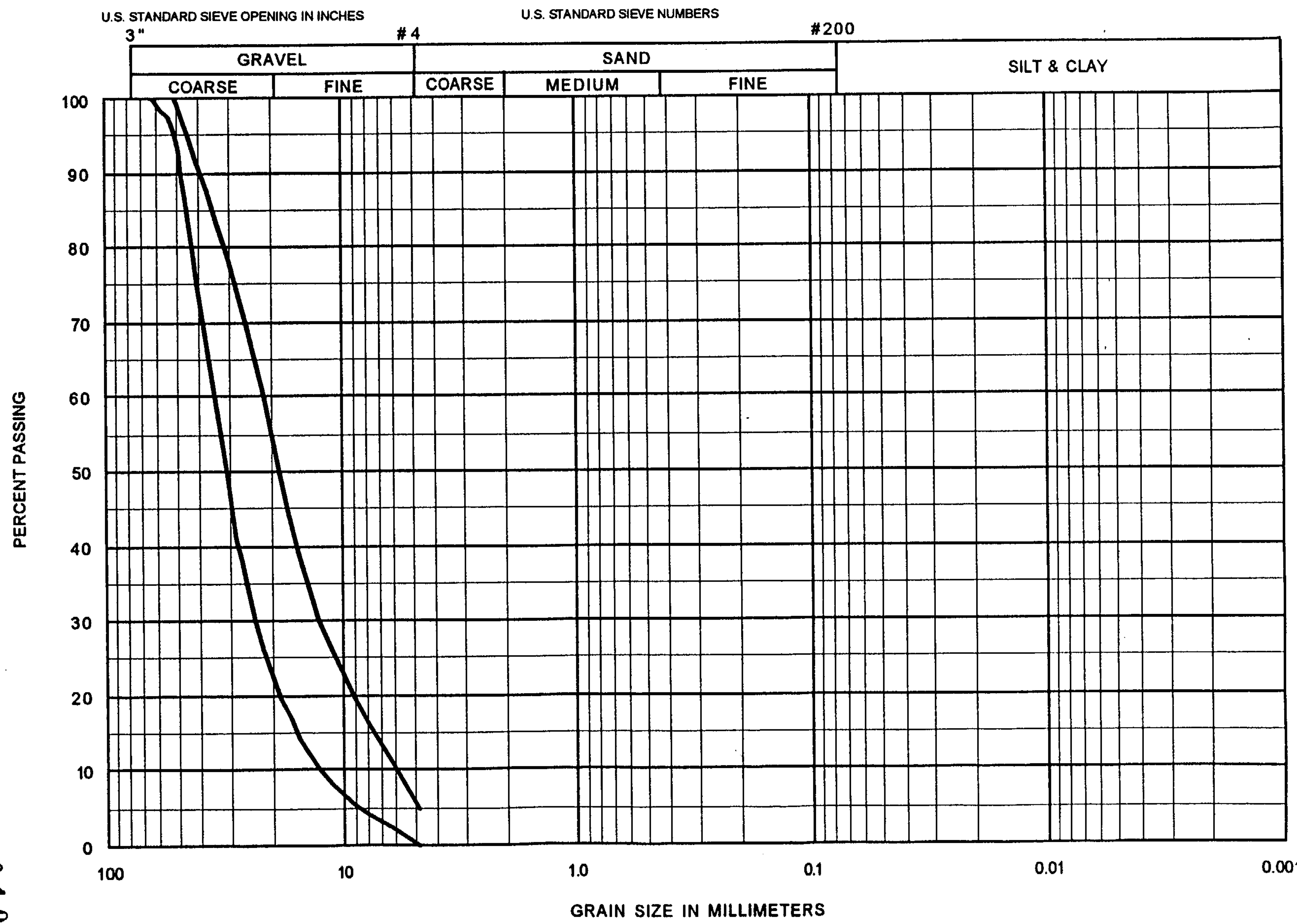
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE, LLC
JO 05996.01 MARCH 1997



~~01035~~

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - SINGLE VALUE JOB-MIX FORMULA 1/2" Aggregate ASPHALT CONCRETE PAVEMENT UTAH DOT TABLE 402-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

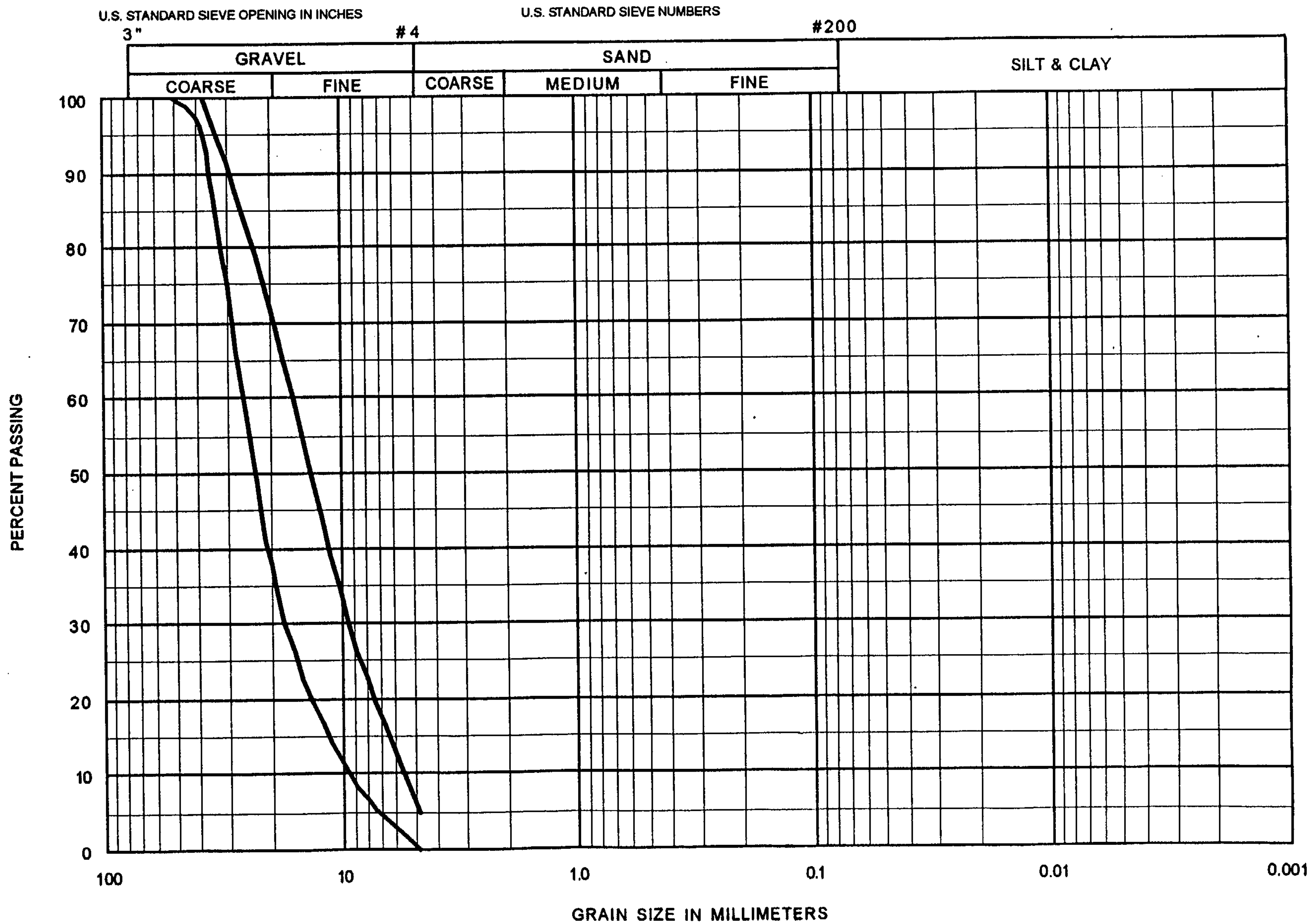
ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B17



01036

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - 2" to #4 COARSE AGGREGATE PORTLAND CEMENT CONCRETE UTAH DOT TABLE 505-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B18

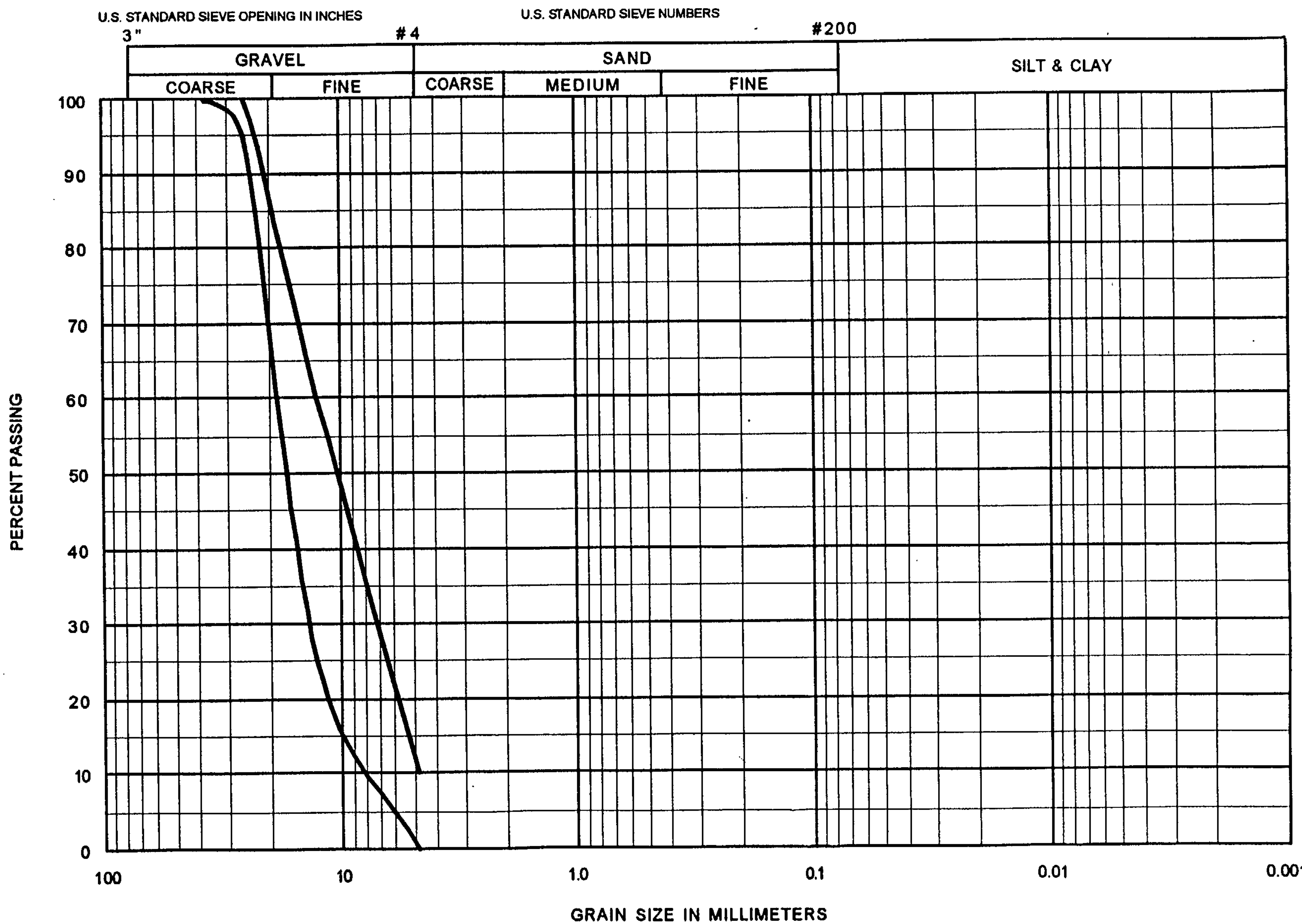


STONE & WEBSTER ENGINEERING CORP
BOSTON, MASSACHUSETTS

GRADATION LIMITS - 1 1/2" to #4 COARSE AGGREGATE
PORTLAND CEMENT CONCRETE
UTAH DOT TABLE 505-1

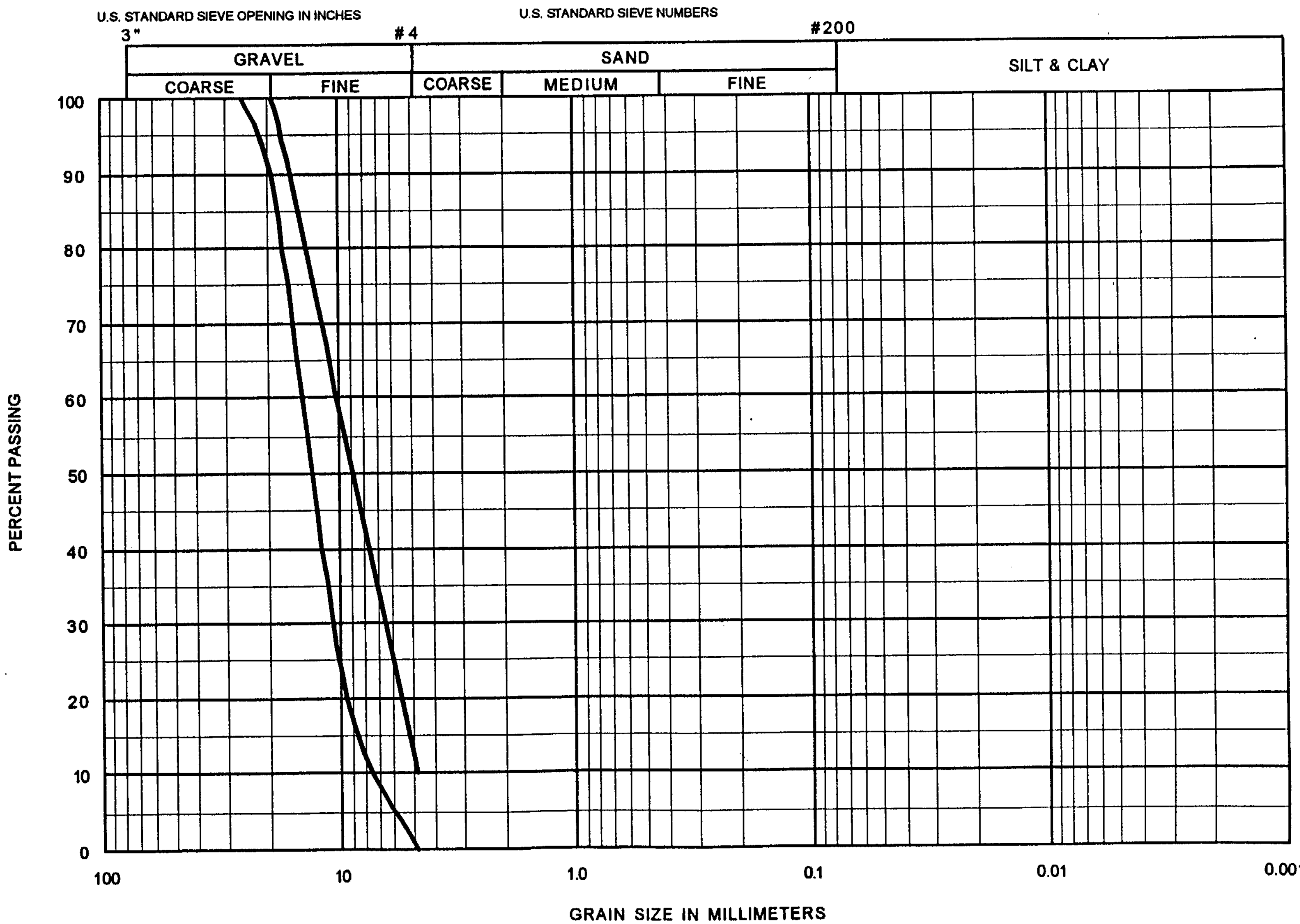
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE, LLC
JO 05996.01 MARCH 1997

ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B19



01038

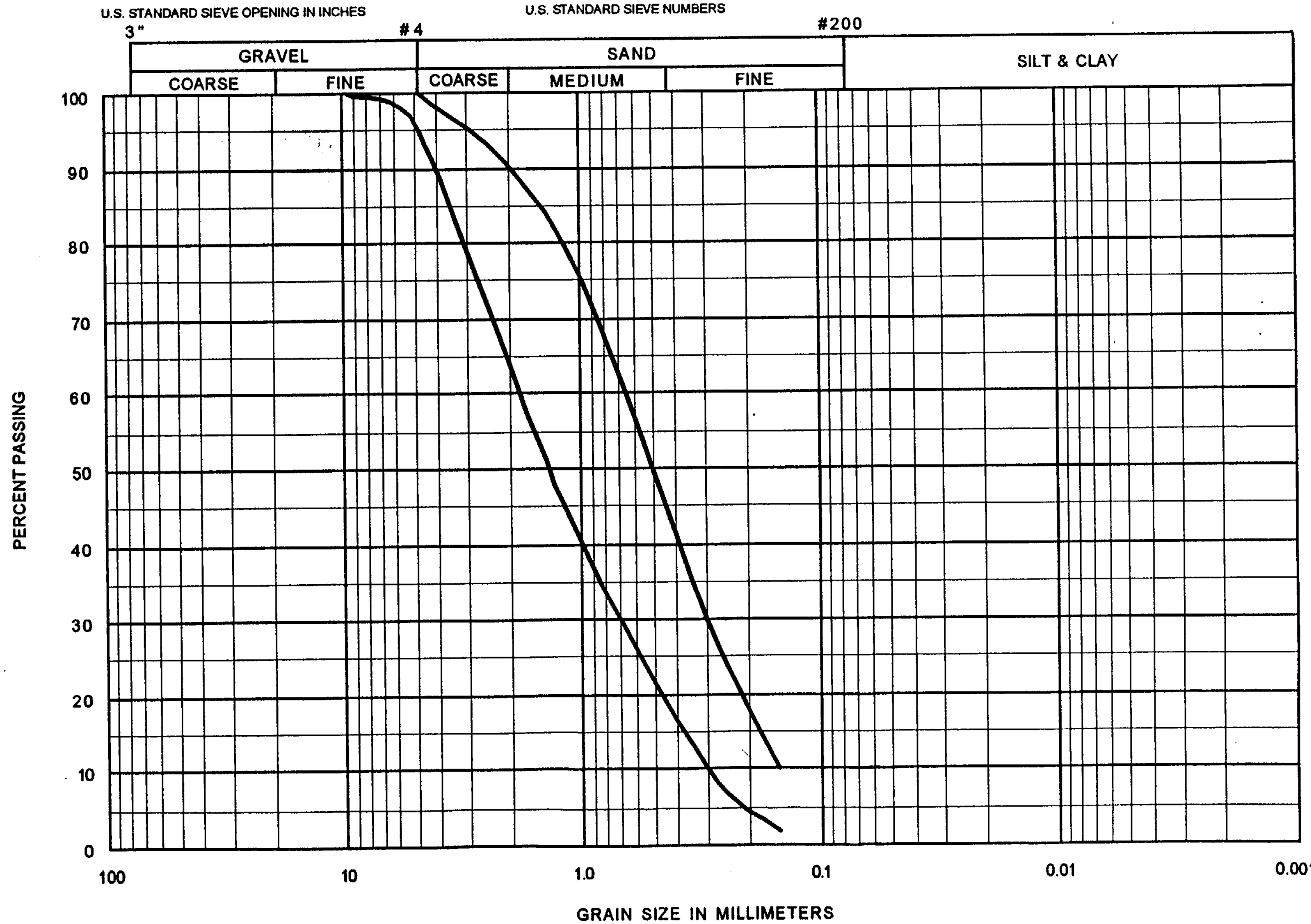
STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - 1" to #4 COARSE AGGREGATE PORTLAND CEMENT CONCRETE UTAH DOT TABLE 505-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997



01039

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - 3/4" to #4 COARSE AGGREGATE PORTLAND CEMENT CONCRETE UTAH DOT TABLE 505-1	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B21



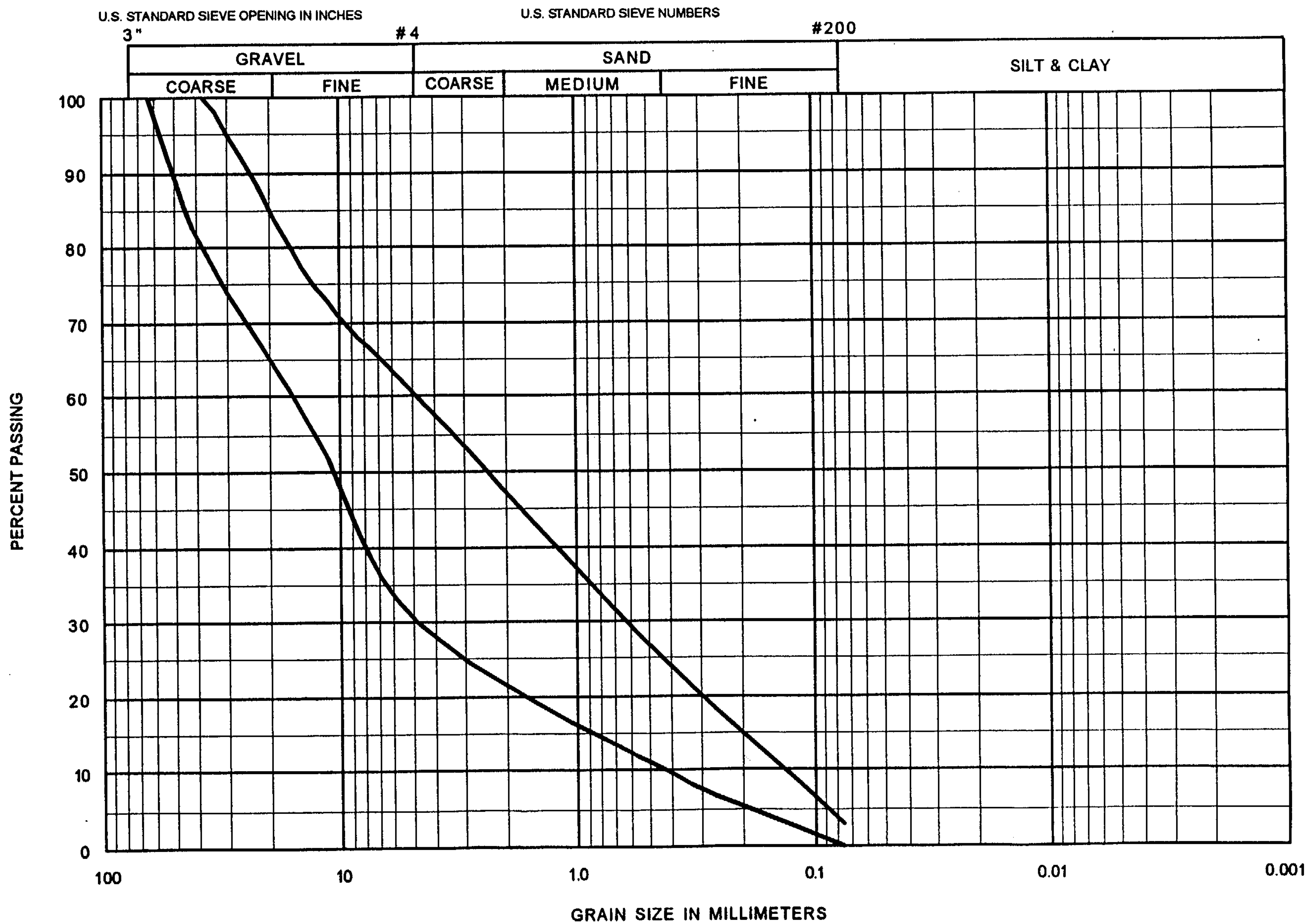
01040

STONE & WEBSTER ENGINEERING CORP
BOSTON, MASSACHUSETTS

GRADATION LIMITS - FINE AGGREGATE
PORTLAND CEMENT CONCRETE
UTAH DOT TABLE 505-3

PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE, LLC
JO 05996.01 MARCH 1997

ATTACHMENT B CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p B22

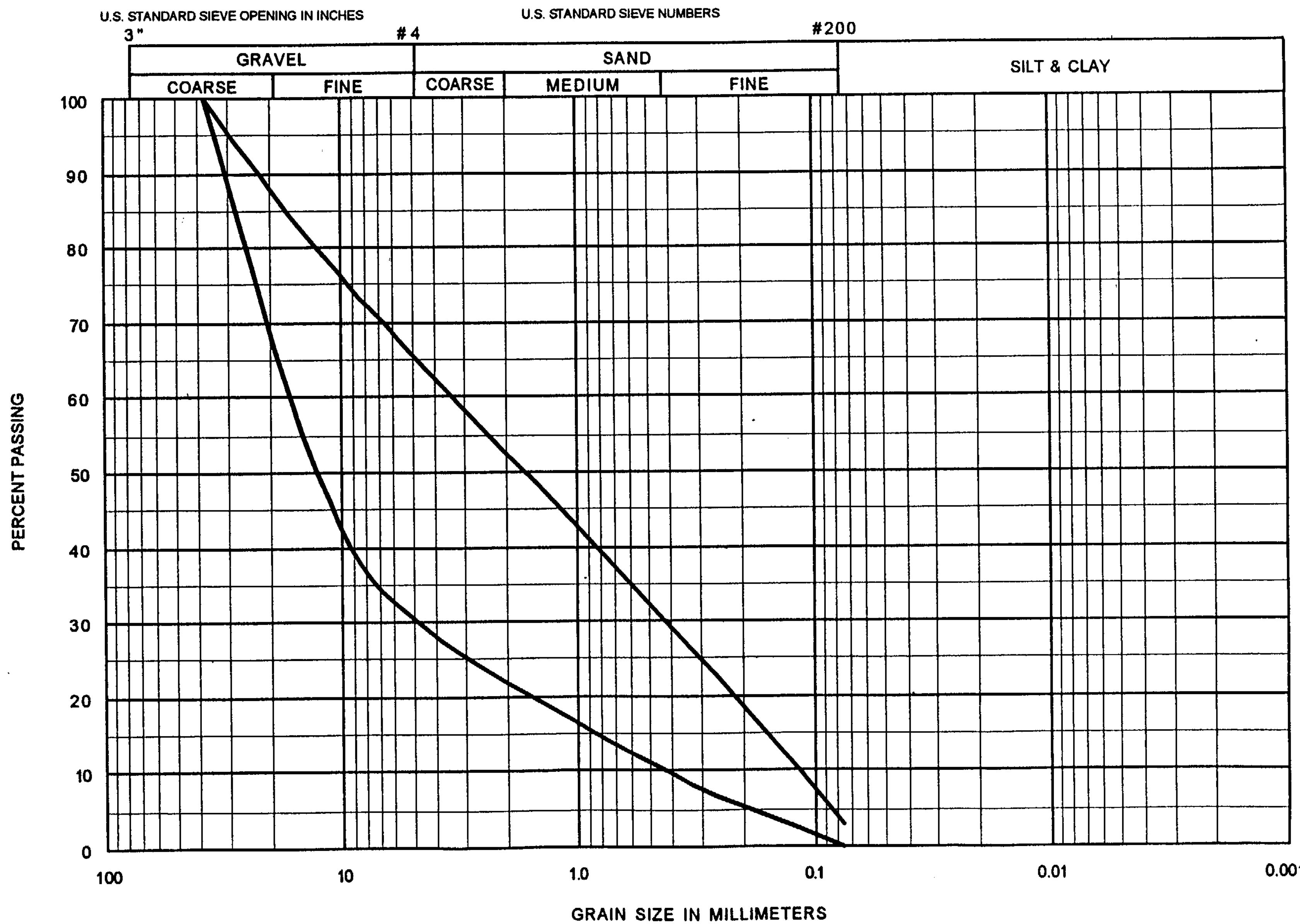


01041

STONE & WEBSTER ENGINEERING CORP
BOSTON, MASSACHUSETTS

GRADATION LIMITS - TYPE A MATERIAL
UNDERDRAIN GRANULAR BACKFILL
UTAH DOT SECTION 916.2.2

PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE, LLC
JO 05996.01 MARCH 1997



~~01042~~

STONE & WEBSTER ENGINEERING CORP BOSTON, MASSACHUSETTS	GRADATION LIMITS - TYPE B MATERIAL UNDERDRAIN GRANULAR BACKFILL UTAH DOT SECTION 916.2.2	PRIVATE FUEL STORAGE FACILITY PRIVATE FUEL STORAGE, LLC JO 05996.01 MARCH 1997

Fax Cover Sheet

STONE & WEBSTER ENGINEERING CORPORATION

Denver Operations Center
7677 East Berry Avenue
Englewood, CO 80111-2137

DATE:	April 28, 1997	**TIME:**	2:00 pm
TO:	Paul Trudeau Stone & Webster	**PHONE:** **FAX:**	617-589-8473 617-589-2959
FROM:	Stan Macie Stone & Webster	**PHONE:** **FAX:**	303-741-7305 303-741-7806
RE:	Storage Cask Weights		

SWEC J.O. NO.: 05996.01

Cover sheet plus 3 **pages**

Message

Paul,

Please use the attached cask vendor's weight data as an attachment to your calculations. These pages are from their latest SARs.

Thank you,

Stan M.

Jb Bk G2-1/1

ATTACHMENT C CALC 05996.02-G(B)-05-2 p C1/5

~~01043~~

TABLE 3.2-1
TranStor™ SYSTEM WEIGHTS AND CENTERS OF GRAVITY

ITEM DESCRIPTION	WEIGHT (lbs)		CENTER OF GRAVITY (inches above bottom)	
	PWR	BWR	PWR	BWR
• Storage Cask Lid	1,235	1,235	N/A	N/A
• Basket Structural Lid	2,730	2,730	N/A	N/A
• Basket Shield Lid	7,470	7,470	N/A	N/A
• Transfer Cask Lid	400	400	N/A	N/A
• Basket (Empty, w/o Lids)	27,870	31,570	88.1	89.2
• Basket (Loaded w/Water and Shield Lid)	87,360	94,950	93.7	97.0
• Basket (Loaded, dry, w/Lids)	76,595	84,460	97.8	100.9
• Storage Cask (Empty, w/o Lid)	222,200	222,200	109.4	109.4
• Storage Cask & Basket (Empty, w/o Lids)	252,540	256,240	110.5	110.6
× • Storage Cask & Basket (Loaded, w/Lids)	297,055	309,130	113.9	113.9
• Transfer Cask (Empty w/o Lid)	126,230	126,230	90.6	90.6
• Transfer Cask with Basket (Empty, w/o Shield Lid)	154,695	158,390	92.4	92.8
• Transfer Cask with Basket (Loaded, w/ water and Lid)	211,870	222,550	98.0	98.2
• Transfer Cask with Basket (Loaded, dry, w/ Lids)	199,205	211,290	99.0	99.2

ATT C CALC 05996.02-G(B)-05-2 PC2

~~01044~~

Table 3.2.1

HI-STORM OVERPACK WEIGHT DATA†

	WEIGHT (lb)	
ITEM	COMPONENT	ASSEMBLY
➤ Overpack		267,664
• Overpack top lid	23,963	
➤ MPC-32		
• Without SNF		35,097
• Fully loaded with SNF		88,857
➤ Overpack with loaded MPC-32		356,521 *
➤ MPC-24		
• Without SNF		38,511
• Fully loaded with SNF		78,831
➤ Overpack with fully loaded MPC-24 (PWR)	348,321** →	346,495
➤ MPC-68		
• Without SNF		38,531
• Fully loaded with SNF		86,131
➤ Overpack with fully loaded MPC-68 (BWR)	355,575** →	353,796
➤ Overpack with minimum weight MPC without SNF		302,761
➤ MPC-GTCC		
• Without GTCC waste		26,000
• Fully loaded with SNF		86,000
➤ Overpack with fully loaded MPC-GTCC		353,664

* 356.5K USED IN BEARING CAPACITY CALC (G(B)-4, REV 3-5) & SETTLEMENT CALC (G(B)-3-2 & -3) BOUNDS WEIGHTS INDICATED IN TABLE 3.2.1 OF HI-STORM TSAR REV 9 JULY 99

** WEIGHTS " " " " " " " " " " PER TELCON W/ JL COOPER & J JOHNS 9-15-99

† All calculated weights are rounded up to the nearest pound.

P.D.

1

ATT C CALC 05996.02-G(B)-05-2 p C3

Table 3.2.3

CENTERS OF GRAVITY OF HI-STORM 100 CONFIGURATIONS

Component	Height of CG Above Datum, inches
Overpack empty	116.3
HI-TRAC with Pool Lid empty	90.2
HI-TRAC with Transfer Lid empty	88.0
MPC-32 with fuel in overpack	118.0
MPC-24 with fuel in overpack	118.0
MPC-68 with fuel in overpack	118.0
HI-TRAC w/ Pool Lid and MPC-32 w/ fuel	93.8
HI-TRAC w/ Pool Lid and MPC-24 w/ fuel	93.7
HI-TRAC w/ Pool Lid and MPC-68 w/ fuel	93.8
HI-TRAC w/ Transfer Lid & MPC-32 w/ fuel	92.2
HI-TRAC w/ Transfer Lid & MPC-24 w/ fuel	91.6
HI-TRAC w/ Transfer Lid & MPC-68 w/ fuel	92.3

The datum used for calculations involving the overpack is the bottom of the overpack baseplate. The datum used for calculations involving the HI-TRAC is the bottom of the pool lid or transfer lid.

ATT C CALC 05996.02-G(8)-05-2 P C4

CALCULATION IDENTIFICATION NUMBER				PAGE C5
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 05-~~1~~2	OPTIONAL TASK CODE	

HOLTEC "HI-STORM Storage Overpack Dimensions

Rev 0 of this calculation included a copy of Holtec Drawing No. 1495, Rev 1, which was marked DRAFT. Per Telcon on 9-15-99, JLCooper & JJohns indicated that this drawing was superseded by Holtec Drawing 1495, Sheet 1 of 6, Rev 7, and Sheet 2 of 6, Rev 8.

The original drawing was used to identify the height and OD of the storage cask. These data are shown in PFSF SAR Table 4.2-2 as 231.25 in. and 132.5 in., respectively. These values did not change from those shown on the DRAFT version of Holtec Drawing No. 1495, Rev 1, that was included in the original version of this calculation.

See PFSF SAR Figure 4.2-3 for an elevation view of the storage cask.

Attachment D

PFSF Drawings Showing Plan & Elevation Views of Structures & J&R Eng'gs Corp Lift-N-Lock Crawler Transporter (pp D11 to D22)

Attachment D of Rev 0 of this calculation included on pp D1 through D10 a transmittal from SMMacie, dated 2-19-97, re: PFSF Foundation Loads & Plan Views of Bldgs and PRELIMINARY copies of SWEC Drawings EA-1, 3 to 7, and EM-1 to 3. The purpose of these pages was to identify the dimensions of the various structures. The dimensions of the various structures are shown on the following drawings:

Dwg No. 0599601-	Rev	Title
EA-1	C	Administration Building, Floor Plan
EA-3	C	Administration Building, Elevations
EA-4	C	OP & Maintenance Building, Floor Plan
EA-5	C	OP & Maintenance Building, Elevations
EA-6	D	Security & Health Physics Building, Floor Plan
EA-7	D	Security & Health Physics Building, Elevations
EA-8	D	Canister Transfer Building, Floor Plan
EA-9	D	Canister Transfer Building, Elevations

D2

ATT D CALC 05996.02-G(B)-05-2 NOTED FEB 19 1997 P. J. Trudeau

J & R ENGINEERING CO., INC.'S

LIFT-N-LOCK CRAWLER TRANSPORTER

MUKWONAGO, WI, USA

~~01058~~

ATT D CALC 05996.02-G(B)-05-2 D3

INTRODUCING...

THE LATEST TECHNOLOGY IN

VENTILATED STORAGE CASK TRANSPORTERS

~~01059~~

ATT D CALC 05996.02-G(B)-05-2

D4

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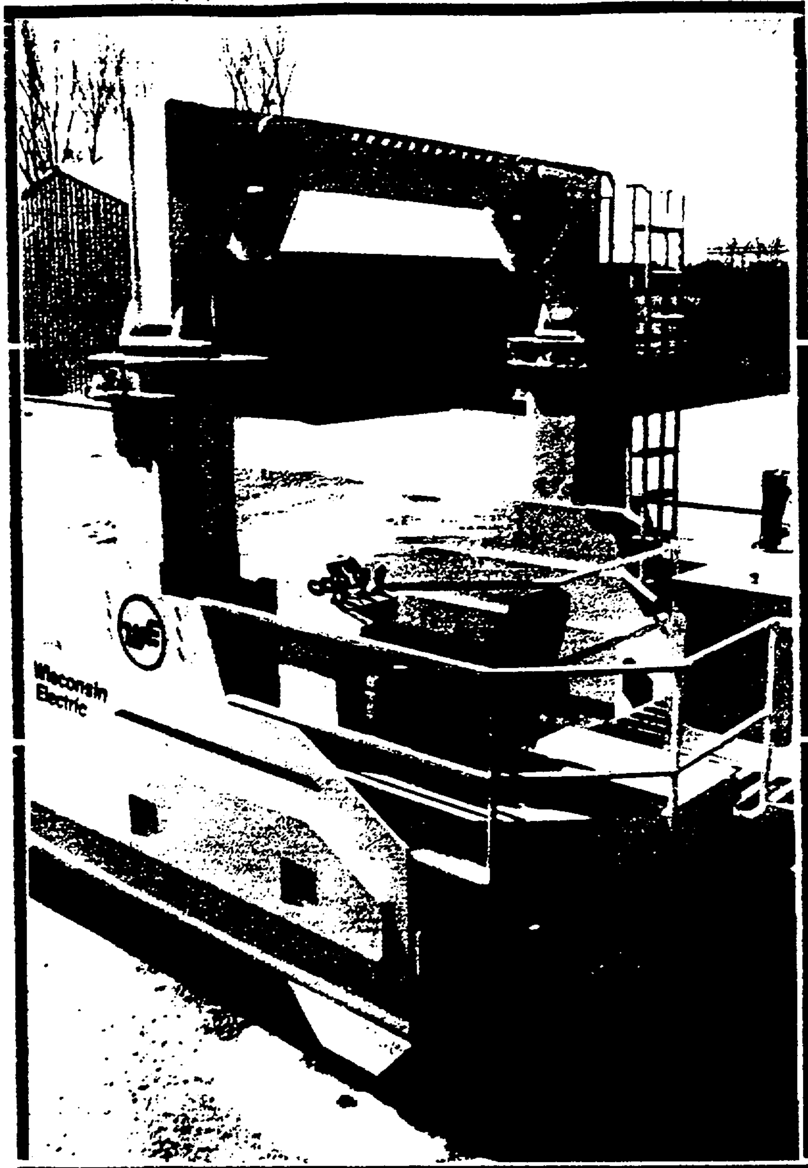
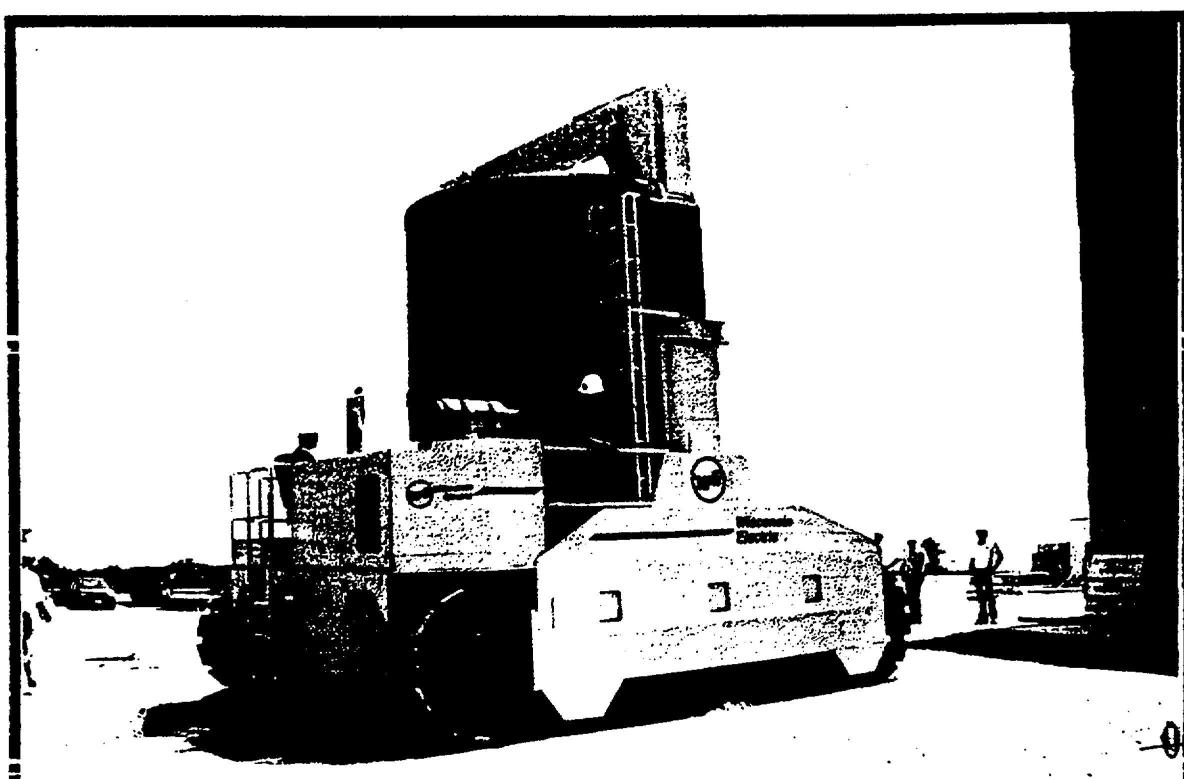


"TOP LIFT" CRAWLER TRANSPORTER



~~01060~~

ATT D CALC 05996.02-G(B)-05-2

D5

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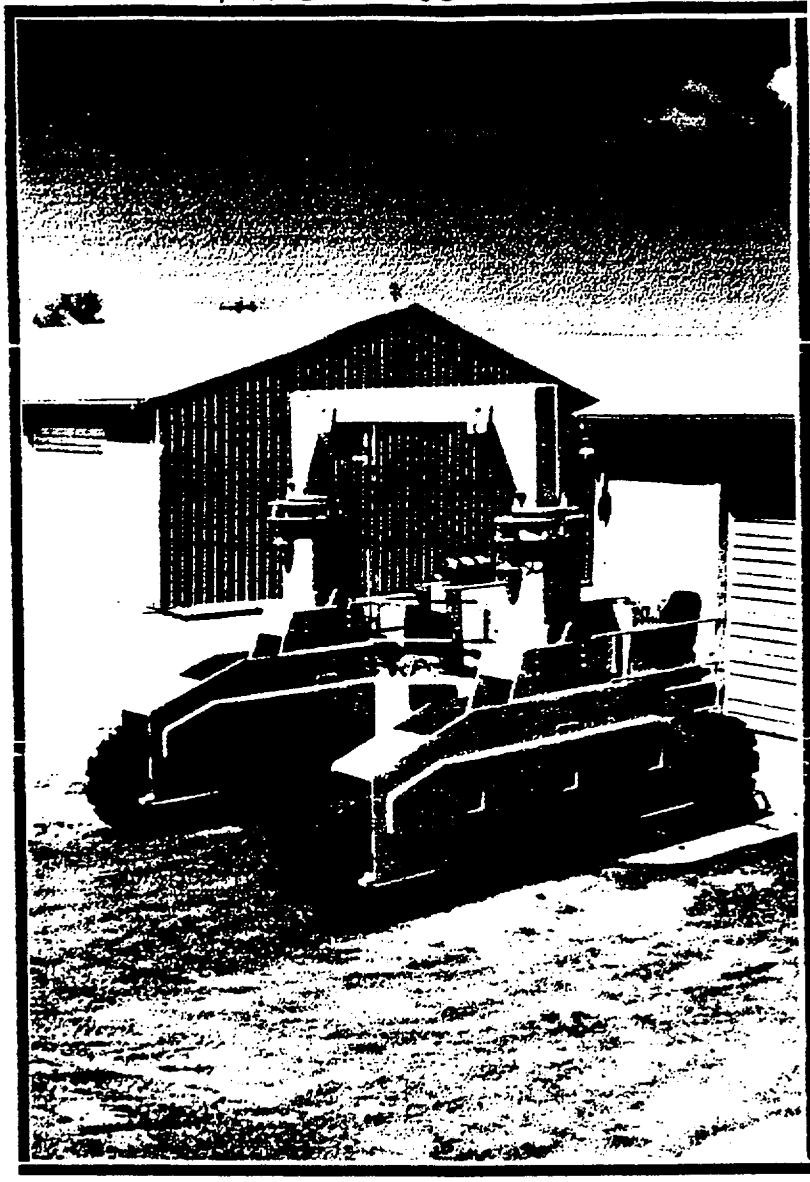

"TOP
LIFT"
CRAWLER
TRANSPORTER

~~01061~~

Att D Calc 05996.02-G(B)-05-2

D6

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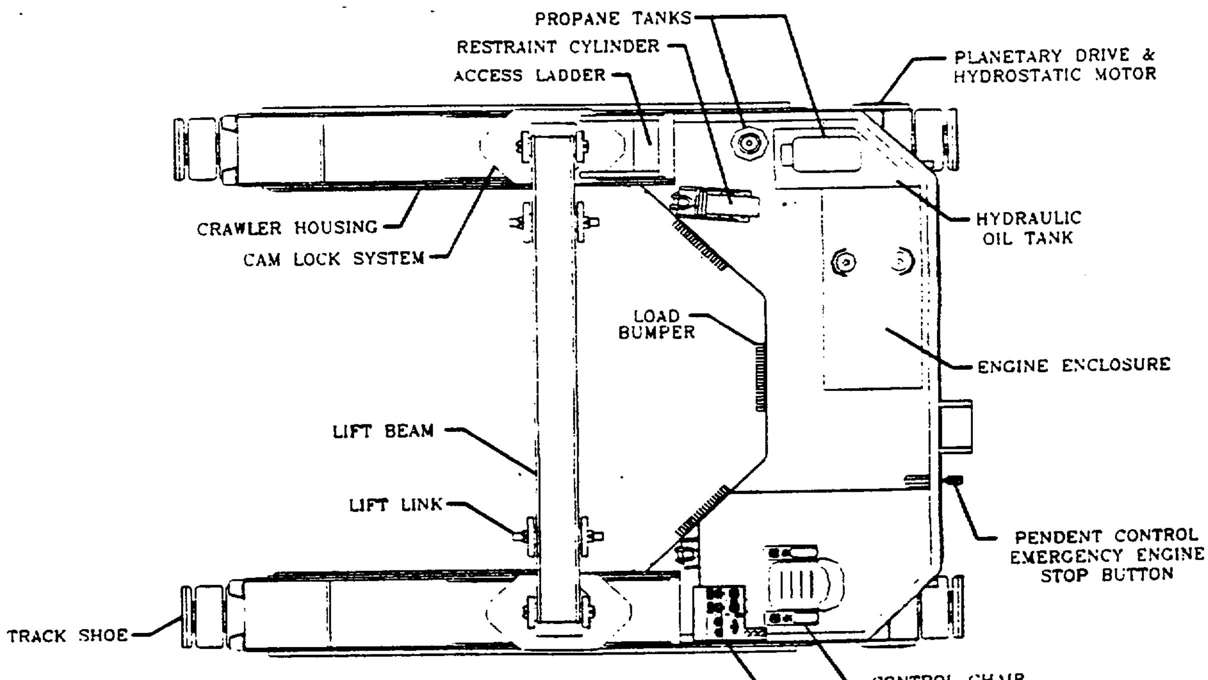
J&R ENGINEERING

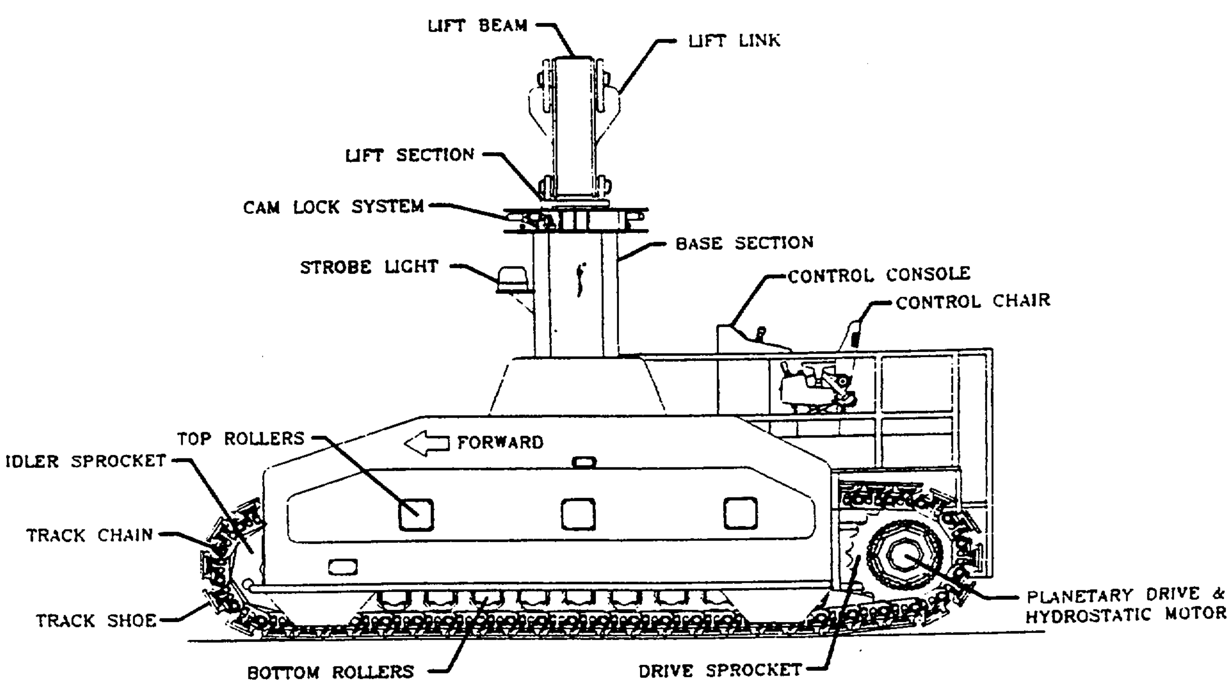
SPECIALIZED LIFTING AND TRANSPORTATION EQUIPMENT

GENERAL ARRANGEMENT

TOP LIFT TRANSPORTER



PLAN VIEW



SIDE VIEW

~~01062~~

Att D CALC 05996.02-G(B)-05-2 D7

ENGINEERING

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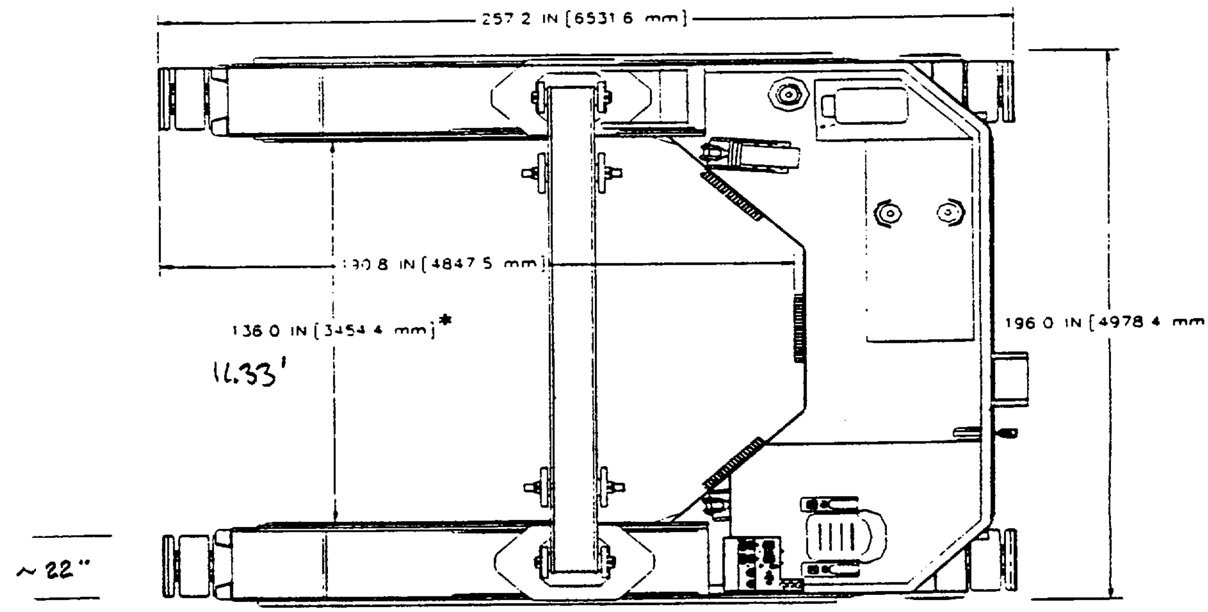
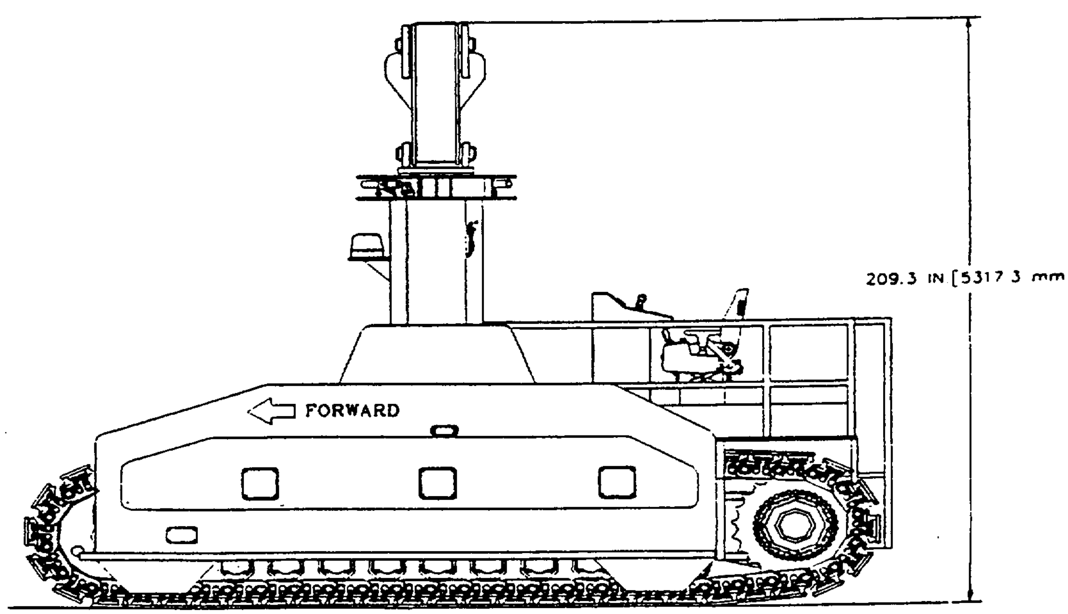
SPECIALIZED LIFTING AND TRANSPORTATION EQUIPMENT

GENERAL DIMENSIONS

TOP LIFT TRANSPORTER



PLAN VIEW
* DETERMINED BY CASK SIZE



SIDE VIEW

~~01063~~

ATT D CALC 05996.02-G(B)-05-2 D8

J&R ENGINEERING CO., INC.
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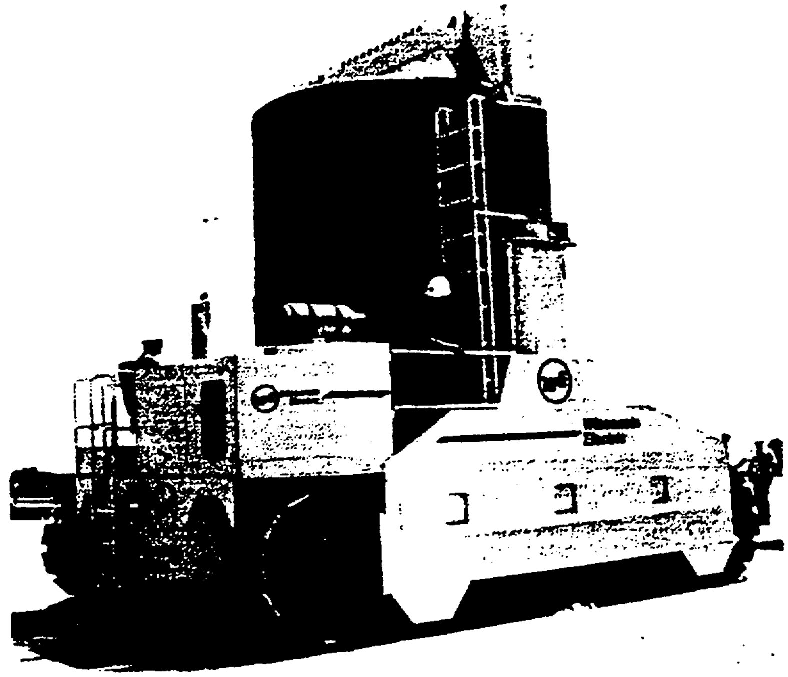
J&R ENGINEERING

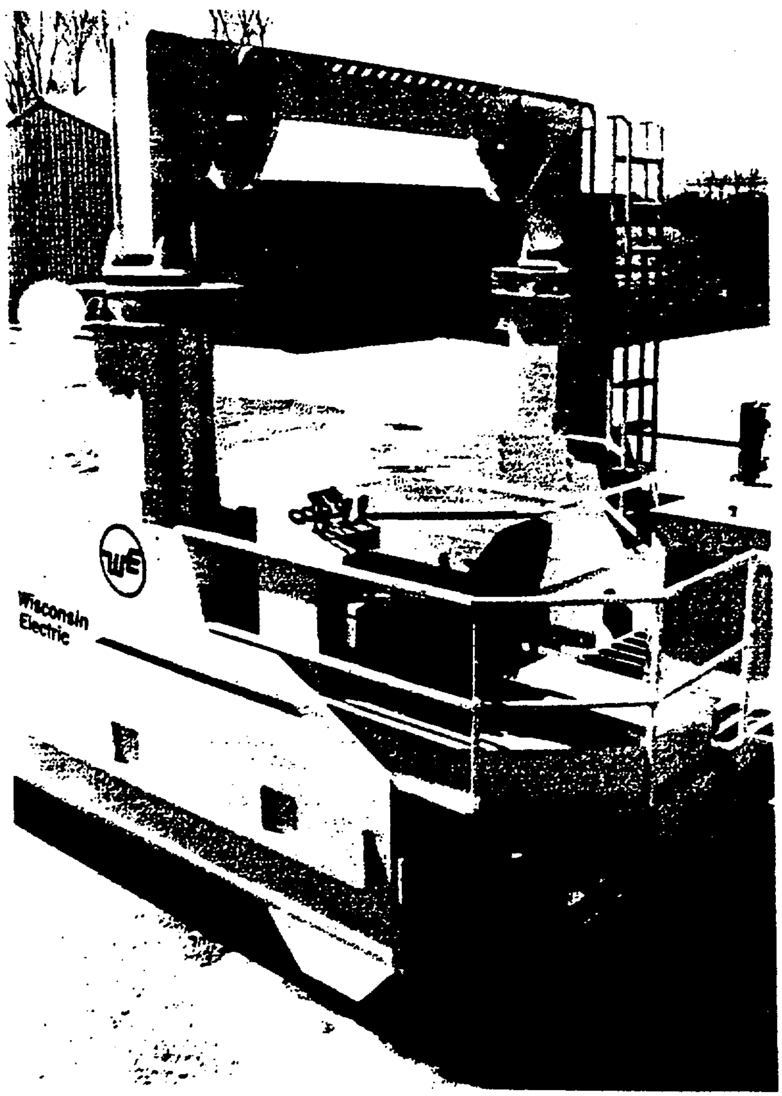
VENTILATED STORAGE CASK TRANSPORTER*

SPECIALIZED LIFTING AND TRANSPORTATION EQUIPMENT

LIFT AND TRAVEL CAPACITY

135 TO 200 U.S. TONS





SPECIFICATIONS

GRADABILITY	25-30%
HORSEPOWER	185 - 220
VARIABLE TRAVEL SPEED	0 - 2.0 MPH
TURNING RADIUS	COUNTER ROTATES*
CAPACITY	135 TO 200 TONS
APPROXIMATE WEIGHT	125,000 TO 135,000 LBS.

CUSTOM CONFIGURATIONS ARE AVAILABLE FOR SPECIAL CLEARANCE PROBLEMS.

* GROUND SURFACES CAN DICTATE RADIUS

* PATENTS PENDING

~~01064~~

ATT D CALC 05996.02-G(B)-05-2

D9

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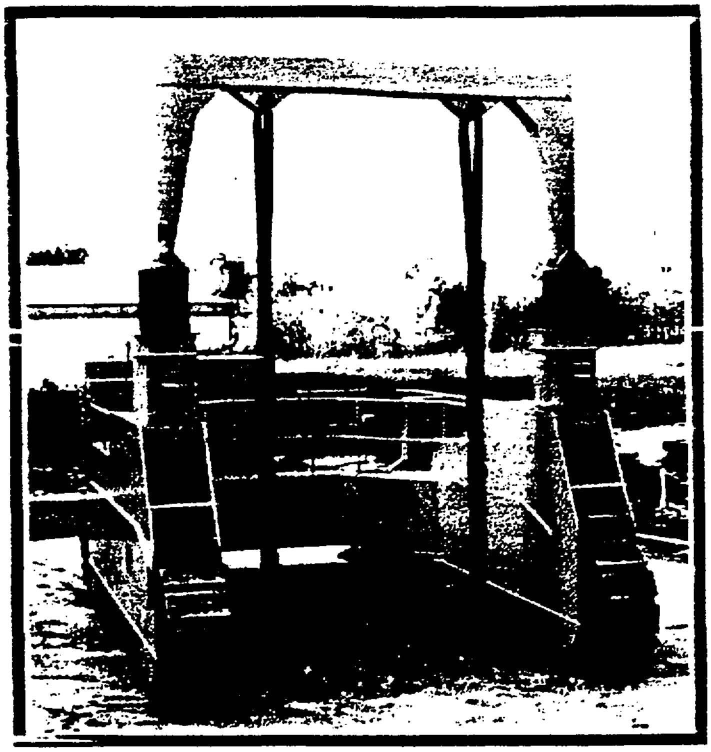
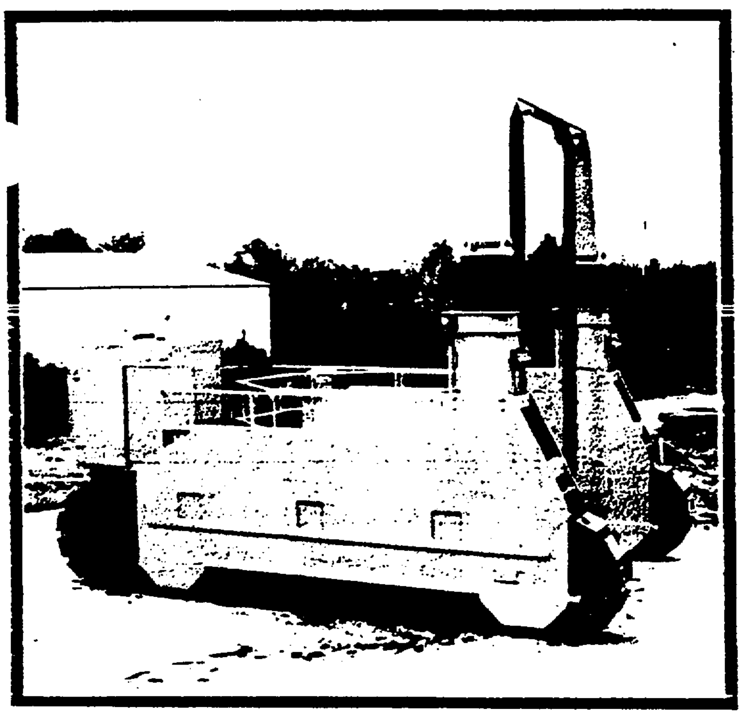
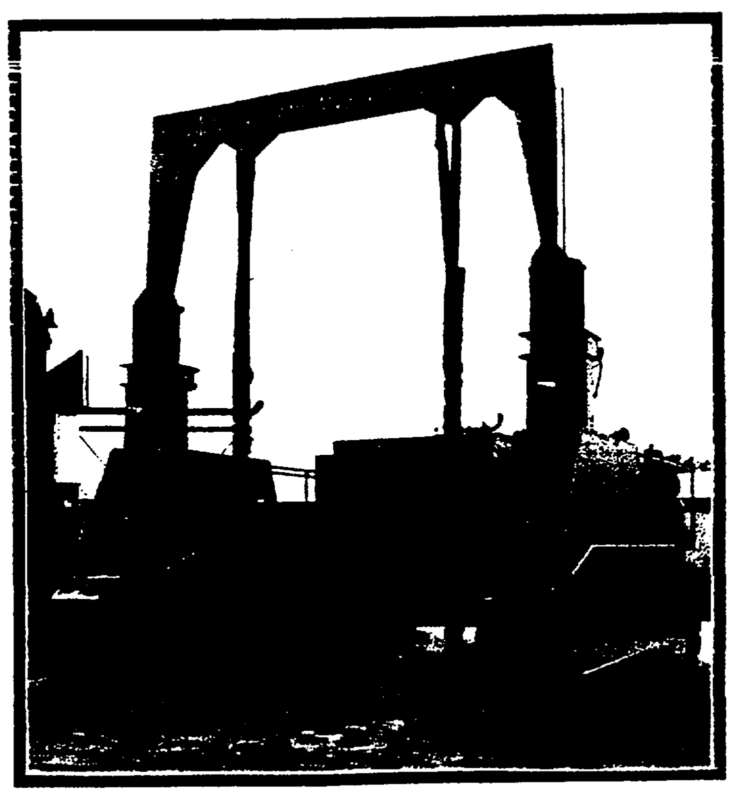


"BOTTOM LIFT" CRAWLER TRANSPORTER





~~01065~~

ATT D CALL 05996.02-G(B)-05-2

D10

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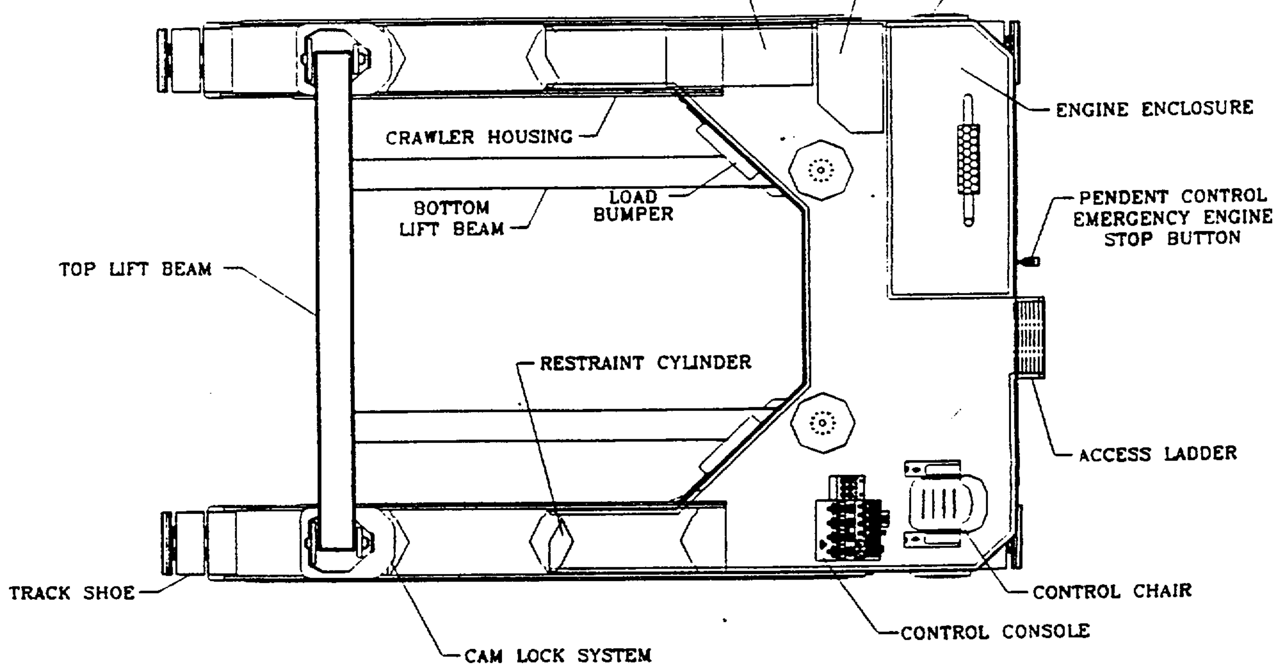
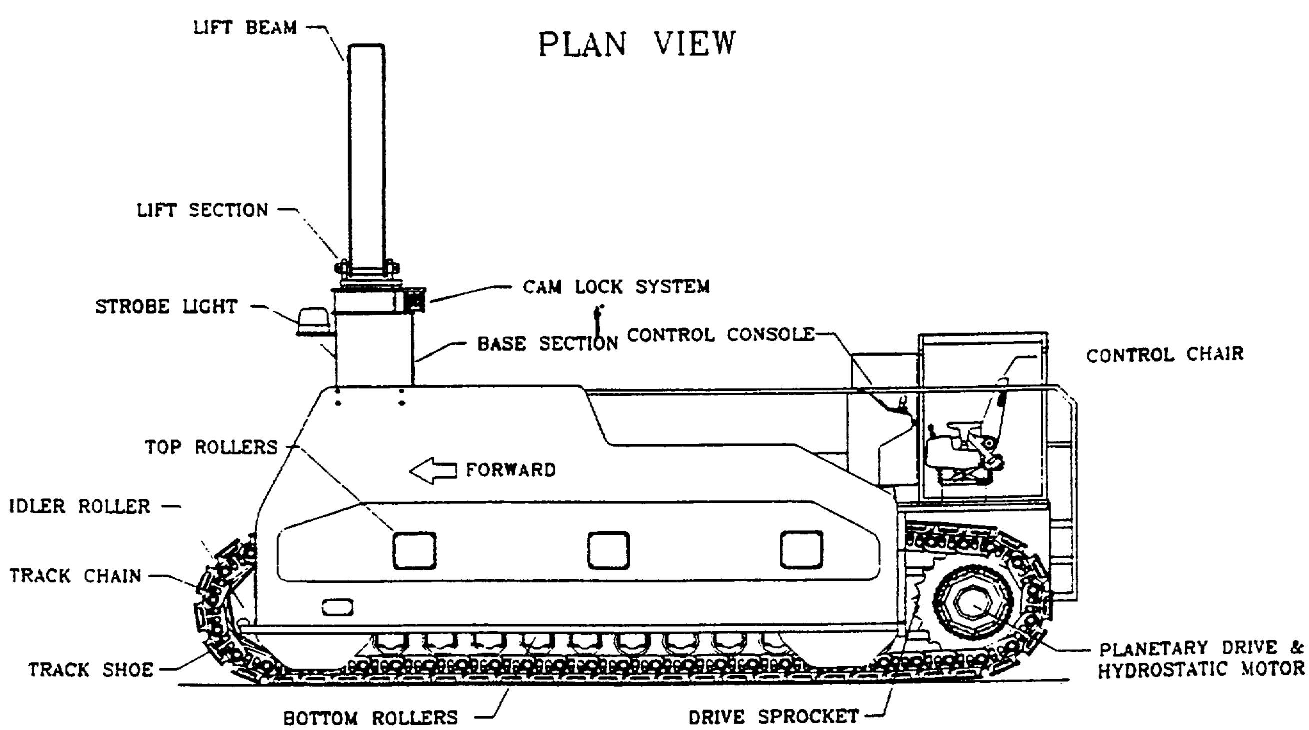
SPECIALIZED LIFTING AND TRANSPORTATION EQUIPMENT

GENERAL ARRANGEMENT

BOTTOM LIFT TRANSPORTER



PLAN VIEW



SIDE VIEW

~~01066~~

Att D Calc 05996.02-G(B)-05-2 D11

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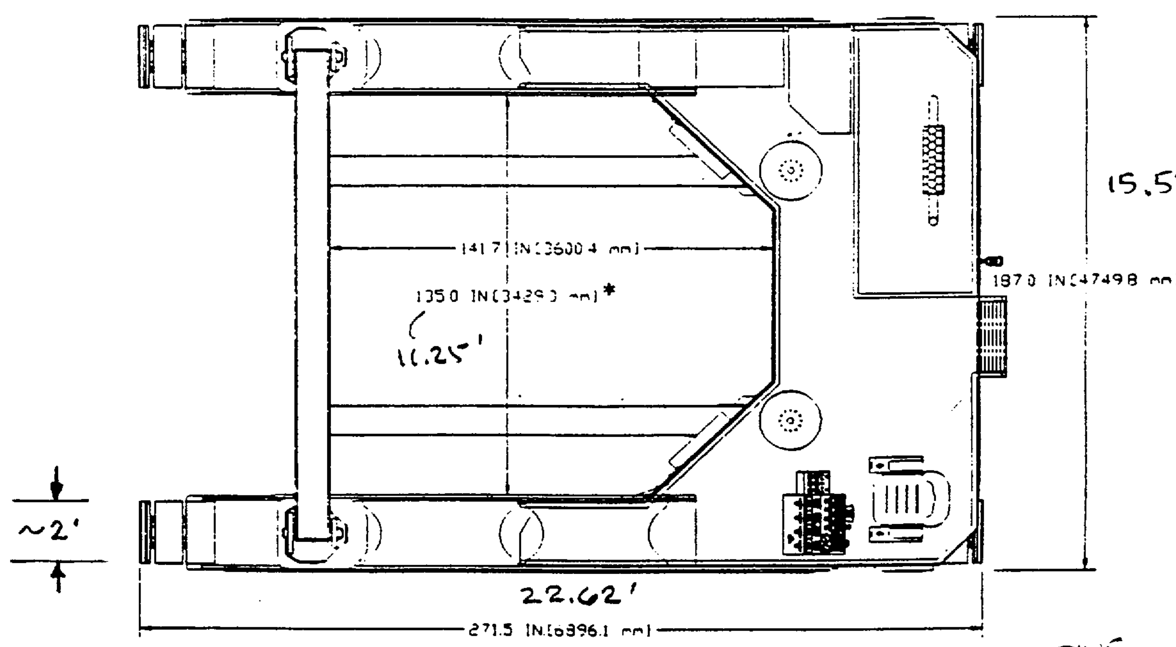
SPECIALIZED LIFTING AND TRANSPORTATION EQUIPMENT

GENERAL DIMENSIONS

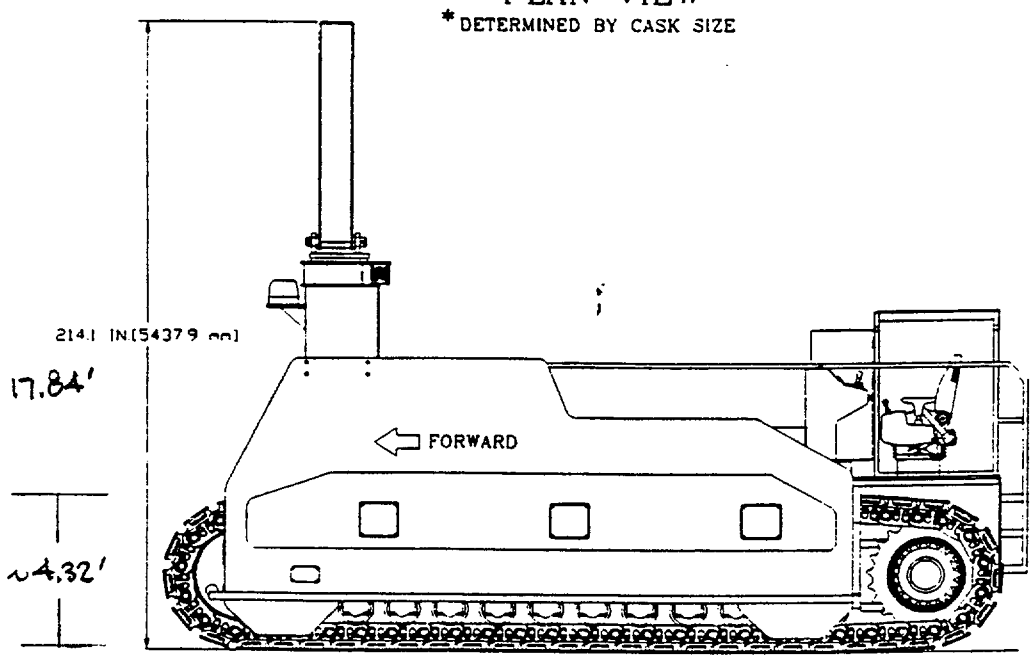
BOTTOM LIFT TRANSPORTER



~15' EFFECTIVE

PLAN VIEW

*DETERMINED BY CASK SIZE



SIDE VIEW

~~01067~~

VSC TRANSPORTER GENERAL SPECIFICATIONS

J&R ENGINEERING CO., INC.
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SPECIALIZED LIFTING AND TRANSPORTATION EQUIPMENT

MAIN FRAME

THE OPEN "C" SECTION ALLOWS FULL ENTRY OF A STORAGE CASK. THE ENGINE, FUEL TANK, HYDRAULIC TANK, AND OPERATOR STATION WITH THE CONTROLS ARE MOUNTED ON THE FRAME REAR CENTER SECTION. FINITE ELEMENT STRUCTURAL ANALYSIS WAS DONE WITH INCREDIBLE EVENT CRITERIA.

PROPEL SYSTEM

TWO INDEPENDENT CLOSED CIRCUIT HYDROSTATIC SYSTEMS EACH DRIVE A 248:1 PLANTETARY THAT DRIVES THE CHAIN SPROCKETS. EACH SYSTEM HAS FULLY VARIABLE PISTON PUMPS AND MOTORS. THE PUMPS ARE INFINITELY VARIABLE FROM 0 TO FULL SPEED BY JOY STICKS THAT ARE MOUNTED IN THE OPERATOR SEAT ARM RESTS. A FIVE SPEED SELECTOR MOUNTED ON THE OPERATING CONSOLE CONTROLS THE VARIABLE MOTORS ALLOWING THE OPERATOR TO SELECT A MAXIMUM SPEED FOR LOADED CONDITIONS. TRAVEL SPEED IS UP TO 1.5 MPH WITHOUT LOAD AND THERE IS 6% GRADEABILITY WITH LOAD.

TRACK SYSTEM

GROUND LOADING IS MINIMIZED BY CHAMFERED FLAT STEEL PLATES MOUNTED TO DOUBLE GROUSER SHOES ON THE CONTINUOUS CHAIN.

BRAKING SYSTEM

DUAL SPRING APPLIED BRAKES ARE AUTOMATICALLY APPLIED WHEN THE OPERATING LEVERS ARE IN NEUTRAL OR THE PARKING BRAKE IS SET.

LIFTING SYSTEM

LIFT-N-LOCK TELESCOPIC LIFTING BOOMS FOR LIFTING THE STORAGE CASKS ARE INTEGRATED INTO THE MAIN FRAME. THE LIFTING CYLINDERS ARE INSIDE THE BOOMS AND HAVE DOUBLE LOCKING VALVES. THE CAM LOCKING SYSTEM ON THE MOVING BOOM SECTIONS ENGAGES AND HOLDS THE LOAD IF THE CYLINDER LOOSES ITS HOLDING POWER. INDICATOR LIGHTS ON THE OPERATING CONSOLE TELL IF THE CAMS ARE DISENGAGED OR SET TO ENGAGE. HEIGHT METERS ON THE OPERATING CONSOLE GIVE 0.1 INCH READINGS FOR EACH LIFTING BOOM.

ATT D CALC 05996.02-G(B)-05-2 D13

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SPECIALIZED LIFTING AND TRANSPORTATION EQUIPMENT

RESTRAINING SYSTEM

HYDRAULIC CYLINDERS ON EACH FRAME ARM HAVE ADJUSTABLE ROD CLEVIS' THAT ATTACH TO A BELT SURROUNDING THE CASK. PRESSURIZING THE CYLINDERS WITH CONTROLS ON THE OPERATING CONSOLE LIMITS CASK MOVEMENT.

UTILITY HYDRAULIC SYSTEM

THE LIFTING AND RESTRAINING SYSTEMS ARE CONTROLLED BY AN INDEPENDENT HYDRAULIC SYSTEM WITH A VARIABLE PISTION PUMP AND PRESSURE COMPENSATED CONTROL VALVES WHICH ARE CONTROLLED BY JOY STICKS ON THE OPERATING CONSOLE.

TOP LIFT CONFIGURATION

LIFT-N-LOCK BOOMS ARE AT THE CENTER OF EACH FRAME ARM WITH A LIFTING BEAM CONNECTED TO THE TOP OF EACH LIFTING BOOM SECTION. BECKETS ON THE BEAMS ARE CONNECTED TO THE LIFTING BECKETS ON THE CASKS.

BOTTOM LIFT CONFIGURATION

TWO LOWER LIFT BEAMS ARE INSERTED INTO THE CASK AIR CHAMBERS AND LIFTED BY LIFT-N-LOCK BOOMS THAT ARE AT THE FRONT OF THE FRAME ARM AS WELL AS LIFT-N-LOCK BOOMS THAT ARE IN THE REAR FRAME CENTER SECTION. THE FORWARD MOUNTED LIFTING BEAM CAN ELEVATE ABOVE THE CASK FOR ENGAGEMENT. AFTER CONNECTING TO THE AIR CHAMBER BEAMS, THE FRONT LIFT BEAMS REMAIN BELOW THE TOP OF THE CASK.

ENGINE

DIESEL OR PROPANE POWER DRIVES A TRIPLE PUMP DRIVE WITH A CLUTCH DISCONNECT.

J & R ENGINEERING CO., INC.
538 Oakland Avenue
Box 447
Mukwonago, WI 53149 USA

FAX COVER SHEET

DATE:	April 15, 1997	TIME:	4:00 pm
TO:	Mr. Paul Trudeau Sturm & Webster	FAX: PHONE:	617-589-8473
FROM:	Roger Johnston J & R Engineering Co., Inc.	PHONE: FAX:	(414) 363-9660 (414) 363-9620

Number of pages including cover sheet: - 11 -

MESSAGE

Per our conversation, enclosed is data on a transporter ground loads. Some additional info from the analysis report is also included.

Please call me if future data is required.

Sincerely,

Roger Johnston
President

ATTACHMENT E CALC 05996.0~~1~~2-G(B)-05
REV 2 PE 1/11

~~01070~~

J&R ENGINEERING COMPANY, INC.
536 OAKLAND AVENUE
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J&R ENGINEERING

JOB NAME: PROPEL SYSTEM
JOB NO.: SN 1374 PAGE ___ OF ___
CALCULATED BY: KLJ DATE: 7/18/95
VERIFIED BY: ___ DATE: ___
SCALE: fn:1374P4

GROUND LOADING WITH 200 TON RATED LOAD

MACHINE WEIGHT = $MW := 1.3 \cdot 10^5$

MAXIMUM LOAD RATING = $LR := 4.0 \cdot 10^5$

NUMBER OF GROUND SHOES = $GS := 2 \cdot 18$ (18 SHOES / TRACK)

GROUND AREA PER SHOE = $A := 10 \cdot 21$ (10" LONG x 21" WIDE)

GROUND LOADING = $LG := \frac{MW + LR}{(GS \cdot A)}$ $LG = 70.106$ LBS. PER SQ. IN. = 10.1 KSF

GROUND LOADING WITH 175 TON LIVE LOAD

MACHINE WEIGHT = $MW := 1.3 \cdot 10^5$

LIVE LOAD = $LR := 3.5 \cdot 10^5$

NUMBER OF GROUND SHOES = $GS := 2 \cdot 18$

GROUND AREA PER SHOE = $A := 10 \cdot 21$

GROUND LOADING = $LG := \frac{MW + LR}{(GS \cdot A)}$ $LG = 63.492$ LBS. PER SQ. IN. = 9.14 KSF

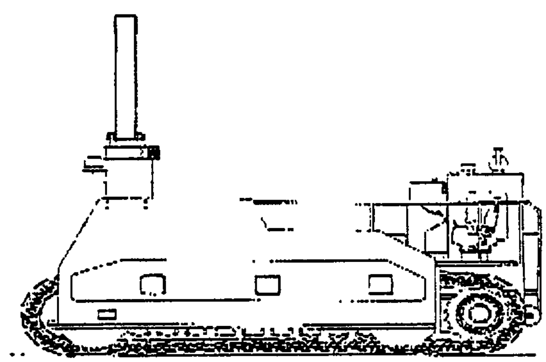
ATTACHMENT E CALC 05996.01-G(B)-05-0 p E2 2/2

~~01071~~

ENGINEERING ANALYSIS REPORT

BOTTOM LIFT VENTILATED STORAGE CASK TRANSPORTER

PROJECT # 1374



PREPARED BY:

J & R ENGINEERING CO., INC

538 OAKLAND AVE., P.O. BOX 447
MUKWONAGO, WI 53149

JULY, 1996

ATTACHMENT E CALC 05996.0~~1~~2-G(B)-05-~~0~~2 p E3

~~01072~~

J&R ENGINEERING

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SPECIALIZED LIFTING AND TRANSPORTATION EQUIPMENT

EXECUTIVE SUMMARY

This data has been compared to the engineering data on the field proven top lift transporter that has similar structures. Also, both the top lift and this bottom lift transporters have been finite element analyzed by the University of Wisconsin College of Engineering with satisfactory results. Their summary analyses are not included in this summary and are available for review at J & R Engineering.

J & R Engineering uses AISC structural recommendations as minimum values and most of the product structures have a target safety factor of 2.2. When the transporters are used in the normal intended operating conditions the theoretical safety factors exceed 2.2 to 1.

The structural safety factors were a prime concern when developing the transporter as well as all of the safety systems such as the cam locks, locking valves and braking system. We believe the machine to be as safe as possible with the components and procedures that are available for manufacturing this type of unit.

JUNE 1996

FN:ENG1374

ATTACHMENT E CALC 05996.02-G(B)-05-2 p E4

TABLE of CONTENTS

ATTACHMENT E CALC 05996.01-G(B)-05-0 p E5

INTRODUCTION

This engineering analysis summary report is on the model 180T "Bottom Lift" LIFT-N-LOCK CRAWLER TRANSPORTER.

The analysis was based on the RATED LOAD conditions and on INCREDIBLE EVENT load conditions.

The RATED LOAD condition is lifting and propelling with the rated load of 200 U.S. tons on level ground not exceeding a 6% grade in any direction. This loading is projected to be the maximum and can never be exceeded. The maximum cask weight is reported to be 175 U.S. tons.

The INCREDIBLE EVENT load conditions are theoretical reaction forces from the machine and load being in an equilibrium instable or near tipping condition. This condition is considered nearly impossible to obtain and the results of these forces are satisfactory if the structures will (theoretically) not have a catastrophic failure. These conditions are described as follows:

1. Front idler force of the GVW plus live load (175 U.S. tons). This constitutes the transporter being in a forward equilibrium tipping mode. The resultant maximum strain levels are known to be forward of the center section structure to track arm structure connection.

2. One half of the GVW plus live load (175 U.S. tons) on one of the track arms. The resultant maximum strain levels are known to be at the center of the center structure from torsional loading.

3. Side track roller force of the GVW plus live load (175 U.S.ton) longitudinally on one track arm structure. This constitutes the transporter being in a side equilibrium tipping mode. The resultant maximum strain levels are known to be forward of the center section structure to track arm structure connection and torsional loading at the center of the center structure.

JUNE 1996

ATTACHMENT E CALC 05996.02-G(B)-05-2 p E6

J&R ENGINEERING COMPANY, INC.
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JOB NAME ALLOWABLE STEEL STRESS DATA
JOB NO. SN 1374 fn:A1 PAGE 1 OF 4
CALCULATED BY RLJ DATE 4/8/96
VERIFIED BY ______ DATE ______
SCALE ______

THE FOLLOWING ALLOWABLE STRESSES ARE BASED PRIMARILY ON AISC MANUAL OF STEEL CONSTRUCTION, FIRST REVISION OF THE NINTH EDITION, JANUARY 1991

STEEL STRESSES

DEFINITION	EQUIVALENT SYMBOLS
Yield strength	Fy
Allowable tensile stress	Ft = .67 Fy
Allowable compressive stress	Fc = .58 Fy
Allowable bending stress (tension)	Fbt = .67 Fy
Allowable bending stress (compression)	Fbc = .58 Fy
Allowable shear stress	Fv = .38 Fy
Allowable bearing stress	Fbrg = .93 Fy
Allowable buckling stress	
< 20	Fa = Fc
20 < < 200	Fa = (1/w) Fc
= effective length / section ratio	kl / r
w = buckling coefficient	

BUTT WELD STRESSES

DEFINITION	EQUIVALENT SYMBOLS
Allowable tensile stress	Ftw = .53 Fy
Allowable compressive stress	Fcw = .52 Fy
Allowable bending stress	Fbw = .46 Fy
Allowable shear stress	Fvw = .31 Fy

FILLET WELD STRESSES

DEFINITION	EQUIVALENT SYMBOLS
Allowable tensile stress	Ftw = .53 Fy
Allowable compressive stress	Fcw = .46 Fy
Allowable shear stress	Fvw = .31 Fy

ATTACHMENT E CALC 05996.01-G(B)-05-0 p E7 of 2 2

01076

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PROPEL DATA

JOB NAME: SN 1374
JOB NO. ____ PAGE ____ OF ____
CALCULATED BY: KLJ DATE: 7/18/95
VERIFIED BY ____ DATE ____ fn:1374H1B
SCALE ____

FINAL REDUCTION DRIVE

MAKE: O&K ANTRIEBSTECHNIK
MODEL: F220
RATIO: 248
RATED OUTPUT TORQUE: 248 kNm (183024 FtLbs)
EFFICIENCY = 90%
CHAIN PIN RADIUS = 18.994"
SHOE PLATE RADIUS = 24.690"
EST. CHAIN EFFICIENCY = 70%

OUTPUT @ SPROCKET

$RPM3 := \frac{RPM}{248}$ $RPM3 = 5.212$ SPEED @ MAX. DISP.

$RPM4 := \frac{RPM2}{248}$ $RPM4 = 16.964$ SPEED @ MIN. DISP.

$T3 := (T1 \cdot 248) \cdot .90$ $T3 = 1.648 \cdot 10^6$ TORQUE @ MAX. DISP.
NOTE: 75% OF RATED TORQUE

$T4 := (T2 \cdot 248) \cdot .90$ $T4 = 9.987 \cdot 10^5$ TORQUE @ MIN. DISP.

SPEED @ TRACK SHOE

$CIR := 2 \cdot 24.69 \cdot \pi$ $CIR = 155.132$

$FtperMin := \left(\frac{CIR}{12}\right) \cdot RPM3$ $FtperMin = 67.383$

MAX. SPEED @ MAX GRADEABILITY

$MPH := FtperMin \cdot .01136$ $MPH = 0.765$

MAX. SPEED @ MIN. GRADEABILITY

$FtperMin2 := \left(\frac{CIR}{12}\right) \cdot RPM4$ $FtperMin2 = 219.31$

$MPH2 := FtperMin2 \cdot .0113$ $MPH2 = 2.491$

NOTE: LIMIT TO 1.5 MPH

TRACTIVE EFFORT

$TE1 := \left(\frac{2 \cdot T3}{24.690}\right) \cdot .70$

$TE1 = 9.342 \cdot 10^4$ Lbs.

$A := \sinh\left(\frac{TE1}{5.3 \cdot 10^5}\right)$

$A = 0.177$ ANGLE = 10.19 DEGREE

GRADEABILITY
WITH 175 TON LIVE LOAD
17.98% LOW SPEED

$TE2 := \left(\frac{2 \cdot T4}{24.690}\right) \cdot .70$

$TE2 = 5.663 \cdot 10^4$ Lbs.

$A1 := \sinh\left(\frac{TE2}{5.3 \cdot 10^5}\right)$

$A1 = 0.107$ ANGLE = 6.14 DEGREE

10.76% HIGH SPEED

ATTACHMENT E CALC 05996.01-G(B)-05-02 p E8

JR ENGINEERING

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SPECIALIZED LIFTING AND TRANSPORTATION EQUIPMENT

DIMENSIONS & WEIGHTS OF MAJOR SUBASSEMBLIES ON ZNPP VSC BOTTOM LIFT TRANSPORTER

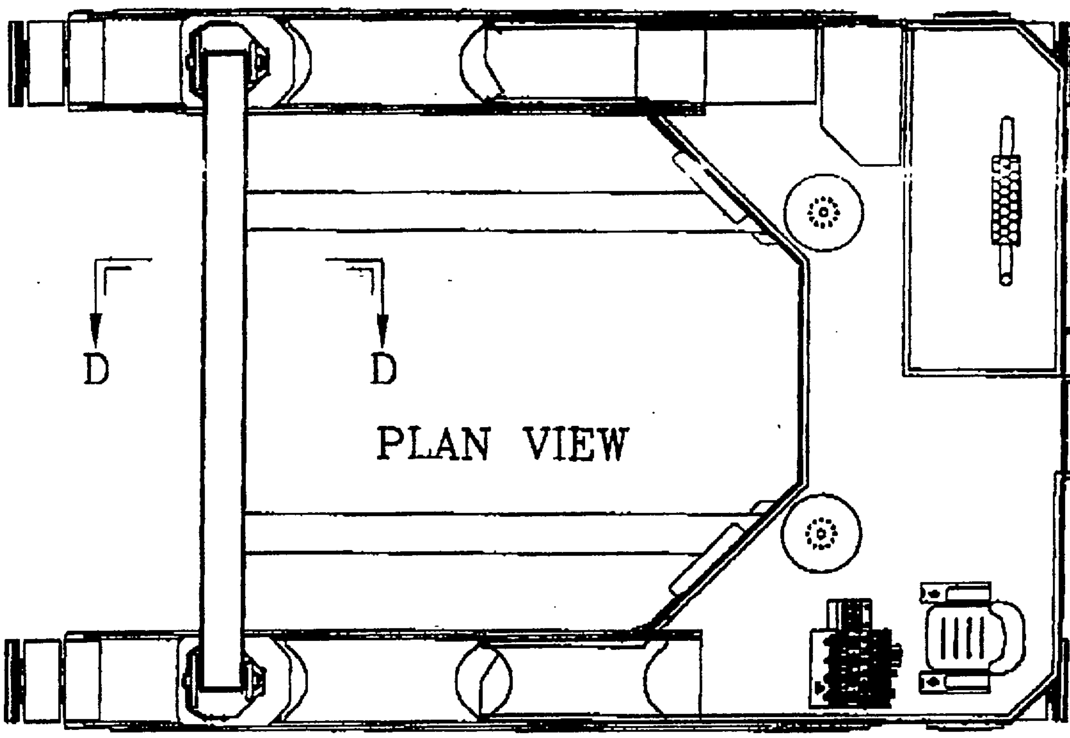
COMPONENT	QTY.	WEIGHT EACH (Lbs./Kg.)	DIMENSIONS (mm)
TOP LIFTING BEAM	1	3,230 / 1,465	330 x 1,981 x 4,216
SHIPPING BEAM	1	1,470 / 667	356 x 356 x 3,480
LIFTING PENDANTS	2	87 / 40	127 dia. x 4,953 to centers
BOTTOM LIFTING BEAM	2	2,500 / 1,134	293 x 420 x 4,140
FRONT LIFT CYLINDER & BOOM	2	3,557 / 1,614	610 x 813 x 2,109
REAR LIFT CYLINDER & BOOM	2	890 / 404	407 x 458 x 991
MAIN ENGINE	1	1,390 / 630	966 x 1,778 x 1,321
CRAWLER TRACK	2	8,060 / 3,656	254 x 534 x 14,072
DRIVE CHAIN TENSION SPRING	2	1,442 / 654	300 x 450 x 1,536
DRIVE CHAIN IDLER SPROCKET	2	1,277 / 579	490 x 835 x 1268
CRAWLER DRIVE PUMP	2	112 / 61	212 x 250 x 290
PLANETARY GEAR HOUSING	2	1,897 / 860	650 x 650 x 580

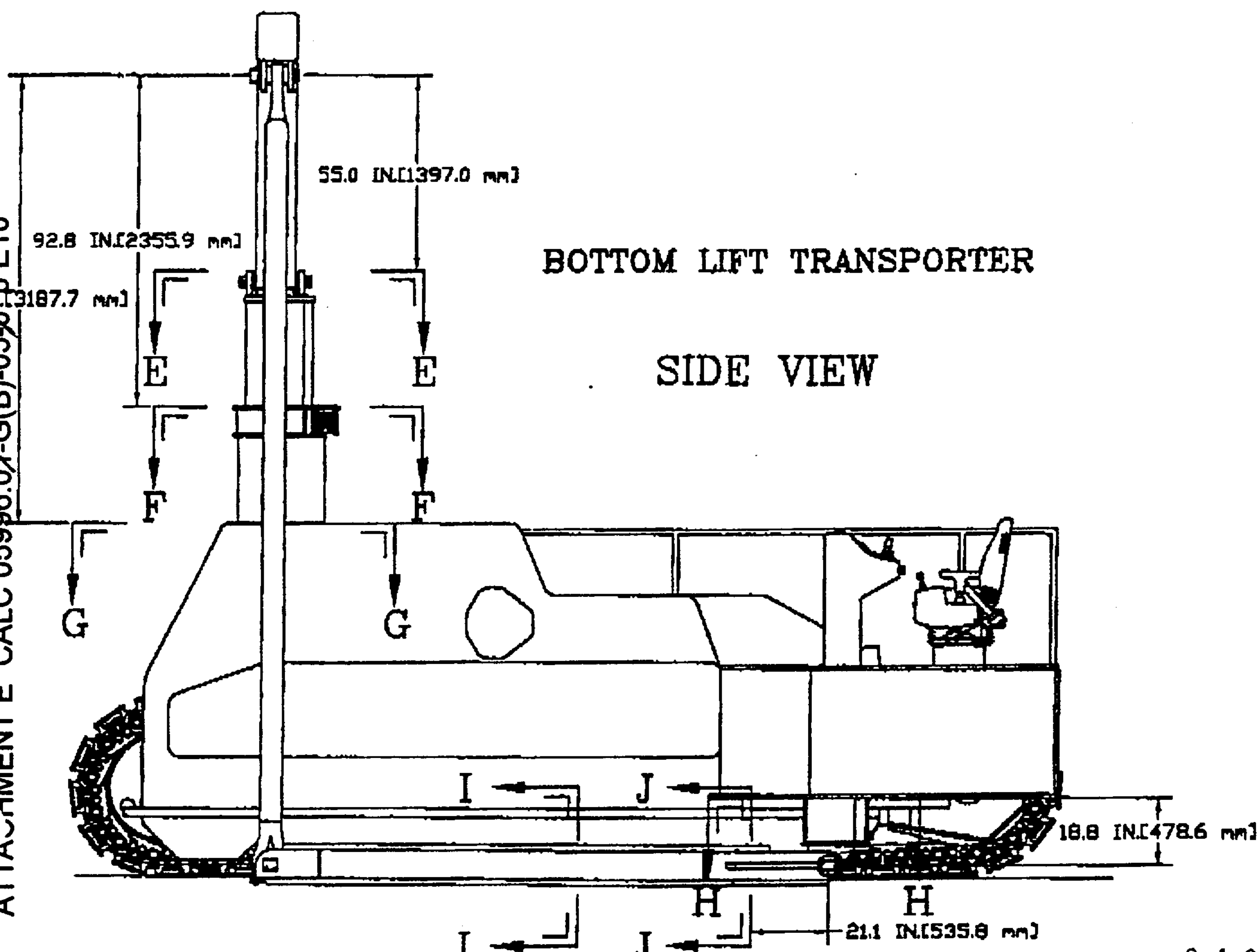
ATTACHMENT E CALC 05996.~~01~~2-G(B)-05-~~0~~2 p E9

~~01078~~

LIFT BEAMS & BOOMS

SECTION DIMENSIONS



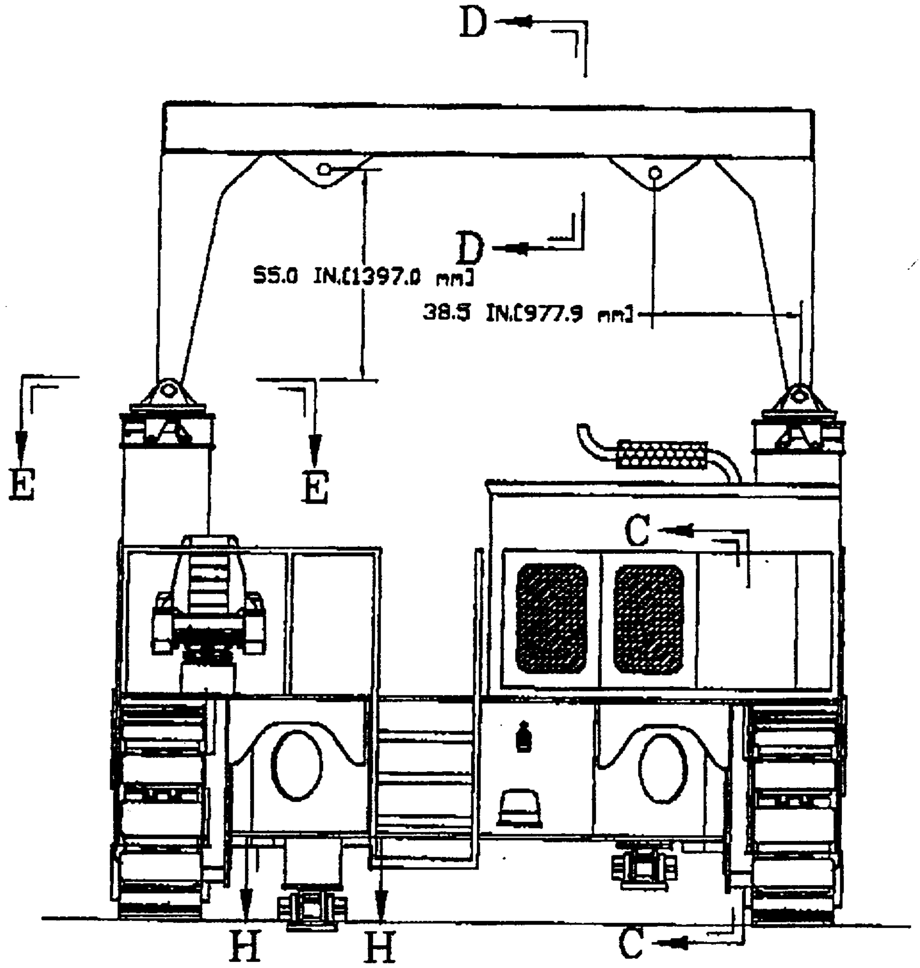


ATTACHMENT E CALC 05996.01-G(B)-05-0 E10

~~01079~~

LIFT BEAMS & BOOMS

SECTION DIMENSIONS



ATTACHMENT E CALC 05996.01-G(B)-05-02 p E11

ATTACHMENT F TO CALC 05996.02-G(B)-05-2 F1/1

NOTES OF TELEPHONE CONVERSATION

PRIVATE FUEL STORAGE FACILITY, LLC

JO 05996.01
Date: 03-19-97
Time: 2:00 PM

To: Richard Weigel: City of Toole,UT, Building Official (801) 843-2133

From: Paul J. Trudeau: SWEC-Boston 245/03 (617)589-8473

Subject: DEPTH OF FOOTINGS REQUIRED FOR FROST PROTECTION IN TOOELE, UT

Discussion:

PJT asked what minimum depth of footing is required for protection against frost for new industrial construction in Tooele, UT, and on which regulation is this based.

Dick Weigel indicated that the minimum depth of footings is 30 inches to provide protection against frost. He also indicated that an additional 6 inch clearance is required from finished grade to any wood in the structure and that the required snow loading is 45 psf. He said that these were based on the Utah Uniform Building Standards Act, annotated(?) 1953 and revised July 1, 1996.

Copy to: NTGeorges - Boston 245/03
SMMacie - Denver

Private Fuel Storage Facility

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-05
Revision No. 2

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
Method			
Identify the method used to verify the "Method" of the calculation			
• By design review	✓		
• Compare the Method with another calculation			✓
• Alternate calculation			✓
If the compare method was used, is the statement identifying the other calculation identified in this calculation?			✓
If an alternate calculation was used for a QA Category I calculation, is it included with the calculation?			✓
Is the calculation method acceptable?	✓		
Assumptions			
Affirmative answers to the following questions are required:			
• Are all assumptions uniquely identified as assumptions and adequately described?	✓		
• Are all assumptions reasonable?	✓		
• Are all assumptions that require confirmation at a later date specifically identified as assumptions that must be confirmed?	✓		
For Revisions to the Calculation			
• Are changes clearly identified?	✓		
• For QA Category I calculations, is a reason for the revision given?	✓		
• Does the calculation identify the calculation, including revision, when applicable, which is superseded?	✓		

Private Fuel Storage Facility

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-05
Revision No. 2

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
• Are affected pages identified with the new calculation number or revision number?	✓		
• When applicable, is an alternate calculation included as part of the calculation?			✓
• When applicable, is a statement identifying the calculation to which the method was compared included as part of the revision?			✓

Thomas Y. Chang	Thomas Y. Chang	June 15, 2000
Printed Name	Signature	Date

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

5010.64 (FRONT)

CLIENT & PROJECT	PAGE 1 OF
PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY	52 + 6 pp OF ATTACH'S

CALCULATION TITLE (Indicative of the Objective):	QA CATEGORY (✓)
STABILITY ANALYSES OF THE CANISTER TRANSFER BUILDING SUPPORTED ON A MAT FOUNDATION	☒ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER

CALCULATION IDENTIFICATION NUMBER

J. O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	G(B)	13	—	345AD

*APPROVALS – SIGNATURE & DATE

PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION *REQUIRED (✓) YES	NO
L.P. SINGH L.P. Singh 12-9-98	D.L. Aloysius D.L. Aloysius 12-10-98	D.L. Aloysius D.L. Aloysius 12-10-98	0	-		✓
DL ALOYSIUS 9-3-99 D.L. Aloysius SY BOAKYE 9-3-99 S.Y. Boakye	SY BOAKYE 9-3-99 S.Y. Boakye DL ALOYSIUS 9-3-99 D.L. Aloysius	TY CHANG 9-3-99 Thomas Y. Chang T Y Chang 9-3-99 Thomas Y. Chang	1	G(C)-13 Rev 0		✓
SEE PAGE 2-1 FOR ID OF PREPARED/REVIEWED BY						
PAUL J. TRUDEAU 1-21-2000 Paul J. Trudeau	Thomas Y. Chang 1-21-2000 Thomas Y. Chang	Thomas Y. Chang 1-21-2000 Thomas Y. Chang	2	G(B)-13 Rev. 1		
Paul J. TRUDEAU 6-19-2000 Paul J. Trudeau	Thomas Y. Chang 6-19-2000 Thomas Y. Chang	Thomas Y. Chang 6-19-2000 Thomas Y. Chang	3	G(B)-13 Rev. 2		

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	FIRE FILE	✓			
GEOTECH	SWEC - BOSTON	✓			

TABLE OF CONTENTS

RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1

Page count increased from 37 to 63.

- Revised seismic loadings to correspond to the PSHA 2,000-yr return period earthquake (p. 9-1)
- Added section on dynamic strength of soils (p. 9-3)
- Added section on seismic sliding resistance of the mat foundation (p. 9-5)
- Added section on evaluation of sliding on a deep slip surface (p. 9-8)
- Updated bearing capacity analysis using revised seismic loadings (p. 34-1)
  - Added additional loading combination: static + 40% seismic uplift + 100% in x (N-S) direction + 40% in z (E-W) direction
- Added additional references (p. 36-1)

NOTE:

SYBoakye prepared/DLAloysius reviewed pp. 9-8 through 9-12. Remaining pages prepared by DLAloysius and reviewed by SYBoakye.

REVISION 2

Major re-write of the calculation.

1. Renumbered pages and figures to make the calculation easier to follow.
2. Changed effective length of mat to 265 ft to make it consistent with Calculation 05996.02-SC-4, Rev 1 (SWEC, 1999a).
3. Added overturning analysis.
4. Corrected calculation of moments for joints 3 and 6 in Table 1 and incorporated revised seismic loads in calculations of overturning stability and dynamic bearing capacity.
5. Revised dynamic bearing capacity analyses to utilize only total strength parameters because these partially saturated soils will not have time to drain fully during the rapid cycling associated with the design basis ground motion. See Calculation 05996.02-G(B)-05-1 (SWEC, 1999b) for additional details.
6. Updated references to current issues of drawings.
7. Added references to foundation profiles through Canister Transfer Building area presented in SAR Figures 2.6-21 through 23.

8. Deleted analyses of bearing capacity on layered profile, as adequate factors of safety are obtained conservatively assuming that the total strengths measured for the clayey soils in the upper ~25' to 30' layer apply for the entire profile under the Canister Transfer Building and revised all of the detailed bearing capacity analyses.
9. Changed "Load Combinations" to "Load Cases" and defined these cases to be consistent throughout the various stability analyses included herein. These are the same cases as are used in the stability analyses of the cask storage pads, Calculation 05996.02-G(B)-04-5 (SWEC, 2000).
10. Added analysis of sliding on a deep plane at the top of silty sand/sandy silt layer, incorporating passive resistance acting on the block of clayey soil and the foundation mat overlying this interface.
11. Revised Conclusions to reflect results of these changes.

REVISION 3

1. Added a 1-ft deep key around the perimeter of the Canister Transfer Building mat to permit use of the cohesive strength of the in situ silty clay/clayey silt in resisting sliding due to loads from the design basis ground motion.
2. Revised shear strength used in the sliding stability analyses of the Canister Transfer Building mat supported on the in situ silty clay to be the strength measured in the direct shear tests performed on samples obtained from elevations approximately at the bottom of the 1-ft deep perimeter key. The shear strength used in this analysis equaled that measured for stresses corresponding to the vertical stresses at the bottom of the mat following completion of construction.
3. Removed static and dynamic bearing capacity analyses based on total-stress strengths.
4. The relative strength increase noted for the deeper lying soils in the cone penetration testing that was performed within the Canister Transfer Building footprint was used to determine a weighted average undrained strength of the soils in the entire upper layer for use in the bearing capacity analyses, since the soils within a depth equal to approximately the width of the foundation are effective in resisting bearing failures. This resulted in the average undrained strength for the bearing capacity analyses of the upper layer equal to 3.18 ksf.
5. Removed dynamic analyses based on increasing strengths of the cohesive soils that were measured in static tests to reflect well known phenomenon that the strength of cohesive soils increases as the rate of loading decreases.
6. Revised undrained shear strength of the clay block overlying the cohesionless layer to 2.2 ksf, based on the UU tests that were performed at confining pressures of 1.3 ksf (reported in Attachment 2 of Appendix 2A of the SAR) in the analysis of sliding of the Canister Transfer Building on deep plane of cohesionless soils.

7. Added shearing resistance available on the ends of the block of clay, since this soil must be sheared along these planes in order for the Canister Transfer Building to slide on a deep plane of cohesionless soils.
8. Revised method of calculating the inclination factor in the bearing capacity analyses to that presented by Vesic in Chapter 3 of Winterkorn and Fang (1975). Vesic's method expands upon the theory developed by Hansen for plane strain analyses of footings with inclined loads. 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 Canister Transfer Building.
9. Replaced Tables 2, 2.6-9, and 2.6-10 with revised results for the changes in shear strength of the in situ soils noted above and deleted Table 3.

OBJECTIVE

To determine the stability against overturning, sliding, and static and dynamic bearing capacity failure of the Canister Transfer Building supported on a mat foundation.

ASSUMPTIONS/DATA

The footprint of the Canister Transfer Building foundation mat is shown on SWEC Drawing 0599601-EA-8-D, Canister Transfer Building – Floor Plan, and Drawing 0599601-EM-1-D, Canister Transfer Building – General Arrangement Sheet 1. The elevation view of the structure is shown on Drawing 0599601-EA-9-D, Canister Transfer Building – Elevations Sheet 1, and Drawing 0599601-EM-1-D, Canister Transfer Building – General Arrangement Sheet 2. As indicated in SAR Section 4.7.1.5.1, Structural Design, the mat foundation is 5 ft thick. The foundation mat is modeled as 165 ft x 265 ft x 5 ft thick. These are the effective dimensions that were developed and used in Calculation 05996.02-SC-4, Rev 1 (SWEC, 1999a).

Figure 1 presents a schematic view of the foundation and identifies the coordinate system used in these analyses. Figure 2 presents the stick model used in the structural analysis of the Canister Transfer Building.

The various static and dynamic loads and load combinations used in these analyses were obtained from Calculation 05996.02-SC-5-1 (SWEC, 1999b). All loads are transferred to the bottom of the mat. Moments, when transferred to the bottom of the mat, result in eccentricity of the applied load with respect to the center of gravity of the mat. Lateral loads, when combined with the vertical load, result in inclination of the vertical load, which decreases the allowable bearing capacity.

The generalized soil profile at the site is shown on Figure 3. The soil profile consists of ~30 ft of silty clay/clayey silt with sandy silt/silty sand layers (Layer 1), overlying ~30 ft of very dense fine sand (Layer 2), overlying extremely dense silt (N ≥100 blows/ft, Layer 3). SAR Figures 2.6-21 through 23 present foundation profiles showing the relationship of the Canister Transfer Building with respect to the underlying soils. These profiles, located as shown in SAR Figure 2.6-18, provide more detailed stratigraphic information, especially within the upper ~30-ft thick layer at the site.

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

GEOTECHNICAL PROPERTIES

Based on laboratory test results presented in Table 3 of Calculation 05996.02-G(B)-5-2 (SWEC, 2000), γ_{moist} = 80 pcf above the bottom of the mat and 90 pcf below the mat.

Table 6 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A) summarizes the results of the triaxial tests that were performed within depths of ~10 ft. The undrained shear strengths (s_u) measured in these tests are plotted vs confining pressure in Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A). 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 A), 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 A) illustrates that the undrained strength of these soils increase as the loadings of the structures are applied; therefore, 2.2 ksf is a very conservative value for use in the bearing capacity analyses of these structures.

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

In determining the bearing capacity of the foundation, the average shear strength of the soils along the anticipated bearing capacity failure slip surface should be used. This slip surface is normally confined to the zone within a depth below the footing equal to the minimum width of the footing. For the Canister Transfer Building, the effective width of the footing is decreased because of the large eccentricity of the load on the mat due to the seismic loading. As indicated in Table 2.6-10, the minimum effective width of the Canister Transfer Building occurs for Load Case IIIA, where B' = 38.2 ft. This is greater than the depth of the upper layer (~30 ft). Therefore, it is reasonable to use the average strength of the soils in the upper layer in the bearing capacity analyses, since all of the soils in the upper layer will be effective in resisting failure along the anticipated bearing capacity slip surface.

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

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

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

Further evidence that this is a conservative value of s_u for the soils in the upper layer is presented in Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A). This plot of s_u vs confining pressure illustrates that this value is slightly less than the average value of s_u measured in the CU triaxial tests that were performed on specimens obtained from depths of ~10 ft at confining stresses of 2.1 ksf. As indicated in this figure, the confining stress of 2.1 ksf used to test these specimens is comparable to the vertical stress that will exist ~5 ft below the Canister Transfer Building mat following completion of construction. Since these tests were performed on specimens of the weakest soils underling the Canister Transfer Building mat (the deeper lying soils are stronger based on the SPT and the cone penetration test data), it is conservative to use the weighted average value of s_u of 3.18 ksf for the soils in the entire upper layer of the profile in the bearing capacity analyses.

Direct shear tests were performed on undisturbed specimens of the silty clay/clayey silt obtained from Borings CTB-6 and CTB-S, which were drilled in the locations shown in SAR Figure 2.6-18. These specimens were obtained from Elevation ~4469, the elevation of the bottom of the 1-ft deep perimeter key proposed at the base of Canister Transfer Building mat. Note, this key is being constructed around the perimeter of the mat to ensure that the full shear strength of the clayey soils is available to resist sliding of the structure due to loads from the design basis ground motion. These direct shear tests were performed at normal stresses that ranged from 0.25 ksf to 3.0 ksf. This range of normal stresses bounds the ranges of stresses expected for static and dynamic loadings from the design basis ground motion.

The results of these tests are presented in Attachments 7 and 8 of the Appendix 2A of the SAR and they are plotted in Figures 9 and 10 of Calc 05996.02-G(B)-05-2 (copies included in Attachment A). Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, sliding stability analyses included below of the Canister Transfer Building constructed directly on the silty clay are performed using the average shear strength measured in these direct shear tests for a normal stress equal to the vertical stress under the building following completion of construction, but prior to imposition of the dynamic loading due to the earthquake. As shown in Figures 9 and 10 of Calc 05996.02-G(B)-05-2 (copies included in Attachment A), this average shear strength is 1.8 ksf and the friction angle is set equal to 0°.

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

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

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

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

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

METHOD OF ANALYSIS

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

The following load combinations are analyzed:

Case I Static

Case II Static + dynamic horizontal forces due to the earthquake

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

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

For Case II, 100% of the dynamic lateral forces in both X and Z directions are combined. For Cases III and IV, 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the dynamic loading acts in the other two directions. For these cases, the suffix "A" is used to designate 40% in the X direction (N-S for the Canister Transfer Building, as shown in Figure 1), 100% in the Y direction (vertical), and 40% in the Z direction (E-W). Similarly, the suffix "B" is used to designate 40% in the X direction, 40% in the Y, and 100% in the Z, and the suffix "C" is used to designate 100% in the X direction and 40% in the other two directions. Thus,

Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.

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

Combining the effects of the three components of the design basis ground motion in this manner is in accordance with ASCE-4 (1986).

ANALYSIS OF OVERTURNING STABILITY

The factor of safety against overturning is defined as:

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

The overturning stability of the Canister Transfer Building is determined using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads are listed in Table 1 (SAR Table 2.6-11), and they were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (SWEC, 1999b) and described in SAR Section 4.7.1.5.3. The masses and accelerations of the joints (see Figure 2 for locations of the joints) used in the model of the Canister Transfer Building in Calculation 05996.02-SC-5 are listed on the left side of Table 1, and the resulting inertial forces and associated moments are listed on the right. Based on building geometry shown schematically in Figure 1 and the forces and moments shown in Table 1, overturning is more critical about the N-S axis (~265 ft) than about the E-W axis (~165 ft).

The resisting moment is calculated as the weight of the building x the distance from one edge of the mat to the center of the mat. The weight of the building is 72,988 K, as shown in Table 1. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of ~165 ft, or 82.5 ft. Therefore,

$\Sigma M_{Resisting}$ = 72,988 K x 82.5 ft = 6,021,510 ft-K.

The driving moments include the ΣM acting about the N-S axis, ΣM_X in Table 1, which is 2,513,041 ft-K, and the moment due to the uplift force ($\Sigma F_{V\,dyn}$ = 57,139 K) x ½ the width of the mat. The vertical force due to the earthquake can act upward or downward. However, when it acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the dynamic vertical force acts in the upward direction, tending to unload the mat.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions at the same time. The moments acting about the E-W axis do not contribute to overturning about the N-S axis; therefore,

$$\sum M_{Driving} = \sqrt{2{,}513{,}041^2 + (57{,}139\text{ K} \times 82.5\text{ ft})^2} = 5{,}341{,}991 \quad \text{ft} - \text{K}$$

and FS_{OT} = 6,021,510 ÷ 5,341,991 = 1.13 about the N-S axis.

Checking overturning about the E-W axis (~165 ft), the resisting moment is calculated as the weight of the building x the distance from one edge of the mat to the center of the mat. The weight of the building is 72,988 K, as shown in Table 1. For overturning about the E-W axis, the moment arm for the resisting moment equals ½ of ~265 ft, or 132.5 ft. Therefore,

$\Sigma M_{Resisting}$ = 72,988 K x 132.5 ft = 9,670,910 ft-K.

The driving moments include the ΣM acting about the E-W axis, ΣM_Y in Table 1, which is 1,961,325 ft-K, and the moment due to the uplift force ($\Sigma F_{V\,dyn}$ = 57,139 K) x ½ the length of the mat. The vertical force due to the earthquake can act upward or downward. However, when it acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the dynamic vertical force acts in the upward direction, tending to unload the mat.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions at the same time. The moments acting about the N-S axis do not contribute to overturning about the E-W axis; therefore,

$$\sum M_{Driving} = \sqrt{1{,}961{,}325^2 + (57{,}139\ K \times 132.5\ ft)^2} = 7{,}820{,}843 \quad ft-K$$

and $FS_{OT} = 9{,}670{,}910 \div 7{,}820{,}843 = 1.24$ about the E-W axis.

These values are greater than the criterion of 1.1; therefore, the Canister Transfer Building has an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion.

Analysis of Sliding Stability

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

$$FS = \text{Resisting Force} \div \text{Driving Force} = T \div V$$

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

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

where, N (normal force) $= \sum F_v = F_{v\,Static} + F_{v\,Eqk}$

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

c = 1.8 ksf, as discussed above under "Geotechnical Properties."

B = 165 feet

L = 265 feet

The driving force, V, is calculated as follows:

$$V = \sqrt{F^2_{HN-S} + F^2_{HE-W}}$$

Sliding Stability of the Canister Transfer Building on In Situ Clayey Soils

The sliding stability of the CTB was evaluated using the foundation loadings developed in the soil-structure interaction analyses (Calculation 05996.02-SC-5, SWEC, 1999b). In this case, the strength of the clayey soils at the bottom of the 1-ft deep key around the CTB mat was based on the average of the two sets of direct shear tests performed on samples of soils obtained from beneath the CTB at the elevation proposed for founding the structure. The results of these tests are included in Attachments 7 and 8 of Appendix 2A of the SAR. As discussed above under Geotechnical Properties, $\phi = 0°$ and a shear strength of 1.8 ksf were used for the clayey soils underlying the Canister Transfer Building in determining resisting forces for the earthquake loading combinations.

Conservatively assume the backfill to be placed around the Canister Transfer Building mat and 1-ft deep key will be the eolian silt that was excavated from the area. For these soils, it is reasonable to assume the lower bound value of γ is 80 pcf, $\phi = 30°$ & c = 0.

$$K_p = \left(\frac{1+\sin\phi}{1-\sin\phi}\right) = 3.0 \text{ for } \phi = 30°$$

For cohesionless soils, $P_p = 0.5 \times \gamma H^2 K_p$

$P_p = 0.5 \times 0.080 \text{ kcf} \times (6 \text{ ft})^2 \times 3.0 = 4.32 \text{ k/LF}$

Based on Drawing 0599602-EC-2-A (See Figure 5), the CTB mat is actually 35' + 145' + 35' = 215' wide in the E-W direction and 182' + 60' + 30' = 272' long in the N-S direction. Therefore, the total passive force available to resist sliding is at least 215' x 4.32 k/LF = 929 k acting in the N-S direction.

Lambe & Whitman (1969, p 165) indicates that little horizontal compression, ~0.5%, is required to reach half of full passive resistance for dense sands. The eolian silts will be compacted to a dense state; therefore, assume that half of the total passive resistance is available to resist sliding of the building. Note, 0.5% of the 6 ft height of the mat + 1-ft deep key = 0.005 x 6 ft x 12 in./ft = 0.36 in. Since there are no safety-related systems that would be severed or otherwise impacted by movements of this small magnitude, it is reasonable to use this passive thrust to resist sliding.

The results of the sliding stability analysis of the Canister Transfer Building are presented in Table 2, and they indicate that the factors of safety are acceptable for all load combinations examined. The lowest factor of safety was 1.10, which applies for Cases IIIB and IVB, where 100% of the dynamic earthquake forces act in the east-west direction and 40% act in the other two directions. These results assume that only one-half of the passive pressures are available to resist sliding and no credit is taken for the fact that the strength of cohesive soils increases as the rate of loading increases (Schimming et al, 1966, Casagrande and Shannon, 1948, and Das, 1993); therefore, they represent a conservative lower-bound value of the sliding stability.

Sliding Stability of the Canister Transfer Building on Cohesionless Soils

The Canister Transfer Building will be founded on clayey soils that have an adequate amount of cohesive strength to resist sliding due to the dynamic forces from the design basis ground motion. As shown in SAR Figures 2.6-21 through 2.6-23, however, some of the soils underlying the building are cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. Analyses presented on the next six pages address the possibility that sliding may occur along a deeper slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

The resistance to sliding is greatly reduced for frictional materials when the dynamic forces due to the earthquake act upward. The normal forces act downward for Case IV loadings and, hence, the resisting forces will be much greater than those for Case III.

Therefore, these analyses are performed only for Load Cases IIIA, IIIB, and IIIC. As described above, these load cases are defined as follows:

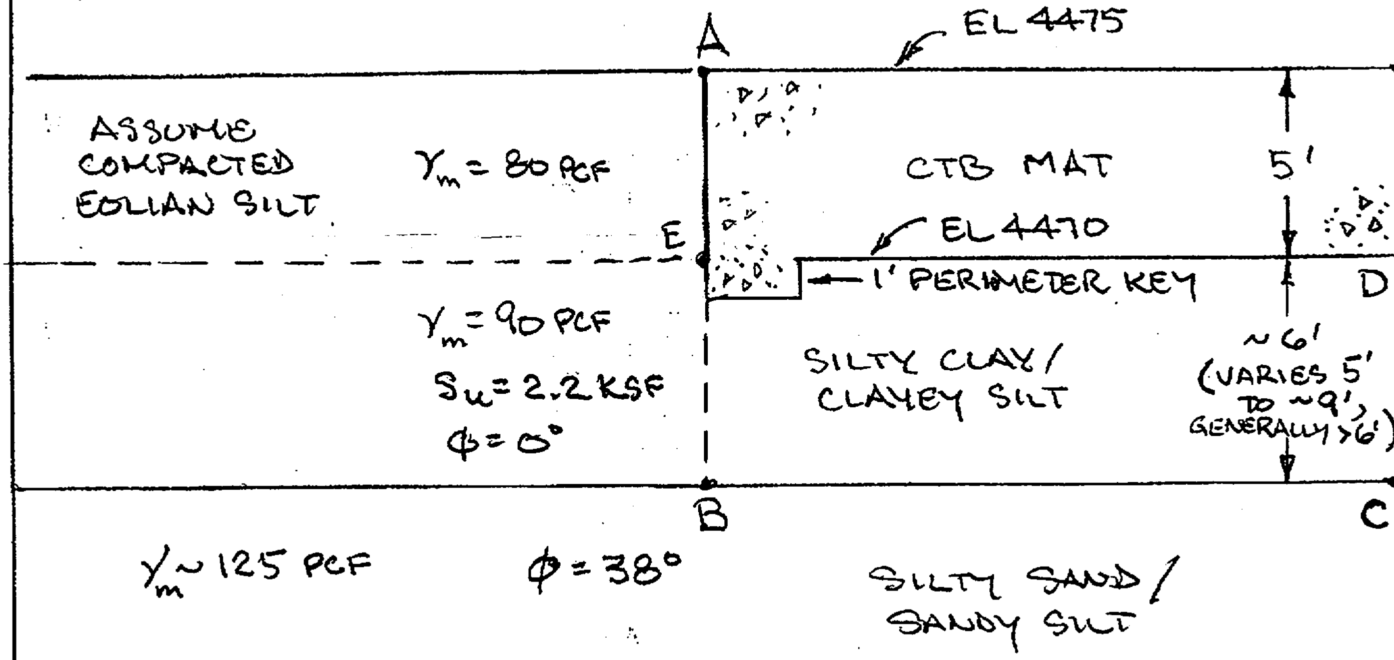
Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.

As shown in SAR Figures 2.6-21 through 2.6-23, the top of the cohesionless layer varies from about 5 ft to about 9 ft below the mat, and it generally is at a depth of about 6 ft below the mat. These analyses include the passive resistance acting on a plane extending from grade down to the top of the cohesionless layer, plus the shear strength available at the ends of the silty clay block under the mat, plus the frictional resistance available along the top of the cohesionless layer. The weight of the clayey soils existing between the top of the cohesionless soils and the bottom of the mat is included in the normal force used to calculate the frictional resistance acting along the top of the cohesionless layer.

A review of the cone penetration test results (ConeTec, 1999) obtained within the top 2 ft of the layer of nonplastic silt/silty sand/sandy silt underlying the Canister Transfer Building indicated that $\phi = 38°$ is a reasonable minimum value for these soils. This review is presented on the next page.

The next five pages illustrate that the factor of safety against sliding along the top of this layer is >1.1 for Load Cases IIIA and IIIC and they illustrate that it is ~1.1 for Load Case IIIB. These analyses include several conservative assumptions. They are based on static strengths of the silty clay block under the Canister Transfer Building mat, even though, as reported in Das (1993), experimental results indicate that the strength of cohesive soils increases as the rate of loading increases. For rates of strain applicable for the cyclic loading due to the design basis ground motion, Das indicates that for most practical cases, one can assume that $c_{u\ dynamic} \sim 1.5 \times c_{u\ static}$. In addition, the silty sand/sandy silt layer is not continuous under the Canister Transfer Building mat, and this analysis neglects cementation of these soils that was observed in the samples obtained in the borings. Therefore, sliding is not expected to occur along the surface of the cohesionless soils underlying the Canister Transfer Building.

SLIDING ON DEEP PLANE AT TOP OF SILTY SAND / SANDY SILT LAYER



NOTE: VALUE OF ϕ BASED ON ϕ DATA FROM CPT-37 & 38, PRESENTED IN CONETEC (1999)

ID	~DEPTH OF SILTY SAND	MIN ϕ	MAX ϕ	AVG ϕ	MEDIAN ϕ	ϕ IN TOP 2'
CPT-37	~11.6' TO ~18.7'	36*	44	40	40	~38
CPT-38	~11' TO ~18'	38	46	43	44	~38

PASSIVE PRESSURES ACTING ON PLANE AB WILL INCREASE AS B GETS DEEPER IN THE SILTY SAND / SANDY SILT LAYER; ∴ USE ϕ NEAR THE TOP OF THE LAYER. ⇒ $\phi = 38°$.

N VALUES ARE HIGH, GENERALLY ≫ 20 BL/FT; ∴ $\phi = 38°$ IS REASONABLE

* EXCLUDING SINGLE VALUE OF $\phi = 34°$ AT $Z = 13.8'$

SLIDING ON DEEP COHESIONLESS PLANE

$$FS_{SLIDING} = \frac{\Sigma \text{ RESISTING FORCES}}{\Sigma \text{ DRIVING FORCES}}$$

RESISTING FORCES INCLUDE: PASSIVE RESISTANCE AVAILABLE ALONG AB + SHEAR RESISTANCE ALONG ENDS OF BLOCK BCDE + FRICTION ALONG BC.

① PASSIVE RESISTANCE AVAILABLE ALONG AB

INCLUDES $\frac{1}{2} \times (0.080 \frac{K}{FT^3}) \times (6')^2 \times 3.0 = 4.32$ K/LF FOR COMPACTED EOLIAN SILT ADJACENT TO 5' MAT + 1' KEY

$+ \frac{1}{2} \gamma H^2 K_p + q_s H K_p + 2cH\sqrt{K_p}$ FOR 5' BLOCK OF SILTY CLAY UNDERLYING THE COMPACTED SILT.

$$\frac{1}{2}\left(0.090 \frac{K}{FT^3}\right) \times (5\,FT)^2 \times \overset{K_p}{1.0} + \overset{H}{6\,FT} \times \overset{\gamma}{0.080 \frac{K}{FT^3}} \times 5\,FT \times \overset{K_p}{1.0}$$

$$+ 2 \times \overset{c}{2.2} \frac{K}{FT^2} \times 5\,FT \times \sqrt{1.0} = 1.125 + 2.40 + 22.0 = 25.52 \frac{K}{FT}$$

∴ TOTAL PASSIVE RESISTANCE AVAILABLE ALONG AB

$= 4.32 + 25.52 = 29.84$ K/LF

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. 13-3	OPTIONAL TASK CODE	PAGE 17

(2) ESTIMATE ADDITIONAL RESISTANCE TO SLIDING AVAILABLE AT THE ENDS OF THE BLOCK OF SILTY CLAY THAT MUST SHEARED BEFORE THE CTB CAN SLIDE. INCLUDE ONLY THE PORTION BELOW THE CTB MAT; i.e., BCDE SHOWN ON PAGE 15.

$S_u = 2.2$ KSF = MINIMUM S_u MEASURED IN UU TRIAXIAL TESTS AT $\sigma_c = 1.3$ KSF

$$\text{AREA BCDE} = 6\text{ FT} \times 215\text{ FT}_{E\text{-}W} = 1290 \frac{\text{FT}^2}{\text{END}}$$

$$\therefore \Delta T_{\text{ENDS}_{E\text{-}W}} = 2\text{ ENDS} \times 1290 \frac{\text{FT}^2}{\text{END}} \times 2.2 \frac{\text{K}}{\text{FT}^2} = 5{,}676\text{ K}_{E\text{-}W}$$

$$\Delta T_{\text{END}_{N\text{-}S}} = 2\text{ ENDS} \times 6' \times \underset{L_{N\text{-}S}}{272'} \times 2.2 \frac{\text{K}}{\text{FT}^2} = 7{,}181\text{ K}_{N\text{-}S}$$

(3) FRICTIONAL RESISTANCE ALONG PLANE BC:

ADD WEIGHT OF SILTY CLAY BLOCK BETWEEN BOTTOM OF MAT & TOP OF SILTY SAND / SANDY SILT TO THE NORMAL FORCE AT BOTTOM OF THE MAT.

$$\Delta N_{\text{CLAY}} = \overset{\Delta H}{6'} \times \overset{\gamma}{0.090 \frac{\text{K}}{\text{FT}^3}} \times \overset{B}{165'} \times \overset{L}{265'} = 23{,}612\text{ K}$$

NOTE: FRICTIONAL RESISTANCE WILL BE LOWER WHEN VERT EARTHQUAKE FORCES ACT UPWARD — ∴ CHECK CASES III A, B, & C.

NOTED JAN 2 1 2000 P. J. Trudeau

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 18
3				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3		

SLIDING ON DEEP PLANES

	N-S	VERT	E-W
CASE IIIA:	40% IN X	−100% IN Y	40% IN Z
FROM TABLE 1	0.4×62,040^K	−57,139 K	$\underbrace{0.4\times 67{,}572}_{L = V_{E\text{-}W}} = 27{,}029$ K

$$\therefore N = \underbrace{72{,}988}_{\text{CTB DL}} - \underbrace{57{,}139\text{K}}_{F_{VD}\text{ or }EQ_V} + \underbrace{23{,}612^{\text{K}}}_{\Delta N_{CLAY}} = 39{,}460\text{ K}$$

$$N\tan\phi = 39{,}460\text{K} \tan 38^\circ = 30{,}830\text{ K}$$

$$\therefore FS_{SLIDING_{N\text{-}S}} = \frac{29.84\,\text{K}/\text{LF} \times \underbrace{215'}_{B_{E\text{-}W}} + 7{,}181\text{K} + 30{,}830\text{K}}{0.4\times 62{,}040\text{K}} = 1.78$$

$$FS_{SLIDING_{E\text{-}W}} = \frac{29.84\frac{\text{K}}{\text{LF}}\times \overset{L}{272'} + 5{,}676\text{K} + 30{,}830\text{K}}{0.4\times 67{,}572\text{ K}} = 1.65 > 1.1 \quad \therefore \text{OK}$$

	N-S	VERT	E-W
CASE IIIB	40% IN X	−40% IN Y	100% IN Z
FROM TABLE 1	0.4×62,040^K	−0.4×57,139^K	67,572^K $L = V_{E\text{-}W}$

$$\therefore N = \underbrace{72{,}988\text{K}}_{\text{CTB DL}} - \underbrace{0.4\times 57{,}139\text{K}}_{F_{VD}} + \underbrace{23{,}612^{\text{K}}}_{\Delta N_{CLAY}} = 73{,}744\text{ K}$$

22,856

$$\Rightarrow T = \Big(\underbrace{4.32\frac{\text{K}}{\text{L.F.}}}_{P_P} + 25.52\frac{\text{K}}{\text{LF.}}\Big)\times \overset{L_{N\text{-}S}}{272'} + 5{,}676\text{ K}_{E\text{-}W}$$

8116 K

$$+ \overset{N}{73{,}744\text{K}} \tan \overset{\phi}{38^\circ} = 71{,}407\text{ K}$$

57,615

$$FS = \frac{\text{RESISTING}}{\text{DRIVING}} = \frac{71{,}407\text{ K}}{67{,}572\text{ K}} = 1.06 \quad \sim 1.1$$

4 CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 19
05996.02	G(B)	13-3		

SLIDING ON DEEP PLANE

	N-S	VERT	E-W
CASE IIIC	100% IN X	-40% IN Y	40% IN Z
FROM TABLE 1	62,040 K	$-0.4 \times 57{,}139^k$	$0.4 \times 67{,}572$

$$\therefore N = \underbrace{72{,}988}_{CTB\ DL} - \underbrace{0.4 \times 57{,}139\,K}_{F_{VD}} + \underbrace{23{,}612}_{\Delta N_{CLAY}} = 73{,}744\,K$$

$$22{,}856$$

$$T_{N\text{-}S} = 29.84\,\frac{K}{LF} \times \underbrace{215'}_{B} + \underbrace{7{,}181\,K}_{\Delta T_{N\text{-}S}} + 73{,}744 \tan 38^\circ = 71{,}212\,K$$

$$FS_{SLIDING} = \frac{T}{V_{N\text{-}S}} = \frac{71{,}212\,K}{62{,}040\,K} = 1.15 \quad > 1.1 \therefore OK$$

THE FACTOR OF SAFETY AGAINST SLIDING ON A DEEP PLANE OF COHESIONLESS SOIL IS > 1.1 FOR LOAD CASES III A & III C, AND IT IS ESSENTIALLY 1.1 FOR LOAD CASE III B. THIS CASE, HOWEVER, IS LESS CRITICAL THAN THE CASE PRESENTED IN THE FOLLOWING SECTION, WHICH DEMONSTRATES THAT EVEN IF THE COHESIONLESS SOIL WAS LOCATED DIRECTLY BENEATH THE CTB MAT, THE ESTIMATED HORIZONTAL DISPLACEMENTS OF THE FOUNDATION DUE LOADS FROM THE DESIGN BASIS GROUND MOTION ARE ≤ 1.2 INCHES.

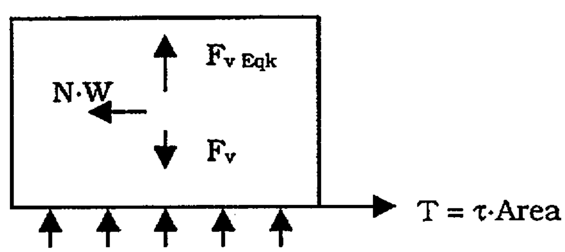
SLIDING STABILITY OF THE CANISTER TRANSFER BUILDING ON COHESIONLESS SOILS (CONT'D)

An additional analysis of sliding on cohesionless soils was performed to define the upper bound of potential movement that might occur due to the earthquake if the mat was founded directly on cohesionless soils. In this analysis it was postulated that the cohesionless soils extend above the depth of about 10 ft and the structure is founded directly on the cohesionless materials. These analyses conservatively assumed that $\phi = 35°$ and $c = 0$ for these soils.

The higher value of ϕ used here, compared to that used in the cask storage pad sliding analysis, is based on the fact that the cohesionless soils underlying the Canister Transfer Building area are sandier than those in the pad emplacement area. Further, this higher value is justified by the results of the cone penetration testing, which indicate that the average and median ϕ range from 40° to 44° for the cohesionless soils underlying the Canister Transfer Building. The high values reported in the CPT results likely are the manifestation of cementation that was observed in many of the specimens obtained in split-barrel sampling. Possible cementation of these soils is also ignored in this analysis, adding to the conservatism.

Because of the magnitude of the dynamic forces resulting from the soil-structure interaction analyses, the factor of safety against sliding of this building would be less than 1 if it were founded directly on cohesionless soils. For this case, the displacements the building may experience were calculated using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes.

NEWMARK'S METHOD OF ESTIMATING DISPLACEMENTS DUE TO EARTHQUAKES



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.

For a block sliding on a horizontal surface, N·W = T,

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

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

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

$\sigma_n =$ Normal Stress

$\phi =$ Friction angle of sand layer

$\sigma_n =$ Net Vertical Force/Area

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

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

$N \cdot W = T$

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

The maximum relative displacement of the mat 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 the data points for N/A less than 0.15 and greater than 0.5, as shown in Figure 6 , 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.

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

ESTIMATION OF HORIZONTAL DISPLACEMENT USING NEWMARK'S METHOD

1. Maximum Ground Motions

The maximum ground accelerations and velocities at the Canister Transfer Building are based on Calculation 05996.02-SC-5, Rev. 1, p. 37 (SWEC, 1999b), which indicates:

	North-South	Vertical	East-West
Acceleration	0.805g	0.720g	0.769g
Velocity	21.7 in./sec	Not Required	19.8 in./sec

2. Load Combinations

The resistance to sliding is greatly reduced for frictional materials when the dynamic forces due to the earthquake act upward. The normal forces act downward for Case IV loadings and, hence, the resisting forces will be much greater than those for Case III. Therefore, these analyses are performed only for Load Cases IIIA, IIIB, and IIIC. As described above, these load cases are defined as follows:

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

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

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

3. Ground Motions for Analysis

Case	North-South		Vertical	East-West	
	Accel (g)	Velocity (in./sec)	Accel (g)	Accel (g)	Velocity (in./sec)
IIIA	0.322	8.68	0.720	0.308	7.92
IIIB	0.322	8.68	0.288	0.769	19.8
IIIC	0.805	21.7	0.288	0.308	7.92

LOAD CASE IIIA: 40% N-S DIRECTION, -100% VERTICAL DIRECTION, 40% E-W DIRECTION.

Static Vertical Force, F_v = W = 72,988 kips (Calculation 05996.02-SC-5, Rev 1 (SWEC, 1999b), p37)

Earthquake Vertical Force, $F_{v\,Eqk}$ = 57,139 kips (Calculation 05996.02-SC-5, Rev 1, p37)

$$\phi = 35°$$

$$N = [(72{,}988 - 57{,}139) \tan 35°] / 72{,}988$$

$$N = 0.152$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\overset{40\%\ N\text{-}S}{0.322^2} + \overset{40\%\ E\text{-}W}{0.308^2})}\,g$

$$A = 0.446g$$

Resultant velocity in horizontal direction, $V = \sqrt{(\overset{40\%\ N\text{-}S}{8.68^2} + \overset{40\%\ E\text{-}W}{7.92^2})}$

$$V = 11.75 \text{ in./sec}$$

$$\Rightarrow \frac{N}{A} = \frac{0.152}{0.446} = 0.34$$

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

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

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

$$\Rightarrow \quad u_m = \left(\frac{(11.75\,\text{in./sec})^2 \cdot (1-0.34)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.152}\right) = 0.8''$$

As shown in Figure 6, 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 where there is symmetrical resistance to sliding. Within the range of values of N/A between 0.15 to 0.5, the following expression gives an upper bound for all data:

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

Substituting the relevant parameters,

$$\Rightarrow \quad u_m = \left(\frac{(11.75\,\text{in./sec})^2}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.152}\right) = 1.2''$$

Therefore, the maximum relative displacement ranges from 0.8" to 1.2" for Load Case IIIA.

LOAD CASE IIIB: 40% N-S DIRECTION, -40% VERTICAL DIRECTION, 100% E-W DIRECTION.

Static Vertical Force, F_v = W = 72,988 kips (Calculation 05996.02-SC-5, Rev 1 (SWEC, 1999b), p37)

Earthquake Vertical Force, $F_{v\,Eqk}$ = 57,139 kips x 0.40 = 22,856 kips acting upward.

$$\phi = 35^\circ$$

$$N = [(72{,}988 - 22{,}856) \tan 35^\circ] / 72{,}988$$

$$N = 0.48$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\overset{40\%\ \text{N-S}}{0.322^2} + \overset{100\%\ \text{E-W}}{0.769^2})}\,g$

$$A = 0.834g$$

Resultant velocity in horizontal direction, $V = \sqrt{(\overset{40\%\ \text{N-S}}{8.68^2} + \overset{100\%\ \text{E-W}}{19.8^2})}$

$$V = 21.6\ \text{in./sec}$$

$$\Rightarrow \quad \frac{N}{A} = \frac{0.48}{0.834} = 0.576$$

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

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

where g is in units of inches/sec².

$$\Rightarrow \quad u_m = \left(\frac{(21.6\,\text{in./sec})^2 \cdot (1-0.576)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.48}\right) = 0.5"$$

As shown in Figure 6, 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 where there is symmetrical resistance to sliding. For Case IIIB, N/A > 0.5; therefore, u_m = 0.5" for this case.

LOAD CASE IIIC: 100% N-S DIRECTION, -40% VERTICAL DIRECTION, 40% E-W DIRECTION.

Static Vertical Force, F_v = W = 72,988 kips (Calculation 05996.02-SC-5, Rev 1 (SWEC, 1999b), p37)

Earthquake Vertical Force, $F_{v\,Eqk}$ = 57,139 kips x 0.40 = 22,856 kips acting upward.

$$\phi = 35°$$

$$N = [(72{,}988 - 22{,}856)\tan 35°] / 72{,}988$$

$$N = 0.48$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\overset{100\%\ N\text{-}S}{0.805^2} + \overset{40\%\ E\text{-}W}{0.308^2})}\, g$

$$A = 0.862g$$

Resultant velocity in horizontal direction, $V = \sqrt{(\overset{100\%\ N\text{-}S}{21.7^2} + \overset{40\%\ E\text{-}W}{7.92^2})}$

$$V = 23.1 \text{ in./sec}$$

$$\Rightarrow \quad \frac{N}{A} = \frac{0.48}{0.862} = 0.558$$

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

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

where g is in units of inches/sec².

$$\Rightarrow \quad u_m = \left(\frac{(23.1\,\text{in./sec})^2 \cdot (1-0.558)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.48}\right) = 0.6"$$

As shown in Figure 6, 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 where there is symmetrical resistance to sliding. For Case IIIC, N/A > 0.5; therefore, u_m = 0.6" for this case.

SUMMARY OF HORIZONTAL DISPLACEMENT CALCULATED USING NEWMARK'S METHOD

The following table presents a summary of the Newmark's analysis of sliding of the Canister Transfer Building, assuming it is founded directly on cohesionless soils.

Load Combination				**Displacement**
Case IIIA	40% N-S	-100% Vertical	40% E-W	0.8 to 1.2 inches
Case IIIB	40% N-S	-40% Vertical	100% E-W	0.5 inches
Case IIIC	100% N-S	-40% Vertical	40% E-W	0.6 inches

These analyses indicate that there is an adequate factor of safety against sliding along the surface of the soils underlying the building that may be cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. The analysis that postulated that these cohesionless soils exist higher in the profile, such that the building was constructed directly on them, includes several conservative assumptions. Even with this high level of conservatism, the estimated relative displacement of the building ranged from 0.5 inches to 1.2 inches.

Motions of this magnitude, occurring at the depth of the silty sand/sandy silt layer, would likely not even be evident at the ground surface. For the building to slide, a surface of sliding must be established between the horizontal sliding surface in the silty sand/sandy silt layer and through the overlying clayey layer. In the simplified model used to estimate these displacements, the contribution of this surface of sliding through the overlying clayey layer to the dynamic resistance to sliding motion is ignored, as is the passive resistance that would act on the embedded portion of the building foundation and the block of soil that is postulated to be moving with it. It is likely, that should such slippage occur within the cohesionless soils underlying the building, it would minimize the level of the accelerations that would be transmitted through the soil and into the structure. In this manner, the cohesionless soils would act as a built-in base-shear isolation system. Any decrease in these accelerations as a result of this would increase the factor of safety against sliding, which would decrease the estimated displacements as well. Further, since there are no Important to Safety systems that would be severed or otherwise impacted by movements of this small amount as a result of the earthquake, such movements do not adversely affect the performance of the Canister Transfer Building.

ALLOWABLE BEARING CAPACITY

Bearing capacity calculations are performed using the method for determining general bearing capacity failure, as presented in Winterkorn and Fang (1975. Local bearing capacity (punching shear) failure is ruled out due to the large size of the mat, 165' x 265'.

The general bearing capacity equation is a modification of Terzaghi's bearing capacity equation, which was developed for strip footings and which indicates that $q_{ult} = cN_c+qN_q+1/2\ \gamma BN_\gamma$. For this relationship, the ultimate bearing capacity of soil consists of three components: 1) cohesion, 2) surcharge, and 3) friction, which are represented by bearing capacity factors N_c, N_q, and N_γ. Terzaghi's bearing capacity equation has been enhanced by various investigators to incorporate shape, depth, and load inclination factors for different foundation geometries and loads as follows:

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

where

q_{ult} = ultimate bearing capacity

c = cohesion or undrained strength

q = effective surcharge at bottom of foundation, = γD_f

γ = unit weight of soil

B = foundation width

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

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

i_c, i_q, i_γ = load inclination factors

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

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

BEARING CAPACITY FACTORS

Bearing capacity factors computed based on relationships proposed by Vesic (1973), which are presented in Chapter 3 of Winterkorn and Fang (1975).

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

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

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

SHAPE FACTORS

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

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

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

DEPTH FACTORS

For $\dfrac{D_f}{B} \leq 1$:

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

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

$$d_\gamma = 1$$

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.

STATIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

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

Case IA Static using undrained strength parameters ($\phi = 0°$ & $c = 3.18$ ksf).

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

Table 2.6-9 presents the results of the bearing capacity analyses for these static load cases. The minimum factor of safety required for static load cases is 3.

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 6 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and $c = 3.18$ ksf, the average undrained strength for the soils in the upper layer at the site, to model the end of construction. Using the estimated effective-stress strength of $\phi = 30°$ and $c = 0$ results in higher allowable bearing pressures. As shown in Table 2.6-9, the gross allowable bearing capacity of the Canister Transfer Building for static loads for these soil strengths is 45 ksf.

CALCULATION IDENTIFICATION NUMBER				PAGE 29
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Static Analysis: **Case IA**

Soil Properties:

$s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer

$\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **165.0** Footing Width - ft (E-W) $L' =$ **265.0** Length - ft (N-S)

$D_f =$ **5** Depth of Footing (ft)

$\beta =$ **0.0** Angle of load inclination from vertical (degrees)

$FS =$ **3** Factor of Safety required for $q_{allowable}$.

$F_V =$ **72,988** k $EQ_V =$ **0** k

$EQ_{H\ E\text{-}W} =$ **0** k + $EQ_{H\ N\text{-}S} =$ **0** k = **0** k for F_H

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

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

$N_c = (N_q - 1)\ \cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\ (N_q + 1)\ \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.12	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\ (B/L)$	=	0.75	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\ D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1 - d_q) / (N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\ (D_f/B)$	=	1.01	Eq 3.27

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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**18,947**	**psf =**	**18,547**	+	**400**	+	**0**

q_{all} = 6,310 **psf = q_{ult} / FS**

q_{actual} = 1,669 **psf = $(F_v + EQ_v) / (B' \times L')$**

FS_{actual} = 11.35 **= q_{ult} / q_{actual}** **> 3 Hence OK**

CALCULATION IDENTIFICATION NUMBER				PAGE 30
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Static Analysis: **Case IB**

Soil Properties:

$s_u =$ **0** Cohesion (psf)

$\phi =$ **30.0** Friction Angle (degrees)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **165.0** Footing Width - ft (E-W) L' = **265.0** Length - ft (N-S)

$D_f =$ **5** Depth of Footing (ft)

$\beta =$ **0.0** Angle of load inclination from vertical (degrees)

$FS =$ **3** Factor of Safety required for $q_{allowable}$.

$F_V =$ **72,988** k $EQ_V =$ **0** k

$EQ_{H\ E\text{-}W} =$ **0** k + $EQ_{H\ N\text{-}S} =$ **0** k = **0** k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	30.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	18.40	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	22.40	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.38	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.36	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.75	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.01	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	1.01	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	N/A	Eq 3.27

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

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**135,005**	**psf =**	**0**	**+**	**10,094**	**+**	**124,911**

q_{all} = 45,000 psf = q_{ult} / FS

q_{actual} = 1,669 psf = $(F_v + EQ_v) / (B' \times L')$

FS_{actual} = 80.88 = q_{ult} / q_{actual} > **3** **Hence OK**

DYNAMIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

The following pages present the details of the bearing capacity analyses for the dynamic load cases. These analyses use the dynamic loads for the building that were developed in Calculation 05996.02-SC-5, (SWEC, 1999b). The development of these dynamic loads is described in Section 4.7.1.5.3 of the SAR. As in the structural analyses discussed in SAR Section 4.7.1.5.3., the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions. The resulting dynamic loading cases are identified as follows:

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

Table 2.6-10 presents the results of the bearing capacity analyses for these cases, which include static loads plus dynamic loads due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the average undrained strength applicable for the soils within the upper layer ($\phi = 0°$ and c = 3.18 ksf). As indicated above, for these cases including dynamic loads from the design basis ground motion, the minimum acceptable factor of safety is 1.1.

Table 2.6-10 indicates the minimum factor of safety against a dynamic bearing capacity failure was obtained for Load Case IIIB, the load combination of full static with 40% of the earthquake loading acting in the N-S direction, 40% acting in the vertical direction, tending to unload the mat, and 100% acting in the E-W horizontal direction. This load case resulted in an actual soil bearing pressure of 3.31 kips per square foot (ksf), compared with an ultimate bearing capacity of ~9 ksf. The resulting factor of safety against a bearing capacity failure for this load case is ~3, which is much greater than 1.1, the minimum allowable factor of safety for seismic loading cases. In these analyses, no credit was taken for the fact that strength of cohesive soil increases as the rate of loading increases. Therefore, the Canister Transfer Building has an adequate factor of safety against a dynamic bearing capacity failure.

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case II — **100 % in X, 0 % in Y, 100 % in Z**

Soil Properties:	s_u =	**3,180**	Average undrained strength (psf) in upper ~30' layer
	ϕ =	**0.0**	Friction Angle (degrees)
	γ =	**90**	Unit weight of soil (pcf)
	γ_{surch} =	**80**	Unit weight of surcharge (pcf)
Foundation Properties:	B' =	**96.1**	Footing Width - ft (E-W) L' = **211.3** Length - ft (N-S)
	D_f =	**5**	Depth of Footing (ft)
	β =	**42.8**	Angle of load inclination from vertical (degrees)
	FS =	**1.1**	Factor of Safety required for $q_{allowable}$.
	F_V =	**72,988**	k EQ_V = 0 k
	$EQ_{H\ E\text{-}W}$ =	**67,572**	k + $EQ_{H\ N\text{-}S}$ = 62,040 k = **91,733** k for F_H

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

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

$N_c = (N_q - 1)\ \cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\ (N_q + 1)\ \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\ (B/L)$	=	0.82	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\ D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q) / (N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\ (D_f/B)$	=	1.02	Eq 3.27
$m_B = (2 + B/L) / (1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B) / (1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	=	0.83	rad
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.51	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\ F_H / B'\ L'\ c\ N_c)$	=	0.58	Eq 3.16a
$i_q = \{ 1 - F_H / [(F_v + EQ_v) + B'\ L'\ c \cot\phi] \}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{ 1 - F_H / [(F_v + EQ_v) + B'\ L'\ c \cot\phi] \}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	10,984	psf =	10,584	+	400	+	0

q_{all} = **9,980** psf = q_{ult} / FS

q_{actual} = **3,594** psf = $(F_v + EQ_v) / (B' \times L')$

FS_{actual} = **3.06** = q_{ult} / q_{actual} > **1.1 Hence OK**

CALCULATION IDENTIFICATION NUMBER				PAGE 33
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IIIA — **40 % in X, -100 % in Y, 40 % in Z**

Soil Properties:

s_u = **3,180** Average undrained strength (psf) in upper ~30' layer
ϕ = **0.0** Friction Angle (degrees)
γ = **90** Unit weight of soil (pcf)
γ_{surch} = **80** Unit weight of surcharge (pcf)

Foundation Properties:

B' = **38.2** Footing Width - ft (E-W) L' = **166.0** Length - ft (N-S)
D_f = **5** Depth of Footing (ft)
β = **59.6** Angle of load inclination from vertical (degrees)
FS = **1.1** Factor of Safety required for $q_{allowable}$.
F_V = **72,988** k EQ_V = **-57,139** k
$EQ_{H\,E\text{-}W}$ = **27,029** k + $EQ_{H\,N\text{-}S}$ = **24,816** k = **36,693** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.04	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	1.00	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1-\sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.83 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.51	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.46	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**8,753**	**psf =**	**8,353**	+	**400**	+	**0**

q_{all} = 7,950 psf = q_{ult} / FS

q_{actual} = 2,503 psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = 3.50 = q_{ult}/q_{actual} > 1.1 Hence OK

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CALCULATION IDENTIFICATION NUMBER				PAGE 34
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IIIB **40 % in X, -40 % in Y, 100 % in Z**

Soil Properties:

$s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer

$\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **64.7** Footing Width - ft (E-W) $L' =$ **233.7** Length - ft (N-S)

$D_f =$ **5** Depth of Footing (ft)

$\beta =$ **53.4** Angle of load inclination from vertical (degrees)

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

$F_V =$ **72,988** k $EQ_V =$ **-22,856** k

$EQ_{H\ E\text{-}W} =$ **67,572** k + $EQ_{H\ N\text{-}S} =$ **24,816** k = **71,985** 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 $\phi = 0$: $d_c = 1 + 0.4\ (D_f/B)$	=	1.03	Eq 3.27
$m_B = (2 + B/L) / (1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B) / (1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	=	1.22 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.59	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\ F_H / B'\ L'\ c\ N_c)$	=	0.54	Eq 3.16a
$i_q = \{ 1 - F_H / [(F_v + EQ_v) + B'\ L'\ c \cot\phi] \}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{ 1 - F_H / [(F_v + EQ_v) + B'\ L'\ c \cot\phi] \}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**9,947**	**psf =**	**9,547**	**+**	**400**	**+**	**0**

q_{all} = **9,040** **psf = q_{ult} / FS**

q_{actual} = **3,313** **psf = $(F_v + EQ_v) / (B' \times L')$**

FS_{actual} = **3.00** **= q_{ult} / q_{actual}** **> 1.1 Hence OK**

CALCULATION IDENTIFICATION NUMBER				PAGE 35
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IIIC — **100 % in X, -40 % in Y, 40 % in Z**

Soil Properties:
- s_u = **3,180** Average undrained strength (psf) in upper ~30' layer
- ϕ = **0.0** Friction Angle (degrees)
- γ = **90** Unit weight of soil (pcf)
- γ_{surch} = **80** Unit weight of surcharge (pcf)

Foundation Properties:
- B' = **124.9** Footing Width - ft (E-W) L' = **186.8** Length - ft (N-S)
- D_f = **5** Depth of Footing (ft)
- β = **28.3** Angle of load inclination from vertical (degrees)
- FS = **1.1** Factor of Safety required for $q_{allowable}$.
- F_V = **72,988** k EQ_V = **-22,856** k
- $EQ_{H\,E\text{-}W}$ = **27,029** k + $EQ_{H\,N\text{-}S}$ = **62,040** k = **67,672** k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.13	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.73	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.02	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.41 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.42	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.75	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**14,435**	**psf =**	**14,035**	+	**400**	+	**0**

q_{all} = 13,120 psf = q_{ult} / FS

q_{actual} = 2,149 psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = 6.72 = q_{ult}/q_{actual} **> 1.1 Hence OK**

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IVA — **40 % in X, 100 % in Y, 40 % in Z**

Soil Properties:

$s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer

$\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **149.6** Footing Width - ft (E-W) $L' =$ **252.9** Length - ft (N-S)

$D_f =$ **5** Depth of Footing (ft)

$\beta =$ **11.7** Angle of load inclination from vertical (degrees)

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

$F_V =$ **72,988** k $EQ_V =$ **57,139** k

$EQ_{H\,E\text{-}W} =$ **27,029** k + $EQ_{H\,N\text{-}S} =$ **24,816** k = **36,693** 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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.12	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.76	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.83 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.51	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.91	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**17,214**	**psf =**	**16,814**	+	**400**	+	**0**

q_{all} = 15,640 psf = q_{ult} / FS

q_{actual} = 3,440 psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = 5.00 = q_{ult}/q_{actual} > **1.1 Hence OK**

CALCULATION IDENTIFICATION NUMBER				PAGE 37
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IVB — $\boxed{\text{40 \% in X, 40 \% in Y, 100 \% in Z}}$

Soil Properties:	s_u =	**3,180**	Average undrained strength (psf) in upper ~30' layer
	ϕ =	**0.0**	Friction Angle (degrees)
	γ =	**90**	Unit weight of soil (pcf)
	γ_{surch} =	**80**	Unit weight of surcharge (pcf)
Foundation Properties:	B' =	**112.6**	Footing Width - ft (E-W) L' = **248.6** Length - ft (N-S)
	D_f =	**5**	Depth of Footing (ft)
	β =	**35.2**	Angle of load inclination from vertical (degrees)
	FS =	**1.1**	Factor of Safety required for $q_{allowable}$.
	F_V =	**72,988**	k EQ$_V$ = **22,856** k
	$EQ_{H\ E\text{-}W}$ =	**67,572**	k + $EQ_{H\ N\text{-}S}$ = **24,816** k = **71,985** k for F_H

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

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

	$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
	$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
	$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
	$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
	$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
	$s_\gamma = 1 - 0.4(B/L)$	=	0.82	"
For $D_f/B \leq 1$:	$d_q = 1 + 2\tan\phi\,(1-\sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
	$d_\gamma = 1$	=	1.00	"
For $\phi > 0$:	$d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$:	$d_c = 1 + 0.4(D_f/B)$	=	1.02	Eq 3.27
	$m_B = (2 + B/L)/(1 + B/L)$	=	1.62	Eq 3.18a
	$m_L = (2 + L/B)/(1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$:	$\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	1.22	rad
	$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.59	Eq 3.18c
For $\phi = 0$:	$i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.75	Eq 3.16a
	$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
	$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**13,976**	**psf =**	**13,576**	+	**400**	+	**0**

q_{all} = 12,700 psf = q_{ult} / FS

q_{actual} = 3,425 psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = 4.08 = q_{ult}/q_{actual} **> 1.1 Hence OK**

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CALCULATION IDENTIFICATION NUMBER				PAGE 38
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IVC **100 % in X, 40 % in Y, 40 % in Z**

Soil Properties: s_u = **3,180** Average undrained strength (psf) in upper ~30' layer
ϕ = **0.0** Friction Angle (degrees)
γ = **90** Unit weight of soil (pcf)
γ_{surch} = **80** Unit weight of surcharge (pcf)

Foundation Properties: B' = **144.0** Footing Width - ft (E-W) L' = **224.1** Length - ft (N-S)
D_f = **5** Depth of Footing (ft)
β = **15.7** Angle of load inclination from vertical (degrees)
FS = **1.1** Factor of Safety required for $q_{allowable}$.
F_V = **72,988** k EQ_V = **22,856** k
$EQ_{H\,E\text{-}W}$ = **27,029** k + $EQ_{H\,N\text{-}S}$ = **62,040** k = **67,672** k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.13	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.74	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.41 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.42	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.82	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	15,646	psf =	15,246	+	400	+	0

q_{all} = **14,220** psf = q_{ult} / FS

q_{actual} = **2,970** psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = **5.27** = q_{ult}/q_{actual} > **1.1 Hence OK**

CONCLUSIONS

OVERTURNING STABILITY OF THE CANISTER TRANSFER BUILDING

The overturning stability of the Canister Transfer Building is analyzed on Pages 8 & 9 using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads, listed in Table 1 (SAR Table 2.6-11), were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (SWEC, 1999b) and are described in SAR Section 4.7.1.5.3. This calculation demonstrates that the factor of safety against overturning of the Canister transfer Building is > 1.1; therefore, the Canister Transfer Building has an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion.

SLIDING SLIDING STABILITY OF THE CANISTER TRANSFER BUILDING

The Canister Transfer Building (CTB) will be founded on clayey soils. The sliding stability of the CTB was evaluated using the loads developed in Calculation 05996.02-SC-5 (SWEC, 1999b). The static strength of the clayey soils at the bottom of the CTB mat was based on the average of two sets of direct shear tests performed on samples of soils obtained from beneath the Canister Transfer Building at the elevation proposed for founding the mat.

The results of the sliding stability analysis are presented in Table 2 of this calculation, and they indicate that for all load combinations examined, the factors of safety were acceptable. The lowest factor of safety was 1.10, which applies for Cases IIIB and IVB, where 100% of the dynamic earthquake forces act in the east-west direction and 40% act in the other two directions. These results assume that only one-half of the passive pressures are available resist sliding and no credit is taken for the fact that the strength of cohesive soils increases as the rate of loading increases (Schimming et al, 1966, Casagrande and Shannon, 1948, and Das, 1993); therefore, they represent a conservative lower-bound value of the sliding stability.

The Canister Transfer Building will be founded on clayey soils that have an adequate amount of cohesive strength to resist sliding due to the dynamic forces from the design basis ground motion. As shown in SAR Figures 2.6-21 through 2.6-23, however, some of the soils underlying the building are cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. Analyses were performed to address the possibility that sliding may occur along a deeper slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

These analyses included the passive resistance acting on a plane extending from grade down to the top of the cohesionless layer, plus the frictional resistance available along the top of the cohesionless layer. The weight of the clayey soils existing between the top of the cohesionless soils and the bottom of the mat was included in the normal force used to calculate the frictional resistance acting along the top of the cohesionless layer. The factor of safety against sliding along the top of this layer was found to be ≥ 1.1 for all of the dynamic load cases; therefore, there is an adequate factor of safety against sliding along the surface of the cohesionless soils underlying the Canister Transfer Building.

Additional analyses of sliding on cohesionless soils, based on Newmark's method for estimating displacements of dams and embankment due to earthquakes, were performed to define the upper bound of potential movement that might occur due to the earthquake if the mat was founded directly on cohesionless soils. In these analyses it was postulated that the cohesionless soils extend above the depth of about 10 ft and the structure is founded directly on the cohesionless materials. Several conservative assumptions were made in these analyses, and even with this high level of conservatism, the estimated relative displacement of the building ranged from 0.5 inches to 1.2 inches.

Motions of this magnitude, occurring at the depth of the silty sand/sandy silt layer, would likely not even be evident at the ground surface. For the building to slide, a surface of sliding must be established between the horizontal sliding surface in the silty sand/sandy silt layer and through the overlying clayey layer. In the simplified model used to estimate these displacements, the contribution of this surface of sliding through the overlying clayey layer to the dynamic resistance to sliding motion is ignored, as is the passive resistance that would act on the embedded portion of the building foundation and the block of soil that is postulated to be moving with it. It is likely, moreover, that should such slippage occur within the cohesionless soils underlying the building, it would minimize the level of the accelerations that would be transmitted through the soil and into the structure. In this manner, these cohesionless soils would act as a built-in base-shear isolation system. Any decrease in these accelerations as a result of this would increase the factor of safety against sliding, which would decrease the estimated displacements as well. Further, since there are no important-to-safety systems that would be severed or otherwise impacted by movements of this small amount as a result of the earthquake, such movements do not adversely affect the performance of the CTB.

BEARING CAPACITY

STATIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

Table 2.6-9 presents the results of the bearing capacity analyses for the following static load cases. The minimum factor of safety required for static load cases is 3.

Case IA Static using undrained strength parameters ($\phi = 0°$ & $c = 3.18$ ksf).

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

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 6 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and $c = 2.2$ ksf, as measured in the UU tests that are reported in Attachment 2 of Appendix 2A of the SAR, to model the end of construction. Using the estimated effective-stress strength of $\phi = 30°$ and $c = 0$ or the total-stress strength of $\phi = 21.1°$ and $c = 1.1$ ksf, as measured in the consolidated undrained triaxial shear tests performed on samples obtained from the Canister Transfer Building area (Attachment 6 of Appendix 2A of the SAR), results in higher allowable bearing pressures (> 20 ksf).

Dynamic Bearing Capacity of the Canister Transfer Building

The dynamic bearing capacity was analyzed using the dynamic loads for the building that were developed in Calculation 05996.02-SC-5, (SWEC, 1999b). The development of these dynamic loads is described in SAR Section 4.7.1.5.3. As in the structural analyses discussed in Section 4.7.1.5.3., the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions.

Table 2.6-10 presents the results of the bearing capacity analyses for the following cases, which include static loads plus dynamic loads due to the earthquake. The minimum factor of safety required for dynamic load cases is 1.1.

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

Table 2.6-10 indicates the minimum factor of safety against a dynamic bearing capacity failure was obtained for Load Case IIIB, the load combination of full static with 40% of the earthquake loading acting in the N-S direction, 40% acting in the vertical direction, tending to unload the mat, and 100% acting in the E-W horizontal direction. This load case resulted in an actual soil bearing pressure of 3.31 kips per square foot (ksf), compared with an ultimate bearing capacity of ~9 ksf. The resulting factor of safety against a bearing capacity failure for this load case is ~3, which is much greater than 1.1, the minimum allowable factor of safety for seismic loading cases. In these analyses, no credit was taken for the fact that strength of cohesive soil increases as the rate of loading increases. Therefore, the Canister Transfer Building has an adequate factor of safety against a dynamic bearing capacity failure.

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.

Casagrande, A. and W.L. Shannon, 1948. "Strength of Soils under Dynamic Loads," Transactions, ASCE, Vol. 74, No.4, April, pp. 591-608.

ConeTec, 1999, Cone penetration testing report, Private Fuel Storage Facility, prepared for Stone and Webster Engineering Corp., Denver, CO, 2 volumes.

Das, B. M., 1993, Principles of Soil Dynamics, PWS-Kent, Boston, MA, 570 pp.

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

Newmark, N.M., 1965. Fifth Rankine Lecture: "Effects of Earthquakes on Dams and Embankments". Geotechnique, Institution of Civil Engineers, London. pp. 139-60.

Schimming, B.B., H.J. Haas, and H.C. Saxe, 1966. Study and Dynamic and Static Envelopes. Journal of Soil Mechanics and Foundation Division, ASCE Vol. 92, No. SM2 (March), pp. 105-24.

SWEC, 1999a. Development of Soil Impedance Functions for Canister Transfer Building. Calculation No. 05996.02-SC-04, Revision 1, Stone & Webster Engineering Corporation.

SWEC, 1999b. Seismic Analysis of Canister Transfer Building. Calculation No. 05996.02-SC-05, Revision 1, Stone & Webster Engineering Corporation.

SWEC, 2000, Calculation No. 05996.02- G(B)-4, Revision 5, Stability Analyses of Storage Pad, Stone & Webster Engineering Corporation.

Vesic, A. S., 1973, "Analysis of Ultimate Loads on Shallow Foundations," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol 99, No. SM1, pp 45-73.

Winterkorn, H. F., and Fang, H., 1975, Foundation Engineering Handbook, Van Nostrand Reinhold Co., New York, NY.

Table 1
Foundation Loadings for the Canister Transfer Building

								N-S SHEAR X	UPLIFT	E-W SHEAR Z	$\Sigma M_{Base\ @\ El\ 95}$	
JOINT	EL.	MASS X	MASS Y	MASS Z	Ax	Ay	Az	F_{HX}	F_{VD}	F_{HZ}	$M_{@x}$	$M_{@z}$
	ft	k-sec^2 / ft	k-sec^2 / ft	k-sec^2 / ft	g	g	g	k	k	k	ft-k	ft-k
1	95	1257.0	1257.0	1257.0	0.805	0.720	0.769	32,583	29,142	31,126	155,628	162,913
2	130	490.7	490.7	490.7	0.864	0.764	0.834	13,652	12,072	13,178	461,218	477,808
3	170	299.2	299.2	157.0	0.939	0.829	0.966	9,047	7,987	4,884	366,264	678,491
4	190	219.8	166.9	219.8	0.955	0.839	1.067	6,759	4,509	7,552	717,417	642,112
5	190	0.0	52.9	0.0	0.000	2.013	0.000	0	3,429	0	0	0
6	170	0.0	0.0	142.2	0.000	0.000	2.366	0	0	10,834	812,515	0
							TOTAL	62,040	57,139	67,572	2,513,041	1,961,325
							WEIGHT		72,988			

Based on sliding and uplift forces from p 37 of Calc 05996.02-SC-5, Rev 1, which are applicable for "High" Moduli received from Geomatrix Calc 05996.02-G(PO18)-2, Rev 0.

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Table 2

Sliding Stability of Canister Transfer Building Using Average Shear Strength from Direct Shear Tests Under Building and Half of Passive Resistance With a 1-ft Deep Perimeter Key

Joint	MASS X	MASS Y	MASS Z	N-S a_x	Vert a_y	E-W a_z	Static F_v	Earthquake $Shear_{N-S}$	Earthquake F_v	Earthquake $Shear_{E-W}$
	k-sec² / ft	*k-sec² / ft*	*k-sec² / ft*	*g*	*g*	*g*	*k*	*k*	*k*	*k*
1	1,257.0	1,257.0	1,257.0	0.805	0.720	0.769	40,475	32,583	29,142	31,126
2	490.7	490.7	490.7	0.864	0.764	0.834	15,801	13,652	12,072	13,178
3	299.2	299.2	157.0	0.939	0.829	0.966	9,634	9,047	7,987	4,884
4	219.8	166.9	219.8	0.955	0.839	1.067	5,374	6,759	4,509	7,552
5	0.0	52.9	0.0	0.000	2.013	0.000	1,703	0	3,429	0
6	0.0	0.0	142.2	0.000	0.000	2.366	0	0	0	10,834
						Totals =	**72,988**	**62,040**	**57,139**	**67,572**

CTB Mat Dimensions: B = **165** ft
L = **265** ft

For ϕ = 0.0 degrees		c = 1.80 ksf				N (*k*)	*Resisting* T (*k*)	*Driving* V (*k*)	FS
Earthquake Vertical Forces Acting Up	IIIA	$F_{v(Static)}$ 72,988	40% $F_{H(NS)}$ 24,816	100% $F_{v(Eqk)}$ -57,139	40% $F_{H(EW)}$ 27,029	15,849	79,169	36,693	**2.16**
	IIIB	$F_{v(Static)}$ 72,988	40% $F_{H(NS)}$ 24,816	40% $F_{v(Eqk)}$ -22,855	100% $F_{H(EW)}$ 67,572	50,132	79,169	71,985	**1.10**
	IIIC	$F_{v(Static)}$ 72,988	100% $F_{H(NS)}$ 62,040	40% $F_{v(Eqk)}$ -22,855	40% $F_{H(EW)}$ 27,029	50,132	79,169	67,672	**1.17**
Earthquake Vertical Forces Acting Down	IVA	$F_{v(Static)}$ 72,988	40% $F_{H(NS)}$ 24,816	100% $F_{v(Eqk)}$ 57,139	40% $F_{H(EW)}$ 27,029	130,126	79,169	36,693	**2.16**
	IVB	$F_{v(Static)}$ 72,988	40% $F_{H(NS)}$ 24,816	40% $F_{v(Eqk)}$ 22,855	100% $F_{H(EW)}$ 67,572	95,843	79,169	71,985	**1.10**
	IVC	$F_{v(Static)}$ 72,988	100% $F_{H(NS)}$ 62,040	40% $F_{v(Eqk)}$ 22,855	40% $F_{H(EW)}$ 27,029	95,843	79,169	67,672	**1.17**

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TABLE 2.6-9

SUMMARY - ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Based on Static Loads

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\ E\text{-}W}$	β_L $EQ_{H\ N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
IA - Static Undrained Strength	72,988	0	0	0	0	0.0	0.0	18.95	**6.31**	0.0	0.0	165.0	265.0	**1.67**	**11.35**
IB - Static Effective Strength	72,988	0	0	0	0	0.0	0.0	135.00	**45.00**	0.0	0.0	165.0	265.0	**1.67**	**80.88**

c =	**3,180**	Undrained strength (psf) & $\phi = 0$.
ϕ =	**30.0**	Effective stress friction angle (deg), c = 0.
B =	**165**	Footing width (ft)
L =	**265**	Footing length (ft)
D_f =	**5.0**	Depth of footing (ft)
γ =	**90**	Unit weight of soil (pcf)
γ_{surch} =	**80**	Unit weight of surcharge (pcf)
FS =	**3**	Factor of safety for static loads.

F_V = Vertical load (Static + EQ_V)

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

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

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

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

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

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

TABLE 2.6-10

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Dynamic Loads Due to Design Basis Ground Motion: PSHA 2,000-yr Return Period

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\ E\text{-}W}$	β_L $EQ_{H\ N\text{-}S}$	GROSS		e_B	e_L	EFFECTIVE			FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	q_{ult} ksf	q_{all} ksf	ft	ft	B' ft	L' ft	q_{actual} ksf	
II	72,988	62,040	67,572	2,513,041	1,961,325	42.8	40.4	10.98	**9.98**	34.4	26.9	96.1	211.3	**3.59**	**3.06**
IIIA	15,849	24,816	27,029	1,005,216	784,530	59.6	57.4	8.75	**7.95**	63.4	49.5	38.2	166.0	**2.50**	**3.50**
IIIB	50,132	24,816	67,572	2,513,041	784,530	53.4	26.3	9.95	**9.04**	50.1	15.6	64.7	233.7	**3.31**	**3.00**
IIIC	50,132	62,040	27,029	1,005,216	1,961,325	28.3	51.1	14.43	**13.12**	20.1	39.1	124.9	186.8	**2.15**	**6.72**
IVA	130,127	24,816	27,029	1,005,216	784,530	11.7	10.8	17.21	**15.64**	7.7	6.0	149.6	252.9	**3.44**	**5.00**
IVB	95,844	24,816	67,572	2,513,041	784,530	35.2	14.5	13.98	**12.70**	26.2	8.2	112.6	248.6	**3.42**	**4.08**
IVC	95,844	62,040	27,029	1,005,216	1,961,325	15.7	32.9	15.65	**14.22**	10.5	20.5	144.0	224.1	**2.97**	**5.27**

$c =$ **3,180** Undrained strength (psf)

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

$B =$ **165** Footing width (ft)

$L =$ **265** Footing length (ft)

$D_f =$ **5.0** Depth of footing (ft)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

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

$F_V =$ Vertical load (Static + EQ_V)

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

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

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

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

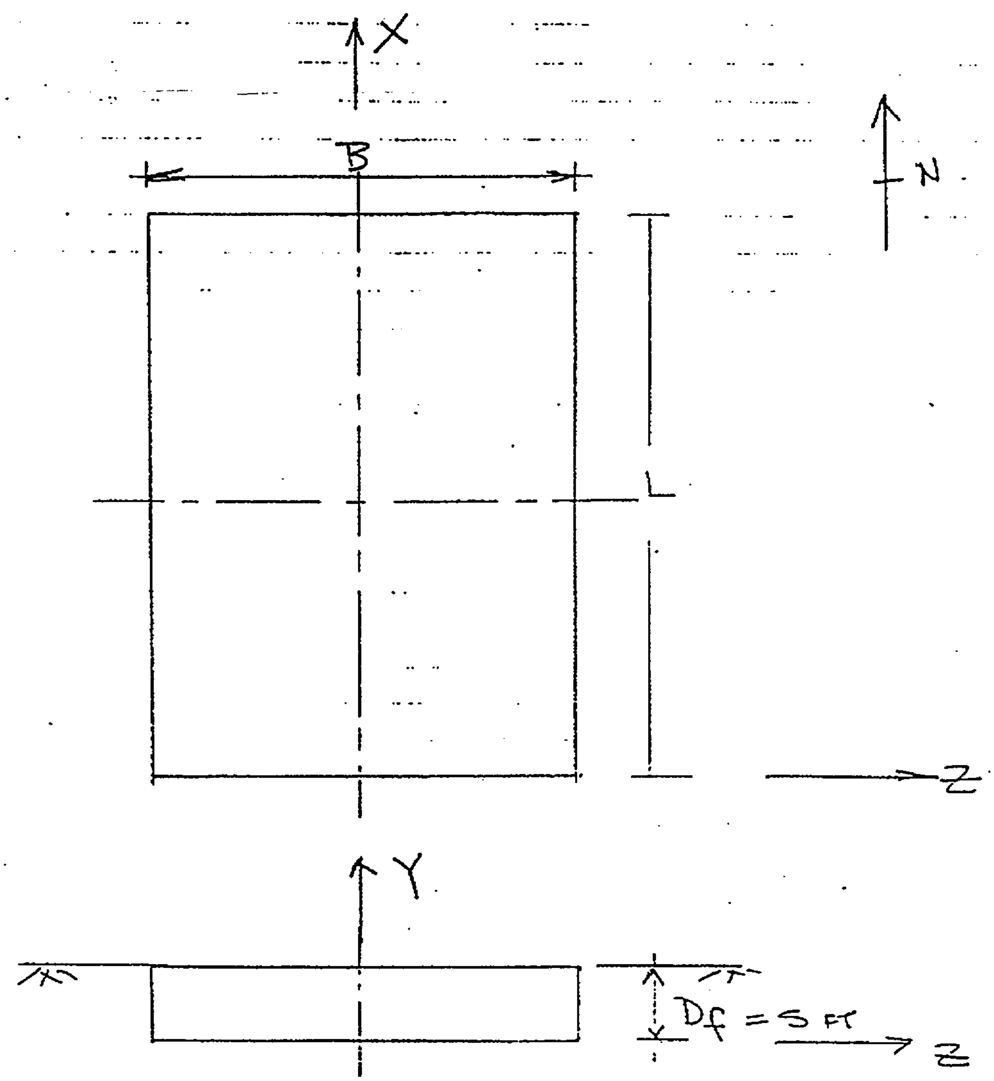
$B' = B - 2\,e_B$ $L' = L - 2\,e_L$

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

[geot]\05996\calc\brng_cap\can_xfr.xls Table 2.6-10

FIGURE 1

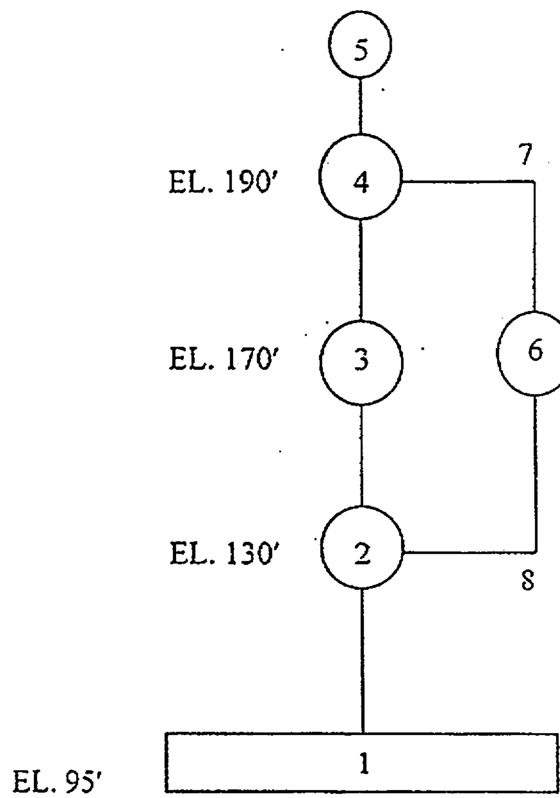
FOUNDATION SCHEMATIC & COORDINATE SYSTEM



Note: The coordinate system is consistent with that used in Calculation 05996.02-SC-5.

Figure 2

Canister Transfer Building Stick Model



Note: From Calculation 05996.02-SC-5, Rev 1.

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 49
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3	N/A	

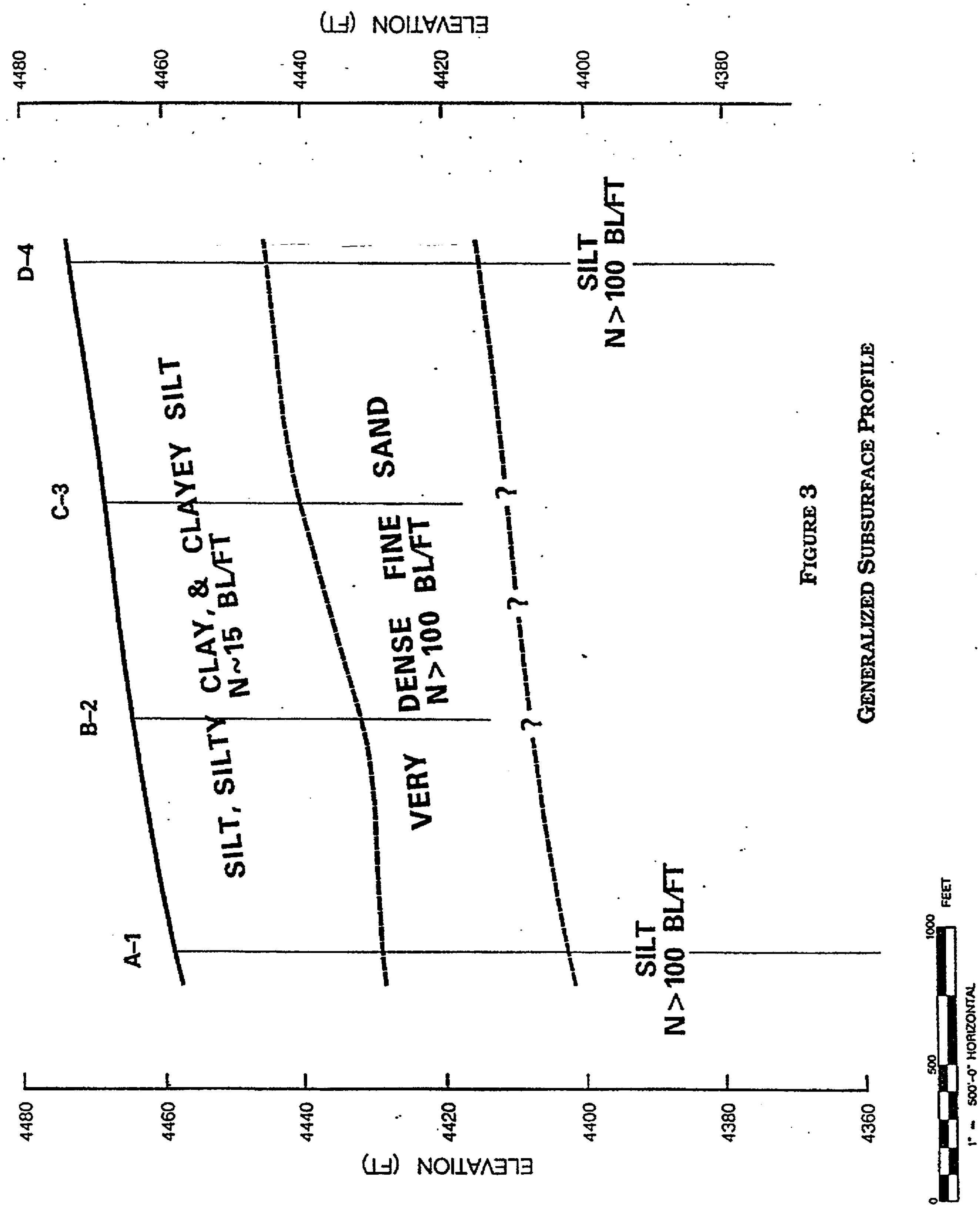


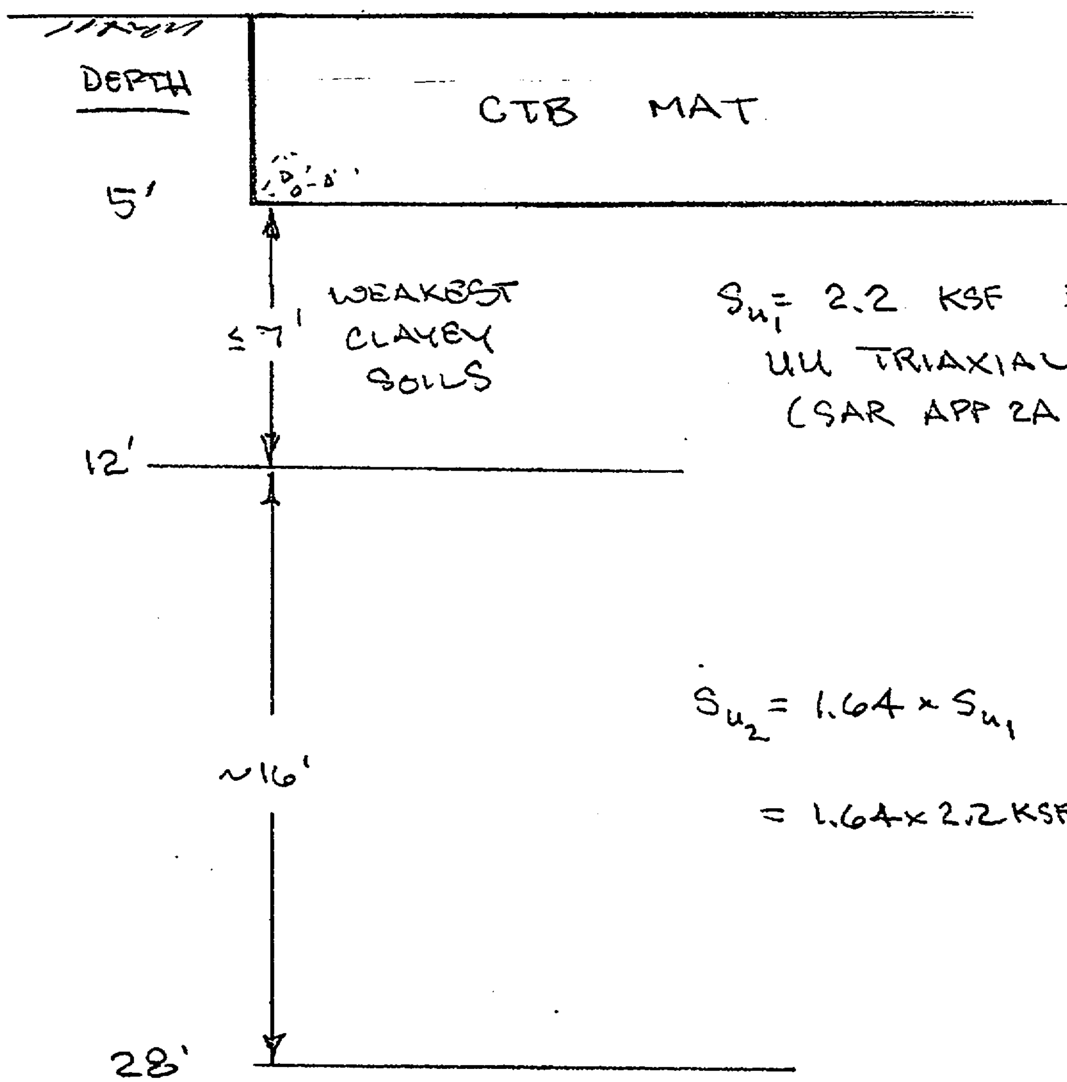
FIGURE 3

GENERALIZED SUBSURFACE PROFILE

Note: From Calculation 05996.01-G(B)-03-1.

FIGURE 4

DETERMINATION OF WEIGHTED AVERAGE VALUE OF S_U BASED ON RELATIVE STRENGTH DIFFERENCE OF DEEPER LYING SOILS MEASURED IN CONE PENETRATION TESTS



$S_{u_1} = 2.2$ KSF BASED ON UU TRIAXIAL TESTS (SAR APP 2A ATT 2)

$S_{u_2} = 1.64 \times S_{u_1}$

$= 1.64 \times 2.2 \text{ KSF} = 3.6 \text{ KSF}$

$$S_{u\,AVG} = \frac{7' \times 2.2 \text{ KSF} + 16' \times 1.64 \times 2.2 \text{ KSF}}{23 \text{ FT}} = 3.18 \text{ KSF}$$

FIGURE 5

ESTIMATE STRESSES UNDER THE CANISTER TRANSFER BUILDING AT COMPLETION OF CONSTRUCTION

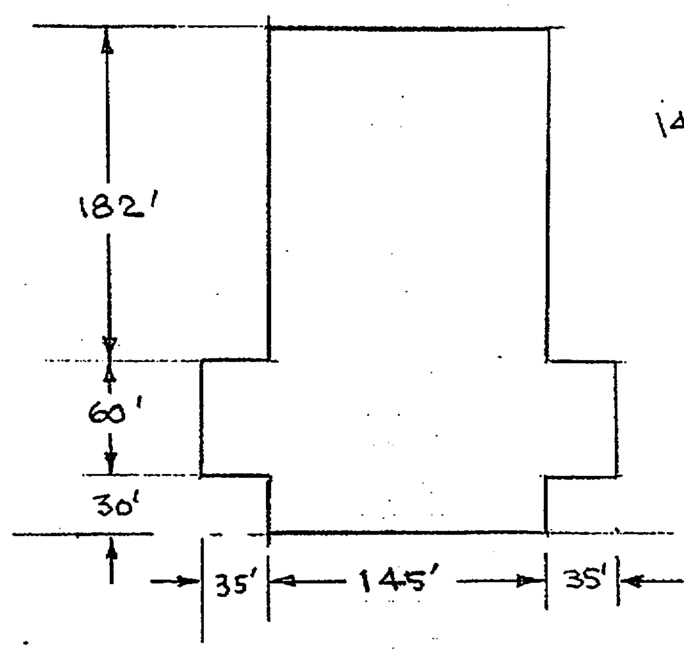
$\sigma_v = \dfrac{F_v}{A}$ F_v = TOTAL DEAD WT

A = AREA OF MAT

SOURCES OF DATA:

F_v = 72,988 K STATIC D.L. p37 OF CALC 05996.02-SC-5-1

MAT AREA: DRAWING 0599602-EC-2-A



AREA (FT^2)

145' × 182' = 26,390

$60' \times (\overset{215'}{145+70}) = 12,900$

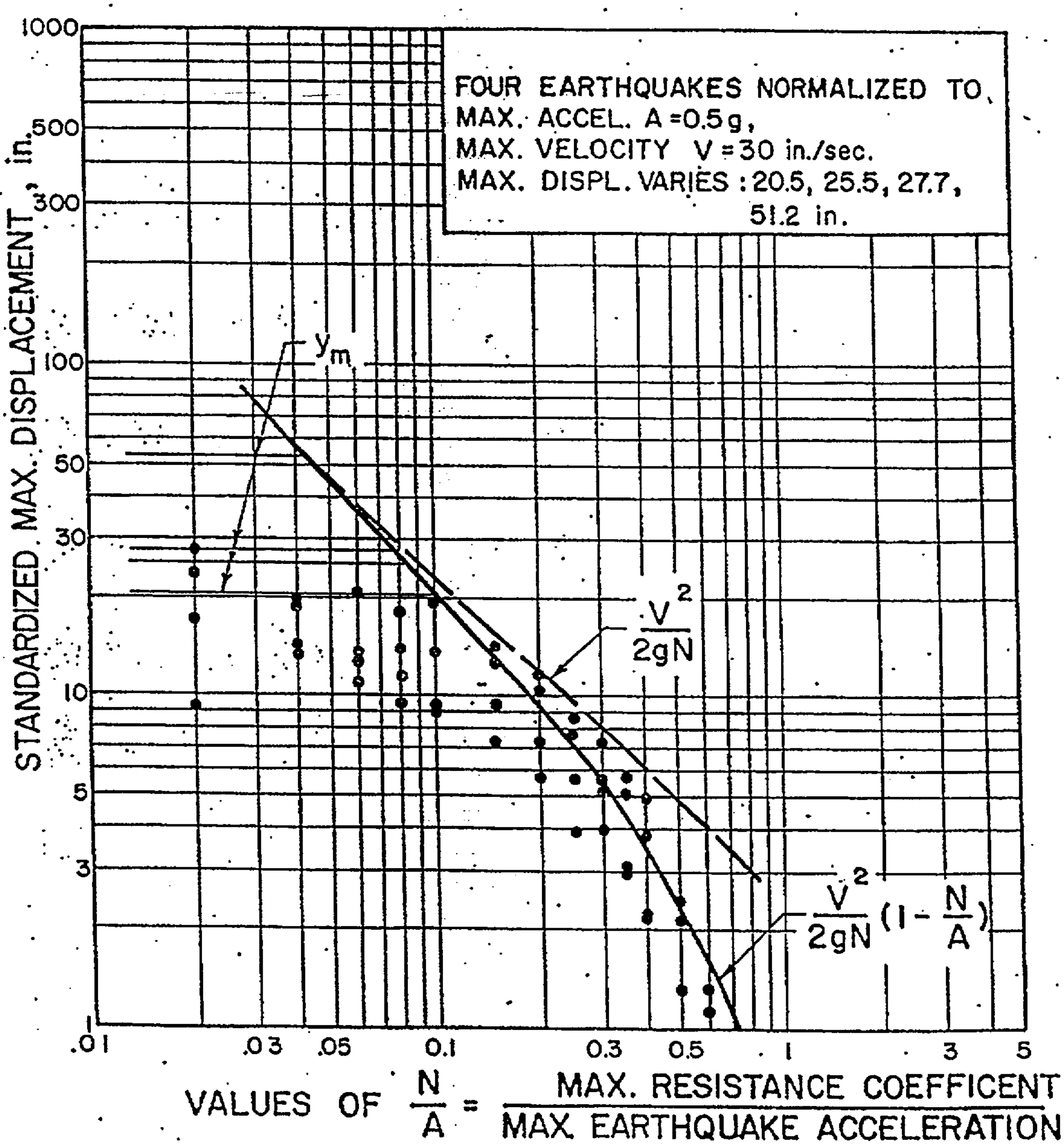
30' × 145' = 4,350

43,640 FT^2

⇒ $\sigma_v = 72{,}988^K / 43{,}640\ FT^2 = 1.67$ KSF

FIGURE 6

STANDARDIZED DISPLACEMENT FOR NORMALIZED EARTHQUAKES (SYMMETRICAL RESISTANCE)



Note: From Newmark (1965)

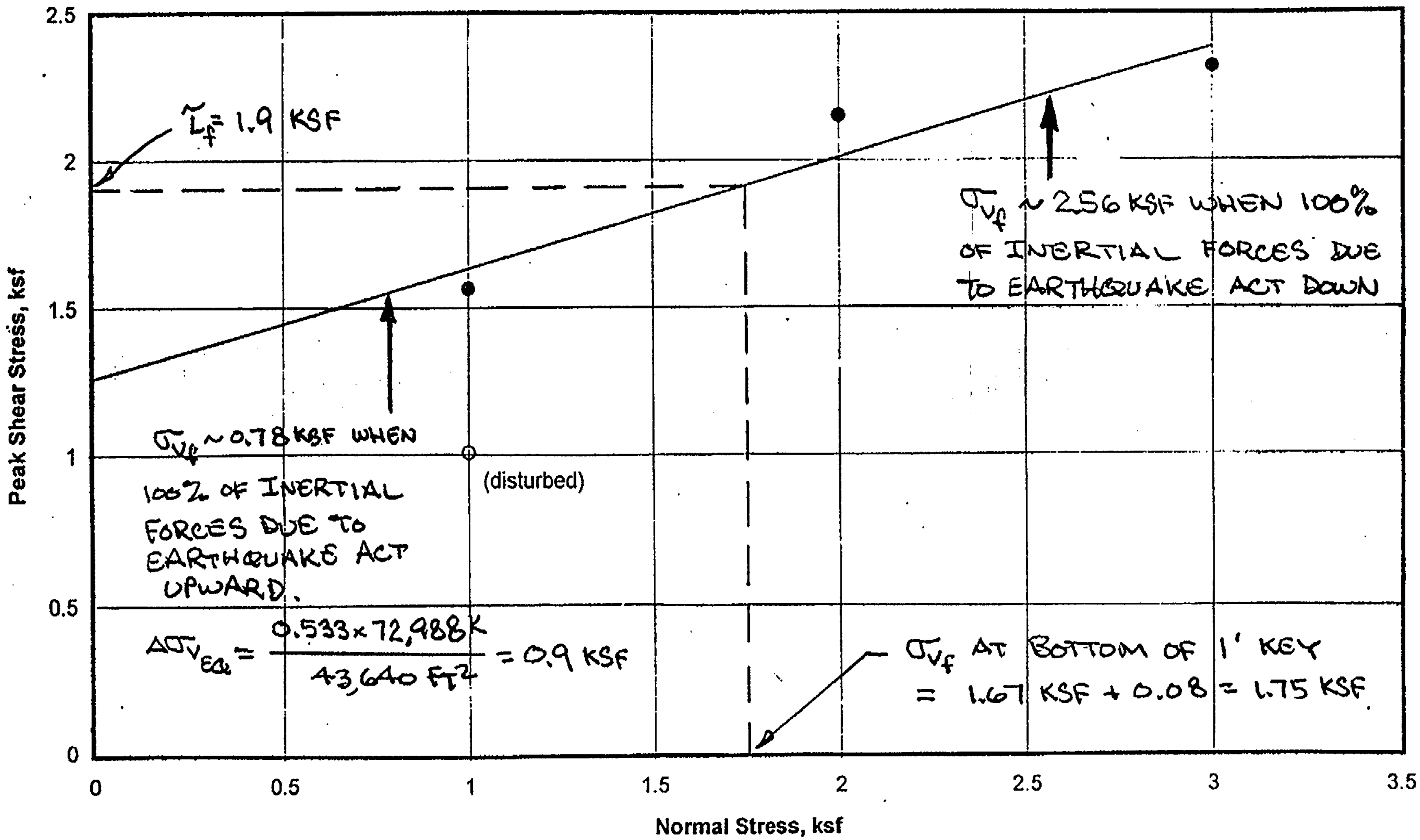
TABLE 6
SUMMARY OF TRIAXIAL TEST RESULTS FOR SOILS WITHIN ~10 FT OF GROUND SURFACE AT THE SITE

Boring	Sample	Depth ft	Elev ft	w %	ATTERBERG LIMITS LL	PL	PI	USC Code	γ_m pcf	γ_d pcf	e_o	σ_c ksf	s_u ksf	ε_a %	Type	Date
B-1	U-2C	5.9	4453.9	47.1	66.1	33.4	32.7	MH	79.3	53.9	2.15	0.0	2.03	1.7	CU	Nov '99
B-1	U-2B	5.3	4454.5	52.9	80.6	40.9	39.7	MH	70.8	46.3	2.67	1.0	2.21	6.0	CU	Nov '99
B-4	U-3D	10.4	4462.1	27.4	42.5	24.7	17.8	CL	85.5	67.1	1.53	1.3	2.18	4.0	UU	Jan '97
C-2	U-2D	11.1	4453.4	35.6	See U-2C & E[1]			CL	78.5	57.9	1.93	1.3	2.39	11.0	UU	Jan '97
CTB-1	U-3D	8.7	4463.7	47.9	See U-3C[2]			CH	91.9	62.1	1.73	1.7	2.84	5.0	CU	June '99
CTB-4	U-2D	9.5	4465.5	45.2	See U-2E[2]			CH	87.7	60.4	1.81	1.7	3.11	6.0	CU	June '99
CTB-6	U-3D	8.3	4467.9	52.7				CH	85.7	56.2	2.02	1.7	2.70	7.0	CU	June '99
CTB-N	U-1B	5.7	4468.4	30.1	41.3	22.5	18.8	CL	100.6	77.3	1.20	1.7	3.00	8.0	CU	Nov '98
CTB-N	U-2B	7.7	4466.4	65.4	See U-2A[2]			MH	74.6	45.1	2.76	1.7	2.41	13.0	CU	June '99
CTB-N	U-3D	10.5	4463.6	52.2	61.1	30.8	30.3	CH	86.3	56.7	1.98	1.7	2.73	7.0	CU	June '99
CTB-S	U-1B	5.8	4468.7	73.6	66.2	40.9	25.3	MH	78.0	44.9	2.78	1.7	2.05	12.0	CU	Nov '98
CTB-S	U-2D	8.4	4466.1	54.6	57.9	28.9	29.0	CH	90.0	58.2	1.92	1.7	2.40	5.0	CU	June '99
B-1	U-2D	6.5	4453.3	45.2	59.8	34.7	25.1	MH	76.7	52.8	2.22	2.1	3.26	15.0	CU	Mar '99
B-3	U-1B	5.2	4463.0	33.5	52.4	25.2	27.2	MH	90.6	67.9	1.50	2.1	3.55	8.0	CU	Mar '99
C-2	U-1D	6.3	4458.2	50.5	70.3	41.3	29.0	MH	74.5	49.5	2.43	2.1	3.03	12.0	CU	Mar '99

NOTES 1 Attachment 2 of SAR Appendix 2A.
2 Attachment 6 of SAR Appendix 2A.

ATTACHMENT A pA1/4
TO CALC 05996.02-G(B)-13-3

FIGURE 9
DIRECT SHEAR TEST
Boring CTB-6, Sample U-3B&C
CANISTER TRANSFER BUILDING AREA



CALC 05996.02-G(B)-05-2 - p 34

PFSF SKULL VALLEY, UT
SAR APP 2A ATT 7

ATTACHMENT A p A2
CALC 05996.02-G(B)-13-3

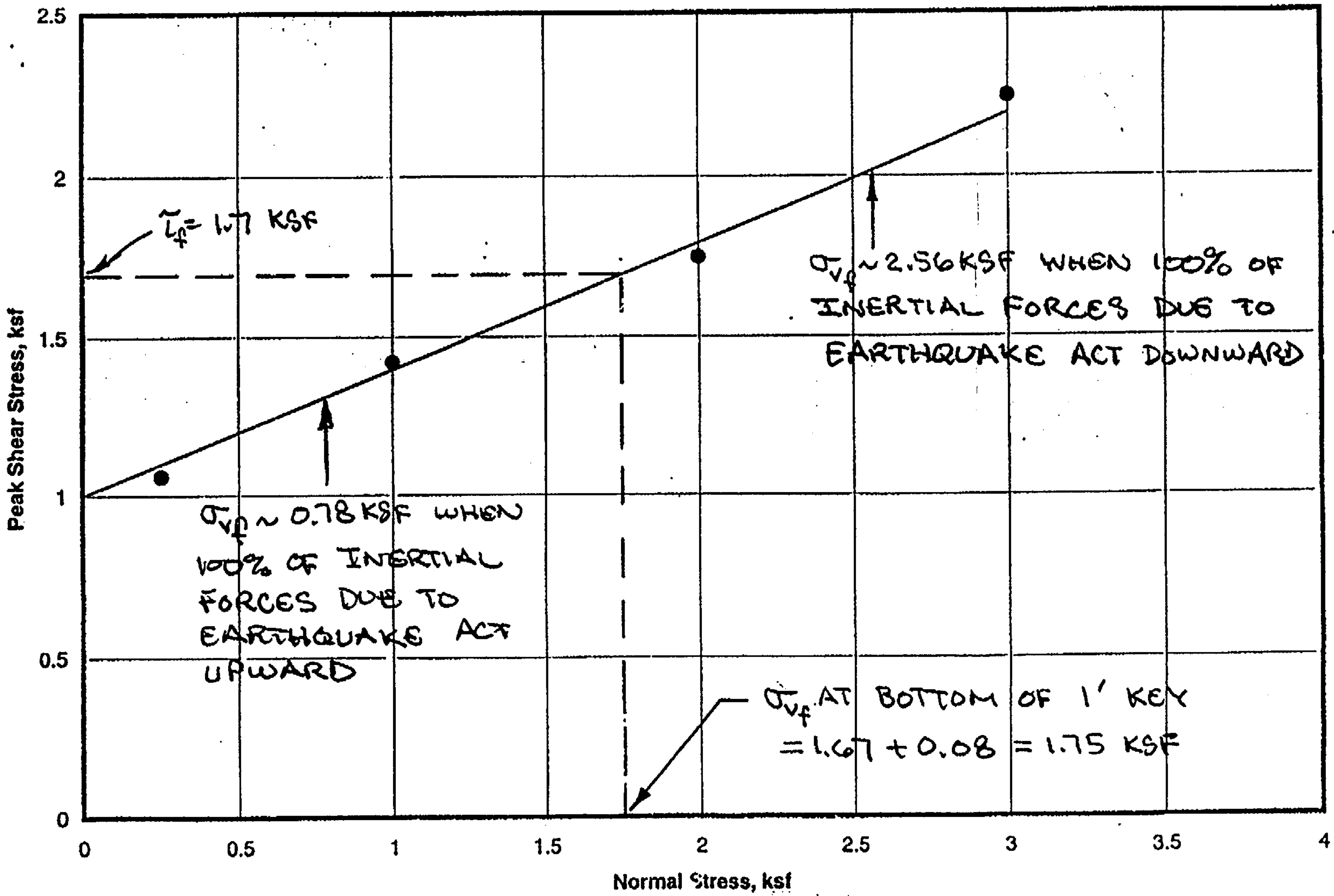
STONE & WEBSTER ENGINEERING CORPORATION

FIGURE 10

DIRECT SHEAR TEST

Boring CTB-S, Sample U-1AA&C

CANISTER TRANSFER BUILDING AREA



CALC 05996.02-G(B)-05-2 P 35

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

J.O. OR W.O. NO. 05996.02

DIVISION & GROUP G(B)

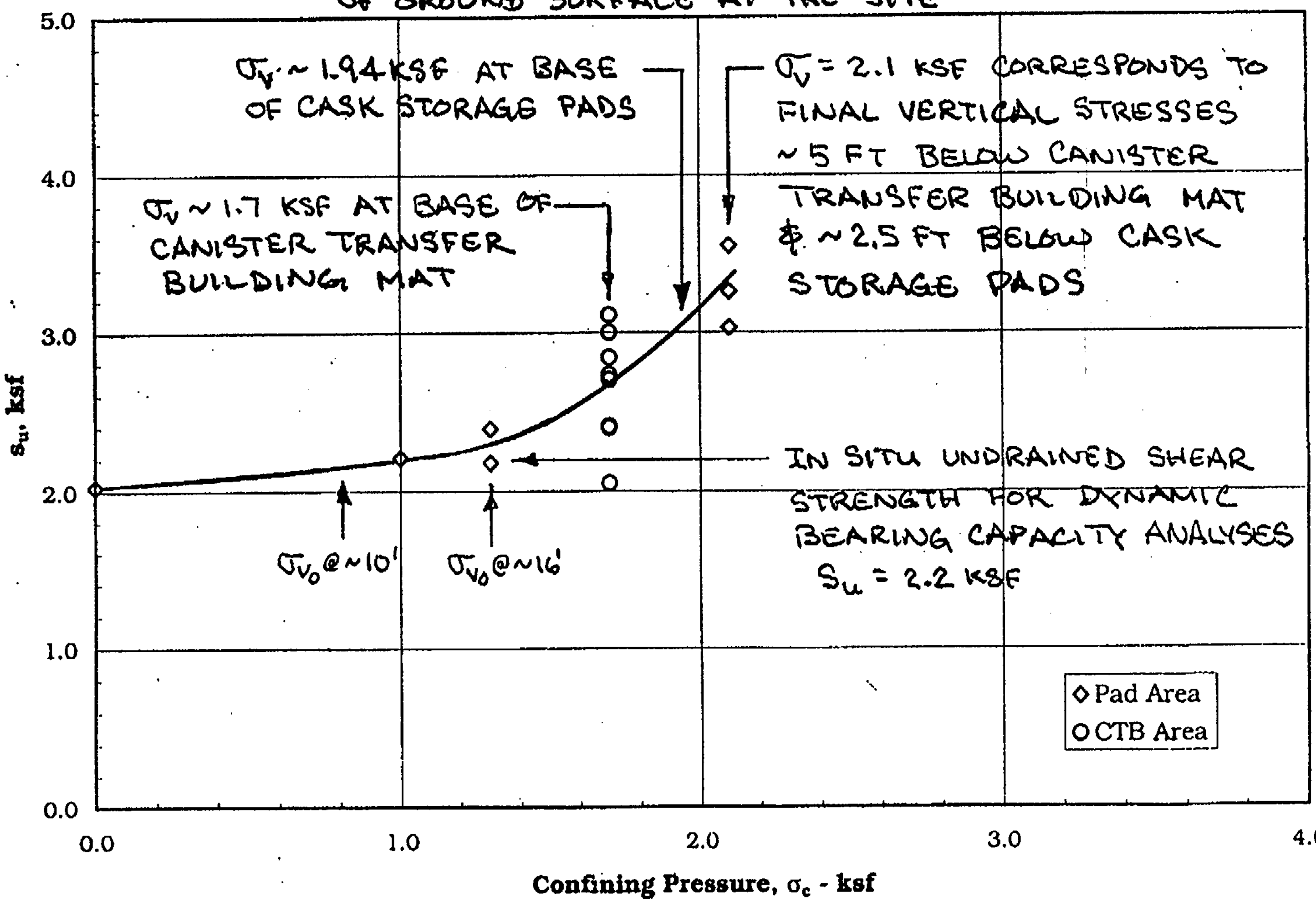
CALCULATION IDENTIFICATION NUMBER

CALCULATION NO. 05-2

OPTIONAL TASK CODE

PAGE 36

Figure 11
Summary of Triaxial Test Results for Soils Within Depth of ~10 ft OF GROUND SURFACE AT THE SITE



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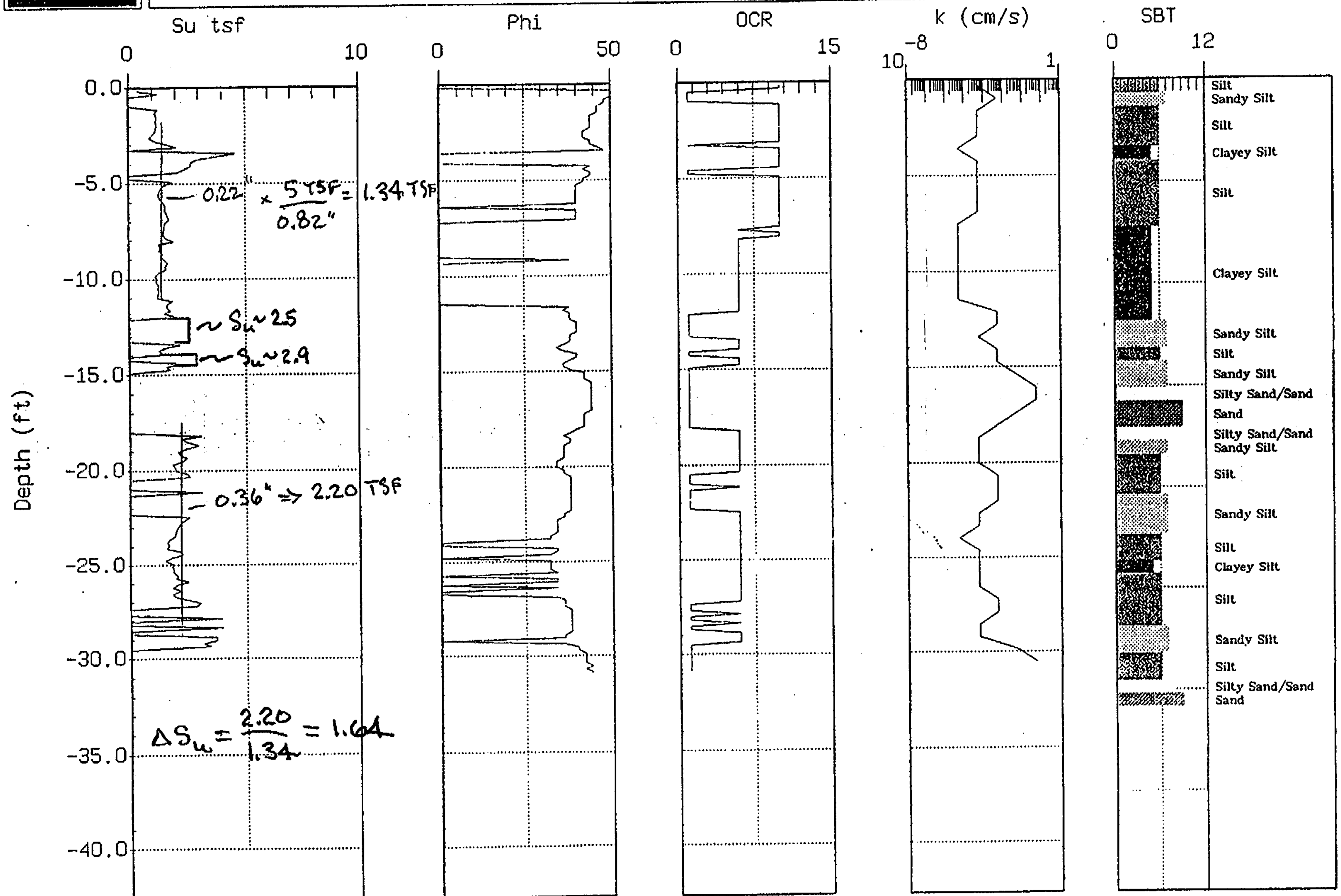
ATTACHMENT A p A4
CALC 05996.02-G(B)-13-3

CONETEC

Stone & Webster

Site: CPT-37
Location: PFSF (05996.02)

Cone: 20 TON A 041
Date: 04:23:99 11:36



Max. Depth: 30.84 (ft)

Depth Inc.: 0.164 (ft)

Permeability k: estimated from soil type

SBT: Soil Behavior Type (Robertson and Campanella 1988)

(5)

IED JUN 2 2000 P.J. Trudeau

ATTACHMENT B

CALC 05996.02-G(B)-13-3

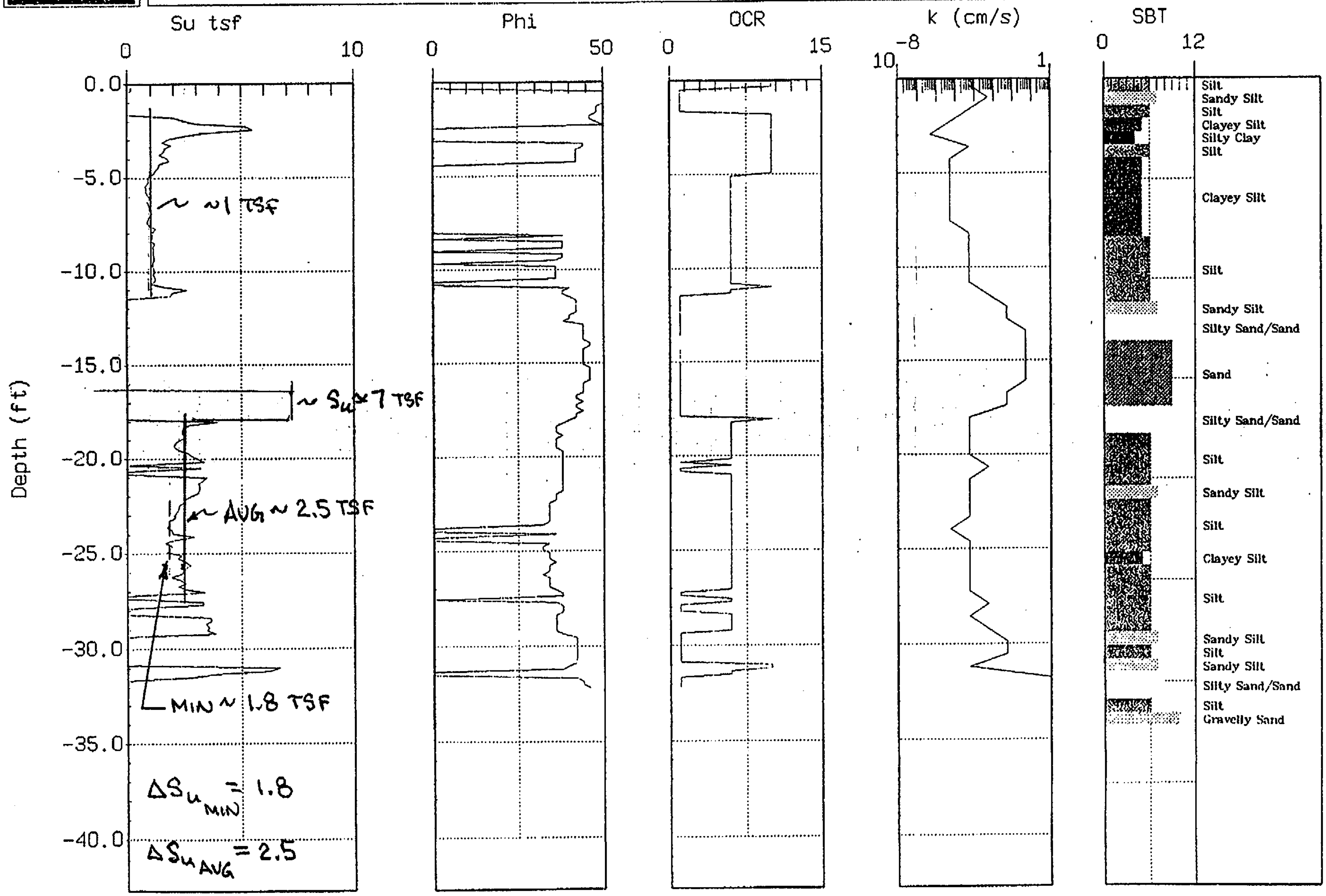
P B1/2

CONETEC

Stone & Webster

Site: CPT-38
Location: PFSF (05996.02)

Cone: 20 TON A 041
Date: 04:23:99 15:06



Max. Depth: 32.15 (ft)

Depth Inc.: 0.164 (ft)

Permeability k: estimated from soil type

SBT: Soil Behavior Type (Robertson and Campanella 1988)

ATTACHMENT B
CALC 05996.02-G(B)-13-3 P B2

JUN 2 2000 P.I.

Private Fuel Storage Facility

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-13
Revision No. 3

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
Method			
Identify the method used to verify the "Method" of the calculation			
• By design review	✓		
• Compare the Method with another calculation			✓
• Alternate calculation			✓
If the compare method was used, is the statement identifying the other calculation identified in this calculation?			✓
If an alternate calculation was used for a QA Category I calculation, is it included with the calculation?			✓
Is the calculation method acceptable?	✓		
Assumptions			
Affirmative answers to the following questions are required:			
• Are all assumptions uniquely identified as assumptions and adequately described?	✓		
• Are all assumptions reasonable?	✓		
• Are all assumptions that require confirmation at a later date specifically identified as assumptions that must be confirmed?	✓		
For Revisions to the Calculation			
• Are changes clearly identified?	✓		
• For QA Category I calculations, is a reason for the revision given?	✓		
• Does the calculation identify the calculation, including revision, when applicable, which is superseded?	✓		

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-13
Revision No. 3

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
• Are affected pages identified with the new calculation number or revision number?	✓		
• When applicable, is an alternate calculation included as part of the calculation?			✓
• When applicable, is a statement identifying the calculation to which the method was compared included as part of the revision?			✓

Thomas Y. Chang
Printed Name

Thomas Y. Chang
Signature

6-19-2000
Date

ATTACHMENT 2

STABILITY EVALUATION OF THE PFSF CASK TRANSPORTER CARRYING A STORAGE CASK WHEN SUBJECTED TO DESIGN BASIS GROUND MOTION AND DESIGN TORNADO MISSILE

Stability of the Cask Transporter Carrying a Storage Cask Loaded with Spent Fuel

The following evaluation is provided to quantify the effects of natural forces on the transporter loaded with a cask full of spent fuel assemblies to show that a loaded transporter will not tip or overturn.

Information was reviewed from two track type cask transporters that have recently been supplied for casks similar to those that will be used at the PFSF to establish a basis for the cask transporter stability analysis, since the actual transporter to be used at the PFSF has not been determined. The transporters are manufactured by J&R Engineering and Lift Systems (References 1 and 2). The following information was collected:

Attribute	J&R Engineering 160 ton unit	Lift Systems 180 ton unit
Width of transporter	228 in.	228 in.
Length of transporter	336 in.	297 in.
Height of transporter (w/ cask)	264 in.	271 in.
Center of Gravity Height	55 in.	66 in.
Weight of transporter (w/o cask)	185,000 lbs.	160,000 lbs.

The transporter by Lift Systems will be used to evaluate the transporter stability since it has the same width, highest center of gravity, highest height, and lowest weight.

The following information regarding the storage casks was obtained from the HI-STORM and TranStor SARs (References 3 and 4):

Attribute	HI-STORM	TranStor
Height of storage cask	231 in.	223 in.
Diameter of storage cask	133 in.	136 in.
Center of Gravity Height	123 in.	114 in.
Weight of loaded storage cask	355,575 lbs.	307,600 lbs.

The TranStor storage cask will be used in the cask transporter stability analysis since it has considerable less weight to resist overturning and approximately the same height and diameter.

a. Stability of a Loaded Cask Transporter with Tornado Missile Impact

The tornado-generated missile loading specified in PFSF SAR Table 3.6-1 used for this analysis is a 3990 lb. automobile traveling at a horizontal velocity of 134 ft/sec. It is assumed this missile would produce the highest momentum for tipping the loaded cask transporter. The tornado missile is assumed to strike the transporter in the worse case direction, which is against the side where the transporter has the least width i.e., resistance to tipover. In addition, the automobile is placed at the top of the transporter for maximum tipping potential.

The impact is assumed to be totally inelastic such that all kinetic energy from the airborne missile is transferred to the loaded transporter into potential energy as the cask transporter tips and the center of gravity lifts. It is also assumed that the transporter components will retain structural integrity during missile impact. In the event a component, such as the lift beam, fails, the cask will simply drop approximately 4" to the ground. The HI-STORM and TranStor storage casks are determined to be structurally sound for drops up to 11 inches and 18 inches respectively, as shown in Section 8.2.6.

Using the conservation of momentum, the loaded transporter angular velocity about the pivot point (ω_p) is:

$$\omega_p = \frac{mm \cdot vcg \cdot Vo}{m_m(v_{cg})^2 + I_p}$$

where:

m_m = mass of missile = 3990 lbs / 386 in/sec^2 = 10.34 lbm
V_o = initial velocity of missile = 134 fps = 1608 in/sec
I_p = moment of inertia of loaded transporter about the pivot point
v_{cg} = vertical distance from center of gravity of a loaded transporter to the ground = combination of the cask center of gravity height when the cask is raised 4 in. above the ground and the transporter center of gravity height or

$$v_{cg} = [(cask_{cg} + 4 \text{ in.}) W_{cask} + (transporter_{cg}) W_{xptr}] / W_t$$

$$v_{cg} = [(114 + 4)\ 307{,}600 + (66)\ 160{,}000] / 467{,}600 = 100 \text{ in.}$$

The moment of inertia of the cask about the pivot point is:

$$I_{p\ cask} = m_{cask}/12(3r_{cask}^2 + h_{cask}^2) + m_{cask}\, d_{cg\ cask}^2$$

where:

m_{cask}	=	mass of cask = 307,600 lbs / 386 in sec^2 = 797 lbm
r_{cask}	=	radius of cask = 136 in./2 = 68 in.
h_{cask}	=	height of cask = 223 in.
$d_{cg\ cask}$	=	distance from cask center of gravity to the pivot point calculated from the cask center of gravity height raised 4" (118") and the horizontal distance from the center of gravity to the pivot point (taken as half the transporter width, 228 in. /2 = 114) or $d_{cg\ cask} = [(118)^2 + (114^2]^{1/2} = 164$ in.

Therefore, the cask moment of inertia is:

$$I_{p\ cask} = 797/12\ [3(68)^2 + (223)^2] + (797)(164)^2 = 25.66 \times 10^6 \text{ in·lb·sec}^2$$

The moment of inertia of the transporter about the pivot point is (assume the transporter is a rectangular parallelepiped that represents the lower "track" portion of the transporter where most of the weight is located):

$$I_{p\ xptr} = m_{xptr}/12\ (h_{xptr}^2 + w_{xptr}^2) + m_{xptr}\, d_{cg\ xptr}^2$$

where:

m_{xptr}	=	mass of transporter = 160,000 lbs/386 in sec^2 = 415 lbm
h_{xptr}	=	height of transporter (assume twice the height of the center of gravity) = 66 in. x 2 = 132 in.
w_{xptr}	=	overall width of transporter = 228 in.
$d_{cg\ xptr}$	=	distance from transporter center of gravity to the pivot point calculated from the transporter center of gravity height (66") and the horizontal distance from the center of gravity to the pivot point (taken as half the transporter width, 228 in./2 = 114") or $d_{cg\ xptr} = [(66)^2 + (114)^2]^{1/2} = 132$ in.

$$I_{p\ xptr} = 415/12\ (132^2 + 228^2) + (415)(132)^2 = 9.63 \times 10^6 \text{ in·lb·sec}^2$$

Total $I_p = 25.66 \times 10^6 + 9.63 \times 10^6 = 35.29 \times 10^6$ in·lb·sec^2

Therefore, the angular velocity (ω_p) about the pivot point is:

$$\omega_p = \frac{(10.34)(100)(1608)}{(10.34)(100)^2 + 35.29\text{x}10^6} = 0.047 \text{ rads/sec}$$

As the loaded transporter tips about the pivot point at impact, the kinetic energy is transferred to potential energy as the center of gravity rises a distance y:

$$E_{tipping} = \text{Kinetic Energy} = \text{Increase in Potential Energy}$$
$$= \tfrac{1}{2} I_p \, \omega_p^2 = W_t \, y$$
$$= \tfrac{1}{2} (35.29\text{x}10^6)(.047)^2 = 467{,}600 \ y$$
$$y = \underline{0.083 \text{ in.}}$$

Clearly, the effect of the airborne automobile impact on the loaded transporter is negligible and will not tip over the cask transporter.

b. Stability of a Loaded Cask Transporter Under Seismic Conditions

The transporter is not designated an important to safety component and therefore is not subject to specific seismic design requirements. However, this section provides the necessary evaluation based on the PFSF design basis ground motion peak ground acceleration ensuring that the loaded transporter will not tip due to seismic loading.

The loaded transporter is generally a flexible system with low frequencies, which would probably not be excited due to the short duration of a seismic event. In the event a seismic load could cause a failure of the transporter structure, the cask would drop or lower to the ground as vehicle members fail or yield. In the event that the cask were to drop, the HI-STORM and TranStor storage casks are determined to be structurally sound for drops up to 11 inches and 18 inches respectively, as shown in PFSF SAR Section 8.2.6.

Since the transporter is rectangular in shape, consider an earthquake in the worst case direction, which is perpendicular to the width of the transporter. In order for the loaded transporter to tip or overturn, the moments caused by the earthquake accelerations must exceed the resisting moment due to the loaded transporter weight. Calculating the moments about the pivot point:

$$M_{p\,eq} = g\, W_t\, v_{cg} + g\, W_t\, h_{cg}$$

$$M_{p\,resist} = W_t\, h_{cg}$$

where:

- g = design earthquake acceleration = 0.53g (horizontal & vertical)
- W_t = total weight of cask & transporter = 307,600 lbs. (cask) + 160,000 lbs. (cask transporter) = 467,600 lbs.
- v_{cg} = vertical distance from center of gravity of a loaded transporter to the ground = 100 in.
- h_{cg} = horizontal distance from center of gravity of a loaded transporter to the pivot point (half the transporter width) = 228 in./2 = 114 in.

Therefore, the moments are:

$$M_{p\,eq} = (0.53)(467{,}600)(100) + (0.53)(467{,}600)(114) = \underline{53{,}035{,}192 \text{ in}\bullet\text{lbs}}$$

$$M_{p\,resist} = (467{,}600)(114) = \underline{53{,}306{,}400 \text{ in}\bullet\text{lbs}}$$

Since the moment due to the earthquake acceleration is less than the moment due to the loaded transporter weight, the loaded transporter will not tip or overturn as a result of the PFSF design basis ground motion.

However, the difference in moments is slight. If the storage cask is carried higher than 4 in. off the ground as allowed by the storage system Technical Specifications, thus raising the loaded transporter center of gravity, it is possible that the moment due to the earthquake could exceed the resisting moment and the transporter could begin to tip. Therefore, to preclude any incipient tipping, the specification to purchase the transporter for PFSF will include requirements to analyze any proposed transporter design to ensure that its dimensions, center of gravity, and weight when carrying a loaded storage cask are such that the loaded transporter will not begin to tip due to the PFSF design basis ground motion.

References

1. J&R Engineering Company, Inc. fax from R. Johnston to DW Lewis of Stone & Webster, J&R Engineering Drawing No. 1481L001, Rev. B, "Preliminary Layout TL250-40 Commonweath Edison," with revisions to suit PFSF, dated June 15, 2000.

2. Lift Systems electronic letter from J. Pelkey to DW Lewis of Stone & Webster, Lift Systems Drawing No. xxxxxxx, Rev. x, "Palo Verde," dated June 14, 2000.

3. Topical Safety Analysis Report for the Holtec International Storage and Transfer Operation Reinforced Module Cask System (HI-STORM 100 Cask System), Holtec Report HI-951312, Docket 72-1014, Revision 10, February 2000.

4. Safety Analysis Report for the TranStor Storage Cask System, SNC-96-72SAR, Sierra Nuclear Corporation, Docket 72-1023, Revision C, November 1998.

ATTACHMENT 3

PROPANE RELEASE ANALYSIS WITH DISPERSION AND DELAYED IGNITION

PROPANE RELEASE ANALYSIS
(DISPERSION AND DELAYED IGNITION)
PFSF, SKULL VALLEY, UT

Prepared by: Kenneth W. Dungan, P.E. and Terry L. Miller Ph.D., P.E.
June 14, 2000

This report addresses dispersion modeling and delayed cloud ignition of propane releases from a proposed 5,000 and 20,000 gallon tank to be located in Skull Valley, Utah. The objective of the modeling was to determine the maximum downwind distance from the tank that the concentration of propane in the plume could be above the lower explosive limit (LEL), and to determine the overpressure created by delayed ignition of the resulting cloud. The LEL for propane is 2.1% by volume (taken from an Air Products Corp. MSDS).

Four (4) different scenarios for the release were evaluated. These were: (1) a 2 inch diameter hole in the top of the tank allowing only propane gas to be released; (2) a 2 inch diameter hole in the bottom of the tank allowing liquid propane to be released; (3) an instantaneous release of the entire contents of a full 20,000 gallon tank; and (4) an instantaneous release of the entire contents of a full 5,000 gallon tank. For each case the tank was assumed to be full of liquid and gaseous propane (2-phases), at a temperature of 20° C (68° F), and at 8.4 atmospheres of pressure. This is the saturation vapor pressure for propane at 20° C. Atmospheric conditions were assumed to be the worst case for dispersion (i.e. nighttime with very stable conditions – stability class F, 20° C, and low wind speeds). The wind speeds used in the different model runs varied between 1-5 m/s, but were in each case the wind speed that caused the highest predicted concentration at a distance of 549 m (1800 ft). This is the distance from the proposed tank(s) to the Canister Transfer Building.

Two different models were used for the dispersion analysis. These were the **TSCREEN** model and the **SLAB** model. The TSCREEN model was developed by the United States Environmental Protection Agency (USEPA) for use in predicting maximum concentrations resulting from toxic chemical releases. It has algorithms to predict the release rate of 2-phase chemicals (like propane) from pressurized tanks with holes of various sizes and uses the Britter & McQuade (B&M) dispersion model to predict the dispersion of denser-than-air plumes. The TSCREEN model was used to calculate the release rates of propane from a 2 inch diameter hole in the tank, and the ambient concentrations resulting from the release. The SLAB model was developed by the University of California (Riverside) to predict the dispersion of large scale releases of 2-phase, denser-than-air plumes from tank spills. The SLAB model is recommended in the TSCREEN users manual for this use. It has been compared with data obtained from field-scale heavy gas dispersion experiments. In these comparisons, SLAB performed well, predicting the lower flammability limit distance in LNG tests to within approximately 15%. Both of these models are commonly used and widely accepted for such applications.

The explosion overpressure calculations applied the TNT equivalency method using a scaled ground distance parameter, Z, value of 45 for a hemispherical surface explosion overpressure of 1 psi.

$R = Z\ (W_e^{1/3})$

R = distance from center of cloud

W_e = TNT equivalent mass = Y (W) (Hc)/(Hc_{TNT})

Y = explosion yield (0.03 for propane)

W = mass of propane in cloud

Hc = Heat of combustion of propane (21,591 Btu/lb)

Hc_{TNT} = heat of combustion of TNT (1,890 Btu/lb)

The results of the modeling are summarized below for each scenario showing the distance downwind to the LEL, and the distance from the tank to reach a 1 psi overpressure.

Scenario	**Distance to LEL meters (ft)**	**Distance to 1 psi meters (ft)**
(1) 2" hole in top of tank (Gas Phase Only)	< 200 (656)	177 (580)
(2) 2" hole in bottom of tank (2-phase release)	450 (1476)	438 (1437)
(3) 20,000 gallon instantaneous release	700 (2296)	651 (2135)
(4) 5,000 gallon instantaneous release	400 (1312)	389 (1276)

The instantaneous release of 20,000 gallon of propane is the only scenario predicted to have concentrations exceeding the LEL beyond 549 meters (1800 ft.). The other scenarios are not expected to have concentrations above the LEL at this distance.

Scenario (1) is a 2" hole in the top of the propane tank above the liquid level. In this circumstance, gases will exit the tank initially under 8.4 atmospheres of pressure, and at sonic velocities. As the pressure drops, liquid propane will flash to vapor (essentially boil) producing more gas phase propane until the liquid is cooled to its boiling point of minus 42° C. If the tank is initially at + 20° C temperature, there is enough heat capacity in the liquid propane to vaporize 37% of the total mass of propane in the tank, leaving 63% of the propane as a sub-cooled liquid in the tank. The emission rate of gaseous propane was calculated using TSCREEN to be 3.49 kg/s. The duration of emissions was predicted to be 66.8 minutes. This duration represents the gas phase release. As the propane evaporated the liquid will cool leaving liquid in the tank after the evaporative cooling. The remaining liquid will boil off but at a much slower rate. The emissions were modeled as if the hole pointed downward minimizing plume rise. It was determined that a wind speed of 3 m/s resulted in the furthest extension of the LEL from the emission source. Ground level concentrations of propane were predicted to exceed the LEL at 100 meters downwind, but not at 200 meters downwind. Based on a 3 m/s wind speed and a travel distance of 200 m, the mass of the cloud was calculated as 67 sec X 3.49 kg/s, or 234 kg (515 lbs.) It was conservatively assumed that all of the propane released from the tank in the 67 second time interval to achieve steady state plume conditions was involved in an explosion. However, some of the propane-air mixture would be below the LEL concentration, and unable to contribute energy to an explosion. In modeling the effects of an explosion, it was assumed that ignition occurred at a point near the center of the plume, and the equivalent energy of a TNT explosion was assumed to be released from this point. The center of the plume was estimated simply by taking one half the distance from the tank to the edge of the plume at the LEL concentration. Although it is very unlikely for a cloud this mass to develop a pressure wave, the radius of a 1 psi overpressure was calculated using the TNT equivalency method (Z = 45 for 1 psi) as 252 ft from the center of the cloud or 580 ft from the tank.

Scenario (2) is a 2" hole in the bottom of the propane tank (below the liquid level). In this circumstance, liquid will flash to vapor and liquid as it exits the tank in a foam-like state. The TSCREEN model predicted the 20,000 gallon tank would empty in 19 minutes. The propane emission rate was calculated by TSCREEN to be 33.2 kg/s. The release was modeled as 37% vapor and 63% aerosol droplets of propane at –42 ° C. As the mixture is warmed by the ground and the atmosphere, the droplets vaporize. The emissions were modeled as if the hole pointed downward minimizing plume rise. Ground level concentrations of propane were predicted to exceed the LEL to a distance of 450 meters. Based on the worst case 3 m/s wind speed and a travel distance of 450 m, the mass of the cloud was calculated as 150 sec X 33.2 kg/s, or 4980 kg (10956 lbs.) The radius of a 1 psi overpressure was calculated using the TNT equivalency method as 213 m (699 ft) from the center of the cloud, or 438 m (1437 ft) from the tank.

Scenario (3) is an instantaneous release of the entire contents of a 20,000 gallon propane tank. In this circumstance, liquid will flash to vapor and liquid as it exits the tank in a foam-like state. The release was modeled as 37% vapor and 63% aerosol droplets of propane at –42 ° C. As the mixture is warmed by the ground and the atmosphere, the droplets vaporize. The emissions were modeled as a cold dense cloud of propane gas and droplets being transported downwind by the wind using the SLAB model. The initial cloud dimensions are 4 meters high x 38 meters in diameter. The SLAB model requires terrain roughness as an input. A value of .0003 meters was used as suggested in the SLAB users manual for "level dessert". The SLAB model predicted a maximum concentration exceeding the LEL out to a distance of 700 meters, with a worst case wind speed of 3 m/s. Based on a cloud mass of 14000 kg (30800 lbs), or 37% of the total mass in the tank contents, the radius of a 1 psi overpressure was calculated using the TNT equivalency method as 301 m (987 ft) from the center of the cloud. The cloud at 700 m has dispersed beyond ignitable. It was assumed that the cloud was ignited half way between the release point (the tank) and the point at which it is no longer ignitable. This yielded a 1 psi overpressure 651 m (2135 ft) from the tank. At 549 m (1800 ft) the scaled ground distance, Z, was calculated as 29.7, yielding an overpressure less than 2 psi at the building.

Scenario (4) is an instantaneous release of the entire contents of a 5,000 gallon propane tank. In this circumstance, liquid will flash to vapor and liquid as it exits the tank in a foam-like state. The release was modeled as 37% vapor and 63% aerosol droplets of propane at –42 ° C. As the mixture is warmed by the ground and the atmosphere, the droplets vaporize. The emissions were modeled as a cold dense cloud of propane gas and droplets being transported downwind by the wind using the SLAB model. The initial cloud dimensions are 4 meters high x 19 meters diameter. The SLAB model requires terrain roughness as an input. A value of .0003 meters was used as suggested in the SLAB users manual for "level dessert". The SLAB model predicted a maximum concentration exceeding the LEL out to a distance of 400 meters. Based on a cloud mass of 3500 kg (7700 lbs), the radius of a 1 psi overpressure was calculated using the TNT equivalency method as 189 m (620 ft) from the center of the cloud. As with scenario 3, ignition was assumed when the cloud was halfway between the release point and the point at which it is no longer ignitable. This yielded a 1 psi overpressure radius of 389 m (1276 ft) from the tank.

Copies of the modeling run input and output files are attached. These files contain all of the modeling details. Scenarios (1) & (2) were run with the TSCREEN model. Scenarios (3) & (4) were run with the SLAB model. All concentration predictions are one second averages.

TSCREEN Model Run - Scenario (1).

```
+-------------- Continuous Leaks from Reservoir - Scenario 2.3 ----
|
| SOURCE PARAMETERS - Page 1 of 4
|
| Enter a unique title for this data's model run:
| Propane Gas Only Release - 2" hole in top of tank.
| SOURCE OF LEAK
|            Area (Ao) of Hole or Opening -> 20.3    cm²
|           Enter P for Pipe - T for tank -> T
|
| FLOW CHARACTERISTIC
|                 Critical Pressure (P*) -> 488809.3 Pa
| +-----------------------------------------------------------
| |               Gas Heat Capacity (Cp) -> 1678    J/kg °K
| |              Reservoir Pressure (P1) -> 844000  Pa
| |               Molecular Weight (Mw) -> 44.1     kg/kmol
| +-----------------------------------------------------------
|                  Flow Characteristic -> Choked
| +-----------------------------------------------------------
| |               Ambient Pressure (Pa) -> 101325   Pa
| +-----------------------------------------------------------

+-------------- Continuous Leaks from Reservoir - Scenario 2.3 ---
|
| SOURCE PARAMETERS - Page 2 of 4
|
| TEMPERATURES
|   Gas Temperature (T*) at Critical Pressure -> 275.5608 °K
| +-----------------------------------------------------------
| |            Reservoir Temperature (T1) -> 293     °K
| +-----------------------------------------------------------
|
|
|              Critical Temperature (Tc) -> 369.67   °K
| VAPOR PRESSURE
|     Vapor Pressure (Pv) at Gas Temperature -> 492357.3 Pa
| +-----------------------------------------------------------
| |  Latent Heat of Vaporization (Lvap) at Tb -> 425740  J/kg
| |        Boiling Point Temperature (Tb) -> 231      °K
| +-----------------------------------------------------------
|
|
```

```
+-------------- Continuous Leaks from Reservoir - Scenario 2.3 ----------
|
| SOURCE PARAMETERS - Page 3 of 4
|
| EMISSION RATE
|                    Emission Rate (Qm) -> 3490.567  g/s
| +-----------------------------------------------------------------
| |    Density at Reservoir Conditions (_1) -> 15.5    kg/cubic m
| +-----------------------------------------------------------------
|
| DISCHARGE CHARACTERISTICS
|              Discharge Temperature (T2) -> 278.1767 °K
|                 Discharge Density (_2) -> 1.932077 kg/cubic m
|
|
|                  Density of Air (_air) -> 1.20209  kg/cubic m
| +-----------------------------------------------------------------
| |             Ambient Temperature (Ta) -> 293      °K
| +-----------------------------------------------------------------
| Buoyancy is Negative

+-------------- Continuous Leaks from Reservoir - Scenario 2.3 ----
| SOURCE PARAMETERS - Page 4 of 4
|
| VERTICALLY DIRECTED JET
|      Does the release result in a vertically
|                     directed jet (Y/N) -> N
|
| TIME
|                  Release Duration (Td) -> 66.84683 min
| +-----------------------------------------------------------
| |   Total Amount of Material Released (Q) -> 14000    kg
| +-----------------------------------------------------------
|
|
```

10:37:54

*** B&M MODEL RUN ***

Propane Gas Only Release - 2" hole in top of tank.

INPUTS:

AMBIENT PRESSURE (ATM)	=	1.000
AMBIENT TEMP (K)	=	293.0
AVERAGING TIME (MIN)	=	.1600E-01
BOILING PT TEMP (K)	=	231.0
DURATION (S)	=	4011.
EMISSION RATE (KG/S)	=	3.491
EXIT TEMP (K)	=	278.2
MASS (KG)	=	.1400E+05
MOL. WEIGHT (G/G-MOLE)	=	44.10
RELATIVE HUMIDITY (%)	=	20.00
VAPOR FRACTION	=	1.000

****

*** SUMMARY OF B&M MODEL RESULTS ***

****

MAX CONC (UG/M**3)	MAX CONC (PPM)	DIST TO MAX (M)	WIND SPEED (M/S)
.9468E+08	.5161E+05	100.	3.

****

** REMEMBER TO INCLUDE BACKGROUND CONCENTRATIONS **

****

****

*** B&M DISTANCES ***

****

DIST (M)	CONC (UG/M**3)	CONC (PPM)	WIND SPEED (M/S)
100.	.9468E+08	.5161E+05	3.
200.	.2736E+08	.1491E+05	3.
300.	.1508E+08	8219.	1.
400.	.8206E+07	4473.	1.
500.	.4490E+07	2448.	2.
549.	.3724E+07	2030.	2.
600.	.3118E+07	1700.	2.
700.	.2291E+07	1249.	2.
800.	.1754E+07	956.1	2.
900.	.1386E+07	755.5	2.
1000.	.1123E+07	611.9	2.
1100.	.9277E+06	505.7	2.
1200.	.7795E+06	424.9	2.
1300.	.6642E+06	362.1	2.
1400.	.5727E+06	312.2	2.

1500.	.4989E+06	272.0	2.
1600.	.4385E+06	239.0	2.
1700.	.3884E+06	211.7	2.
1900.	.3109E+06	169.5	2.
2000.	.2806E+06	153.0	2.
2100.	.2545E+06	138.8	1.
2300.	.2122E+06	115.7	2.
2500.	.1796E+06	97.91	2.
2700.	.1540E+06	83.94	2.
2900.	.1335E+06	72.76	2.
3100.	.1168E+06	63.68	2.
3300.	.1031E+06	56.19	2.
3600.	.8661E+05	47.22	2.
3900.	.7380E+05	40.23	2.
4200.	.6363E+05	34.69	2.
4500.	.5543E+05	30.22	2.
5000.	.4490E+05	24.48	2.

```
 CALCULATED VALUES:
    DENSITY OF DEPRESSURIZED CONTAMINANT (KG/M**3) =     1.932
    DENSITY OF AMBIENT AIR (KG/M**3)               =     1.203
    MOLE FRACTION                                  =     1.000
    MIN DIST INST (M)                              =    .1337E+06
    MAX DIST CNST (M)                              =    .3209E+05
```

***** NOTES & DEFINITIONS *****

(a) "inst" refers to an instantaneous release (Section 3.6 of B-M Workbook)

(b) "cnst" refers to a continuous release (Section 3.6 of B-M Workbook)

(c) "MIN DIST INST" is the minimum distance downwind at which the release may be treated as instantaneous

(d) "MAX DIST CNST" is the maximum distance downwind at which the release may be treated as continuous

```
**********************************
***     END OF B&M OUTPUT      ***
**********************************
```

TSCREEN Model Run – Scenario (2)

```
+----- Continuous 2-Phase Saturated Liquid from Pressurized Storage - 3.2
|
| SOURCE PARAMETERS - Page 1 of 4
|
| Enter a unique title for this data's model run:
| Propane Leak from a 2 inch hole in a 20000 gal tank (saturated)
| SOURCE OF LEAK
|          Area (Ao) of Hole or Opening -> 20.3    cm²
|          Enter P for Pipe - T for Tank -> T
||
|
| DISCHARGE TEMPERATURE
|          Discharge Temperature (T2) -> 231     °K
| +----------------------------------------------------------------
| |              Ambient Pressure (Pa) -> 101325   Pa
| |      Boiling Point Temperature (Tb) -> 231      °K
| |    Latent Heat of Vaporization (Lvap) -> 425740   J/kg
| |              Molecular Weight (Mw) -> 44.1     kg/kmol
| +----------------------------------------------------------------

+----- Continuous 2-Phase Saturated Liquid from Pressurized Storage - 3.2
|
| SOURCE PARAMETERS - Page 2 of 4
|
| VAPOR FRACTION AFTER DEPRESSURIZATION
|  Vapor Fraction after Depressurization (X2) -> 0.366985
| +----------------------------------------------------------------
| |         Liquid Heat Capacity (Cpl) -> 2520    J/kg °K
| |         Reservoir Temperature (T1) -> 293     °K
| +----------------------------------------------------------------
|
| EMISSION RATE
|              Emission Rate (Qm) -> 33193.01 g/s
| +----------------------------------------------------------------
| |            Reservoir Pressure (P1) -> 844000   Pa
| |    Liquid Heat Capacity at T1 (Cpl) -> 2520    J/kg °K
| |     Contaminant Liquid Density (_1) -> 500     kg/cubic m
| +----------------------------------------------------------------
|
| +----------------------------------------------------------------
| |            Vapor Heat Capacity (Cp) -> 1678    J/kg °K
| +----------------------------------------------------------------
```

```
+----- Continuous 2-Phase Saturated Liquid from Pressurized Storage - 3.2
|
|  SOURCE PARAMETERS - Page 3 of 4
|
|  DISCHARGE DENSITY
|                  Discharge Density (_2) -> 6.289454 kg/cubic m
|
|  DENSITY OF AIR
|                   Density of Air (_air) -> 1.20209  kg/cubic m
|  +------------------------------------------------------------------
|  |              Ambient Temperature (Ta) -> 293     °K
|  +------------------------------------------------------------------
|  Buoyancy is Negative

+----- Continuous 2-Phase Saturated Liquid from Pressurized Storage - 3.2
|  SOURCE PARAMETERS - Page 4 of 4
|
|  VERTICALLY DIRECTED JET
|       Does the release result in a vertically
|                       directed jet (Y/N) -> N
|
|  TIME
|                    Release Duration (Td) -> 19.00501 min
|  +------------------------------------------------------------------
|  |     Total Amount of Material Released (Q) -> 37850    kg
|  +------------------------------------------------------------------
|
```

09:49:55
*** B&M MODEL RUN ***

Propane Leak from a 2 inch hole in a 20000 gal tank (saturated)

INPUTS:

AMBIENT PRESSURE (ATM)	=	1.000
AMBIENT TEMP (K)	=	293.0
AVERAGING TIME (MIN)	=	.1666E-01
BOILING PT TEMP (K)	=	231.0
DURATION (S)	=	1140.
EMISSION RATE (KG/S)	=	33.19
EXIT TEMP (K)	=	231.0
GAS HEAT CAPACITY(J/KG K)	=	1678.
LATENT HEAT (J/KG)	=	.4257E+06
MASS (KG)	=	.3785E+05
MOL. WEIGHT (G/G-MOLE)	=	44.10
RELATIVE HUMIDITY (%)	=	20.00
VAPOR FRACTION	=	.3670

****
*** SUMMARY OF B&M MODEL RESULTS ***
****

MAX CONC (UG/M**3)	MAX CONC (PPM)	DIST TO MAX (M)	WIND SPEED (M/S)
.8605E+08	.4691E+05	100.	1.

****
** REMEMBER TO INCLUDE BACKGROUND CONCENTRATIONS **
****

****
*** B&M DISTANCES ***
****

DIST (M)	CONC (UG/M**3)	CONC (PPM)	WIND SPEED (M/S)
100.	.8605E+08	.4691E+05	1.
200.	.8605E+08	.4691E+05	1.
300.	.6944E+08	.3786E+05	3.
400.	.4327E+08	.2359E+05	3.
500.	.2978E+08	.1624E+05	4.
549.	.2451E+08	.1336E+05	4.
600.	.2037E+08	.1111E+05	4.
700.	.1478E+08	8056.	3.
800.	.1184E+08	6454.	4.
900.	.9842E+07	5365.	4.
1000.	.8343E+07	4548.	4.
1100.	.7185E+07	3917.	4.
1200.	.6273E+07	3420.	3.

1300.	.5589E+07	3047.	3.
1400.	.4886E+07	2664.	4.
1500.	.4375E+07	2385.	4.
1600.	.3946E+07	2151.	4.
1700.	.3581E+07	1952.	4.
1900.	.2637E+07	1438.	4.
2000.	.2288E+07	1247.	4.
2100.	.1991E+07	1086.	4.
2300.	.1538E+07	838.2	4.
2500.	.1101E+07	600.1	3.
2700.	.9308E+06	507.4	3.
2900.	.8070E+06	439.9	3.
3000.	.7541E+06	411.1	3.
3100.	.7063E+06	385.0	3.
3300.	.6233E+06	339.8	3.
3600.	.5238E+06	285.6	3.
3900.	.4916E+06	268.0	3.
4000.	.4673E+06	254.7	3.
4200.	.4239E+06	231.1	3.
4500.	.3692E+06	201.3	3.
5000.	.2991E+06	163.0	3.
6000.	.2120E+06	115.6	3.
8000.	.1480E+06	80.67	4.
10000.	.1120E+06	61.04	5.

```
 CALCULATED VALUES:
    DENSITY OF DEPRESSURIZED CONTAMINANT (KG/M**3) =     1.749
    DENSITY OF AMBIENT AIR (KG/M**3)               =     1.203
    MOLE FRACTION                                  =     .2785
    MIN DIST INST (M)                              =     .3801E+05
    MAX DIST CNST (M)                              =     9122.


 ***** NOTES & DEFINITIONS *****
(a) "inst" refers to an instantaneous release (Section 3.6 of B-M
Workbook)
(b) "cnst" refers to a continuous release (Section 3.6 of B-M Workbook)
(c) "MIN DIST INST" is the minimum distance downwind at which the
release
    may be treated as instantaneous
(d) "MAX DIST CNST" is the maximum distance downwind at which the
release
    may be treated as continuous

 *********************************
 ***     END OF B&M OUTPUT     ***
 *********************************
```

SLAB Model Run – Scenario (3)

```
Propane Cloud (Instantaneous Release) from a 20,000 gal tank.
Emissions are for a 14,000 kg vapor cloud and 23,850 kg of liquid
droplets.
Wind Speed = 3 m/s, Stab = F, Zo = .0003 m. Initial Cloud Area = 1500
sq. m.

problem input

  idspl =         4
  ncalc =         1
  wms   =   .044100
  cps   =   1678.00
  tbp   =    231.00
  cmed0 =       .63
  dhe   =   425740.
  cpsl  =   2520.00
  rhosl =    500.00
  spb   =     -1.00
  spc   =       .00
  ts    =    231.00
  qs    =       .00
  as    =   1500.00
  tsd   =        0.
  qtis  =  37850.00
  hs    =       .00
  tav   =      1.00
  xffm  =   5000.00
  zp(1) =       .00
  zp(2) =       .00
  zp(3) =       .00
  zp(4) =       .00
  z0    =   .000300
  za    =      3.00
  ua    =      3.00
  ta    =    293.00
  rh    =     20.00
  stab  =      6.00


release gas properties

 molecular weight of source gas (kg)                - wms   =  4.4100E-02
 vapor heat capacity, const. p.  (j/kg-k)           - cps   =  1.6780E+03
 temperature of source gas (k)                      - ts    =  2.3100E+02
 density of source gas (kg/m3)                      - rhos  =  2.3266E+00
 boiling point temperature                          - tbp   =  2.3100E+02
 liquid mass fraction                               - cmed0=   6.3000E-01
 liquid heat capacity (j/kg-k)                      - cpsl  =  2.5200E+03
 heat of vaporization (j/kg)                        - dhe   =  4.2574E+05
 liquid source density (kg/m3)                      - rhosl=   5.0000E+02
 saturation pressure constant                       - spa   =  9.7756E+00
 saturation pressure constant (k)                   - spb   =  2.2582E+03
 saturation pressure constant (k)                   - spc   =  0.0000E+00
```

```
spill characteristics

 spill type                                       - idspl=          4
 mass source rate (kg/s)                          - qs    =  0.0000E+00
 continuous source duration (s)                   - tsd   =  0.0000E+00
 continuous source mass (kg)                      - qtcs  =  0.0000E+00
 instantaneous source mass (kg)                   - qtis  =  3.7850E+04
 source area (m2)                                 - as    =  1.5000E+03
 vertical vapor velocity (m/s)                    - ws    =  0.0000E+00
 source half width (m)                            - bs    =  1.9365E+01
 source height (m)                                - hs    =  4.0447E+00
 horizontal vapor velocity (m/s)                  - us    =  0.0000E+00


field parameters

 concentration averaging time (s)                 - tav   =  1.0000E+00
 mixing layer height (m)                          - hmx   =  2.6000E+02
 maximum downwind distrace (m)                    - xffm  =  5.0000E+03
 concentration measurement height (m)             - zp(1)=  0.0000E+00
                                                  - zp(2)=  0.0000E+00
                                                  - zp(3)=  0.0000E+00
                                                  - zp(4)=  0.0000E+00


ambient meteorological properties

 molecular weight of ambient air (kg)            - wmae  =  2.8908E-02
 heat capacity of ambient air at const p. (j/kg-k)- cpaa  =  1.0084E+03
 density of ambient air (kg/m3)                  - rhoa  =  1.2024E+00
 ambient measurement height (m)                  - za    =  3.0000E+00
 ambient atmospheric pressure (pa=n/m2=j/m3)     - pa    =  1.0133E+05
 ambient wind speed (m/s)                        - ua    =  3.0000E+00
 ambient temperature (k)                         - ta    =  2.9300E+02
 relative humidity (percent)                     - rh    =  2.0000E+01
 ambient friction velocity (m/s)                 - uastr =  9.5448E-02
 atmospheric stability class value               - stab  =  6.0000E+00
 inverse monin-obukhov length (1/m)              - ala   =  1.7255E-01
 surface roughness height (m)                    - z0    =  3.0000E-04


additional parameters

 sub-step multiplier                             - ncalc =          1
 number of calculational sub-steps               - nssm  =          3
 acceleration of gravity (m/s2)                  - grav  =  9.8067E+00
 gas constant (j/mol- k)                         - rr    =  8.3143E+00
 von karman constant                             - xk    =  4.1000E-01
1
```

time averaged (tav = 1. s) volume concentration: maximum concentration (volume fraction) along centerline.

downwind distance x (m)	height z (m)	maximum concentration c(x,0,z)	time of max conc (s)	cloud duration (s)
0.00E+00	0.00E+00	1.00E+00	0.00E+00	2.41E+02
1.01E-03	0.00E+00	1.00E+00	3.11E-01	2.41E+02
4.59E-03	0.00E+00	1.00E+00	6.57E-01	2.41E+02
1.15E-02	0.00E+00	1.00E+00	1.04E+00	2.41E+02
2.29E-02	0.00E+00	1.00E+00	1.47E+00	2.41E+02
3.96E-02	0.00E+00	1.00E+00	1.95E+00	2.41E+02
6.29E-02	0.00E+00	1.00E+00	2.48E+00	2.41E+02
9.40E-02	0.00E+00	1.00E+00	3.08E+00	2.41E+02
1.34E-01	0.00E+00	1.00E+00	3.74E+00	2.41E+02
1.86E-01	0.00E+00	1.00E+00	4.47E+00	2.41E+02
2.51E-01	0.00E+00	1.00E+00	5.29E+00	2.41E+02
3.32E-01	0.00E+00	1.00E+00	6.20E+00	2.41E+02
4.35E-01	0.00E+00	1.00E+00	7.22E+00	2.41E+02
5.67E-01	0.00E+00	1.00E+00	8.35E+00	2.41E+02
7.42E-01	0.00E+00	1.00E+00	9.61E+00	2.41E+02
9.79E-01	0.00E+00	1.00E+00	1.10E+01	2.41E+02
1.31E+00	0.00E+00	1.00E+00	1.26E+01	2.41E+02
1.79E+00	0.00E+00	1.00E+00	1.43E+01	2.41E+02
2.46E+00	0.00E+00	1.00E+00	1.62E+01	2.41E+02
3.42E+00	0.00E+00	1.00E+00	1.84E+01	2.41E+02
4.73E+00	0.00E+00	1.00E+00	2.08E+01	2.41E+02
6.48E+00	0.00E+00	9.97E-01	2.35E+01	2.41E+02
8.76E+00	0.00E+00	8.78E-01	2.64E+01	2.41E+02
1.16E+01	0.00E+00	7.70E-01	2.97E+01	2.41E+02
1.52E+01	0.00E+00	6.73E-01	3.34E+01	2.41E+02
1.96E+01	0.00E+00	5.86E-01	3.75E+01	2.41E+02
2.48E+01	0.00E+00	5.10E-01	4.21E+01	2.41E+02
3.11E+01	0.00E+00	4.45E-01	4.72E+01	2.41E+02
3.87E+01	0.00E+00	3.89E-01	5.29E+01	2.41E+02
4.75E+01	0.00E+00	3.39E-01	5.92E+01	2.41E+02
5.78E+01	0.00E+00	2.96E-01	6.62E+01	2.42E+02
6.99E+01	0.00E+00	2.57E-01	7.40E+01	2.44E+02
8.39E+01	0.00E+00	2.23E-01	8.27E+01	2.46E+02
1.00E+02	0.00E+00	1.93E-01	9.24E+01	2.50E+02
1.19E+02	0.00E+00	1.66E-01	1.03E+02	2.53E+02
1.40E+02	0.00E+00	1.43E-01	1.15E+02	2.58E+02
1.65E+02	0.00E+00	1.22E-01	1.29E+02	2.63E+02
1.93E+02	0.00E+00	1.04E-01	1.44E+02	2.68E+02
2.26E+02	0.00E+00	8.81E-02	1.60E+02	2.74E+02
2.63E+02	0.00E+00	7.44E-02	1.79E+02	2.81E+02
3.06E+02	0.00E+00	6.25E-02	1.99E+02	2.89E+02
3.56E+02	0.00E+00	5.22E-02	2.22E+02	2.98E+02
4.12E+02	0.00E+00	4.34E-02	2.48E+02	3.08E+02
4.77E+02	0.00E+00	3.59E-02	2.76E+02	3.19E+02
5.51E+02	0.00E+00	2.95E-02	3.08E+02	3.30E+02
6.36E+02	0.00E+00	2.42E-02	3.43E+02	3.44E+02
7.34E+02	0.00E+00	1.97E-02	3.82E+02	3.59E+02
8.46E+02	0.00E+00	1.60E-02	4.26E+02	3.75E+02
9.75E+02	0.00E+00	1.29E-02	4.75E+02	3.94E+02
1.12E+03	0.00E+00	1.04E-02	5.29E+02	4.15E+02

1.29E+03	0.00E+00	8.32E-03	5.89E+02	4.39E+02
1.49E+03	0.00E+00	6.65E-03	6.56E+02	4.65E+02
1.71E+03	0.00E+00	5.29E-03	7.31E+02	4.95E+02
1.96E+03	0.00E+00	4.20E-03	8.14E+02	5.28E+02
2.26E+03	0.00E+00	3.33E-03	9.07E+02	5.66E+02
2.59E+03	0.00E+00	2.63E-03	1.01E+03	6.08E+02
2.97E+03	0.00E+00	2.07E-03	1.13E+03	6.55E+02
3.41E+03	0.00E+00	1.63E-03	1.25E+03	7.08E+02
3.91E+03	0.00E+00	1.29E-03	1.40E+03	7.68E+02
4.49E+03	0.00E+00	1.01E-03	1.55E+03	8.34E+02
5.14E+03	0.00E+00	7.96E-04	1.73E+03	9.09E+02

SLAB Model – Scenario (4)

```
Propane Cloud (Instantaneous Release) from a 5000 gal tank.
Emissions are for a 3500 kg vapor cloud and 5963 kg of liquid droplets.
Wind Speed = 3 m/s, Stab = F, Zo = .0003 m. Initial Cloud Area = 375
sq. m.


problem input

  idspl =        4
  ncalc =        1
  wms   =  .044100
  cps   =  1678.00
  tbp   =   231.00
  cmed0 =      .63
  dhe   =  425740.
  cpsl  =  2520.00
  rhosl =   500.00
  spb   =    -1.00
  spc   =      .00
  ts    =   231.00
  qs    =      .00
  as    =   375.00
  tsd   =       0.
  qtis  =  9463.00
  hs    =      .00
  tav   =     1.00
  xffm  =  5000.00
  zp(1) =      .00
  zp(2) =      .00
  zp(3) =      .00
  zp(4) =      .00
  z0    =  .000300
  za    =     3.00
  ua    =     3.00
  ta    =   293.00
  rh    =    20.00
  stab  =     6.00


release gas properties

 molecular weight of source gas (kg)                   - wms   =  4.4100E-02
 vapor heat capacity, const. p.  (j/kg-k)              - cps   =  1.6780E+03
 temperature of source gas (k)                         - ts    =  2.3100E+02
 density of source gas (kg/m3)                         - rhos  =  2.3266E+00
 boiling point temperature                             - tbp   =  2.3100E+02
 liquid mass fraction                                  - cmed0=  6.3000E-01
 liquid heat capacity (j/kg-k)                         - cpsl  =  2.5200E+03
 heat of vaporization (j/kg)                           - dhe   =  4.2574E+05
 liquid source density (kg/m3)                         - rhosl=  5.0000E+02
 saturation pressure constant                          - spa   =  9.7756E+00
 saturation pressure constant (k)                      - spb   =  2.2582E+03
 saturation pressure constant (k)                      - spc   =  0.0000E+00
```

```
spill characteristics

 spill type                                              - idspl=          4
 mass source rate (kg/s)                                 - qs    =  0.0000E+00
 continuous source duration (s)                          - tsd   =  0.0000E+00
 continuous source mass (kg)                             - qtcs  =  0.0000E+00
 instantaneous source mass (kg)                          - qtis  =  9.4630E+03
 source area (m2)                                        - as    =  3.7500E+02
 vertical vapor velocity (m/s)                           - ws    =  0.0000E+00
 source half width (m)                                   - bs    =  9.6825E+00
 source height (m)                                       - hs    =  4.0449E+00
 horizontal vapor velocity (m/s)                         - us    =  0.0000E+00


field parameters

 concentration averaging time (s)                        - tav   =  1.0000E+00
 mixing layer height (m)                                 - hmx   =  2.6000E+02
 maximum downwind distrace (m)                           - xffm  =  5.0000E+03
 concentration measurement height (m)                    - zp(1)=  0.0000E+00
                                                         - zp(2)=  0.0000E+00
                                                         - zp(3)=  0.0000E+00
                                                         - zp(4)=  0.0000E+00


ambient meteorological properties

 molecular weight of ambient air (kg)                   - wmae   =  2.8908E-02
 heat capacity of ambient air at const p. (j/kg-k)- cpaa   =  1.0084E+03
 density of ambient air (kg/m3)                         - rhoa   =  1.2024E+00
 ambient measurement height (m)                         - za     =  3.0000E+00
 ambient atmospheric pressure (pa=n/m2=j/m3)            - pa     =  1.0133E+05
 ambient wind speed (m/s)                               - ua     =  3.0000E+00
 ambient temperature (k)                                - ta     =  2.9300E+02
 relative humidity (percent)                            - rh     =  2.0000E+01
 ambient friction velocity (m/s)                        - uastr  =  9.5448E-02
 atmospheric stability class value                      - stab   =  6.0000E+00
 inverse monin-obukhov length (1/m)                     - ala    =  1.7255E-01
 surface roughness height (m)                           - z0     =  3.0000E-04


additional parameters

 sub-step multiplier                                    - ncalc  =           1
 number of calculational sub-steps                      - nssm   =           3
 acceleration of gravity (m/s2)                         - grav   =  9.8067E+00
 gas constant (j/mol- k)                                - rr     =  8.3143E+00
 von karman constant                                    - xk     =  4.1000E-01
1

time averaged (tav =    1. s) volume concentration:  maximum
concentration (volume fraction) along centerline.


    downwind                        maximum    time of      cloud
```

distance x (m)	height z (m)	concentration c(x,0,z)	max conc (s)	duration (s)
0.00E+00	0.00E+00	1.00E+00	0.00E+00	1.57E+02
1.82E-03	0.00E+00	1.00E+00	3.11E-01	1.57E+02
8.24E-03	0.00E+00	1.00E+00	6.57E-01	1.57E+02
2.04E-02	0.00E+00	1.00E+00	1.04E+00	1.57E+02
3.90E-02	0.00E+00	1.00E+00	1.47E+00	1.57E+02
6.48E-02	0.00E+00	1.00E+00	1.95E+00	1.57E+02
9.84E-02	0.00E+00	1.00E+00	2.48E+00	1.57E+02
1.41E-01	0.00E+00	1.00E+00	3.08E+00	1.57E+02
1.94E-01	0.00E+00	1.00E+00	3.74E+00	1.57E+02
2.61E-01	0.00E+00	1.00E+00	4.47E+00	1.57E+02
3.49E-01	0.00E+00	1.00E+00	5.29E+00	1.57E+02
4.70E-01	0.00E+00	1.00E+00	6.20E+00	1.57E+02
6.44E-01	0.00E+00	1.00E+00	7.22E+00	1.57E+02
9.01E-01	0.00E+00	1.00E+00	8.35E+00	1.57E+02
1.29E+00	0.00E+00	1.00E+00	9.61E+00	1.57E+02
1.84E+00	0.00E+00	1.00E+00	1.10E+01	1.57E+02
2.62E+00	0.00E+00	1.00E+00	1.26E+01	1.57E+02
3.66E+00	0.00E+00	9.08E-01	1.43E+01	1.57E+02
5.01E+00	0.00E+00	7.92E-01	1.62E+01	1.57E+02
6.71E+00	0.00E+00	6.90E-01	1.84E+01	1.57E+02
8.83E+00	0.00E+00	6.00E-01	2.08E+01	1.57E+02
1.14E+01	0.00E+00	5.22E-01	2.35E+01	1.57E+02
1.45E+01	0.00E+00	4.56E-01	2.64E+01	1.57E+02
1.83E+01	0.00E+00	3.98E-01	2.97E+01	1.57E+02
2.28E+01	0.00E+00	3.48E-01	3.34E+01	1.57E+02
2.81E+01	0.00E+00	3.03E-01	3.75E+01	1.57E+02
3.43E+01	0.00E+00	2.64E-01	4.21E+01	1.57E+02
4.15E+01	0.00E+00	2.29E-01	4.72E+01	1.58E+02
4.99E+01	0.00E+00	1.98E-01	5.29E+01	1.59E+02
5.96E+01	0.00E+00	1.71E-01	5.92E+01	1.61E+02
7.09E+01	0.00E+00	1.47E-01	6.62E+01	1.63E+02
8.39E+01	0.00E+00	1.26E-01	7.40E+01	1.65E+02
9.90E+01	0.00E+00	1.08E-01	8.27E+01	1.68E+02
1.16E+02	0.00E+00	9.16E-02	9.24E+01	1.71E+02
1.36E+02	0.00E+00	7.76E-02	1.03E+02	1.75E+02
1.59E+02	0.00E+00	6.54E-02	1.15E+02	1.79E+02
1.85E+02	0.00E+00	5.49E-02	1.29E+02	1.83E+02
2.16E+02	0.00E+00	4.59E-02	1.44E+02	1.89E+02
2.50E+02	0.00E+00	3.81E-02	1.60E+02	1.94E+02
2.90E+02	0.00E+00	3.16E-02	1.79E+02	2.01E+02
3.36E+02	0.00E+00	2.60E-02	1.99E+02	2.08E+02
3.88E+02	0.00E+00	2.14E-02	2.22E+02	2.16E+02
4.48E+02	0.00E+00	1.75E-02	2.48E+02	2.26E+02
5.17E+02	0.00E+00	1.42E-02	2.76E+02	2.36E+02
5.96E+02	0.00E+00	1.15E-02	3.08E+02	2.48E+02
6.86E+02	0.00E+00	9.29E-03	3.43E+02	2.61E+02
7.90E+02	0.00E+00	7.47E-03	3.82E+02	2.76E+02
9.08E+02	0.00E+00	5.99E-03	4.26E+02	2.93E+02
1.04E+03	0.00E+00	4.79E-03	4.75E+02	3.12E+02
1.20E+03	0.00E+00	3.82E-03	5.29E+02	3.34E+02
1.38E+03	0.00E+00	3.04E-03	5.89E+02	3.58E+02
1.58E+03	0.00E+00	2.41E-03	6.56E+02	3.85E+02
1.82E+03	0.00E+00	1.91E-03	7.31E+02	4.15E+02
2.08E+03	0.00E+00	1.51E-03	8.14E+02	4.50E+02
2.39E+03	0.00E+00	1.19E-03	9.07E+02	4.88E+02

2.74E+03	0.00E+00	9.38E-04	1.01E+03	5.32E+02
3.14E+03	0.00E+00	7.39E-04	1.13E+03	5.80E+02
3.59E+03	0.00E+00	5.81E-04	1.25E+03	6.35E+02
4.11E+03	0.00E+00	4.57E-04	1.40E+03	6.95E+02
4.70E+03	0.00E+00	3.59E-04	1.55E+03	7.63E+02
5.38E+03	0.00E+00	2.82E-04	1.73E+03	8.38E+02