ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS

STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

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

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS

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

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

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

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

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

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

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CALCULATION IDENTIFICATION NUMBER				PAGE 104
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

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TABLE 1

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

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

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TABLE 2.6-6

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Static Loads

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	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	3,757	0	0	0	0	0.0	0.0	13.08	**4.36**	0.0	0.0	30.0	67.0	**1.87**	**7.0**
IB - Static Effective Strength	3,757	0	0	0	0	0.0	0.0	29.22	**9.73**	0.0	0.0	30.0	67.0	**1.87**	**15.6**

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

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

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

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

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

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

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

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

F_V = Vertical load (Static + EQ_V)

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

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

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

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

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

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

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TABLE 2.6-7

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

0.711 $g = a_H$

0.695 $g = a_V$

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

TABLE 2.6-8

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

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

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

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

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

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

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

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

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

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

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

F_V = Vertical load (Static + EQ_V)

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

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

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

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

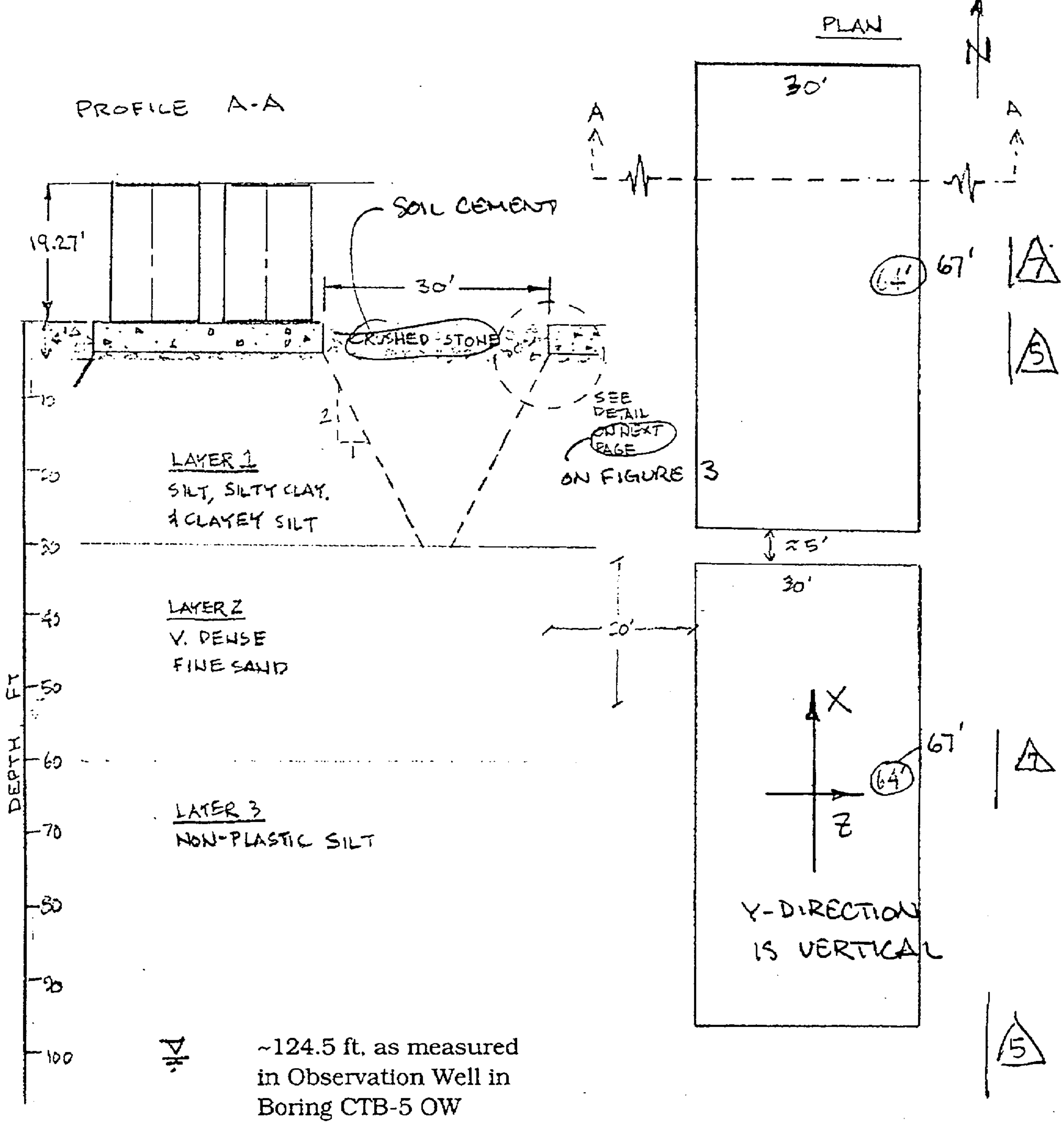
$B' = B - 2\,e_B$ $\quad$ $L' = L - 2\,e_L$

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

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

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

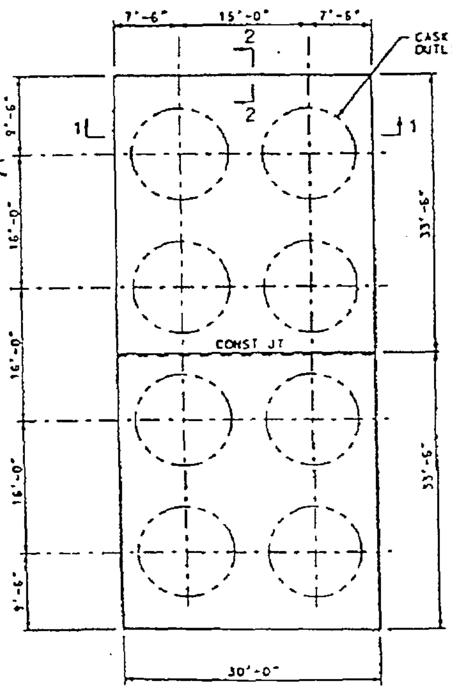
STATIC FOUNDATION LOAD / PRESSURE

Total Load

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

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

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



PLAN
CASK STORAGE PAD
(500 REQD)

Bearing Pressure:

$P_{actual} = \text{Load}/\text{Area}$

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

$P_{actual} = 1.87$ KSF

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

FIGURE 3

DETAIL OF SOIL CEMENT UNDER & ADJACENT TO CASK STORAGE PADS

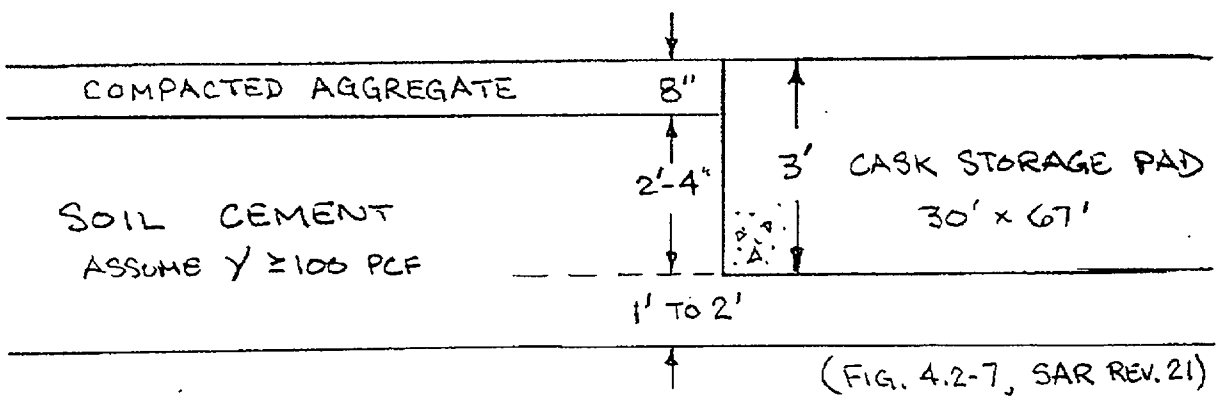
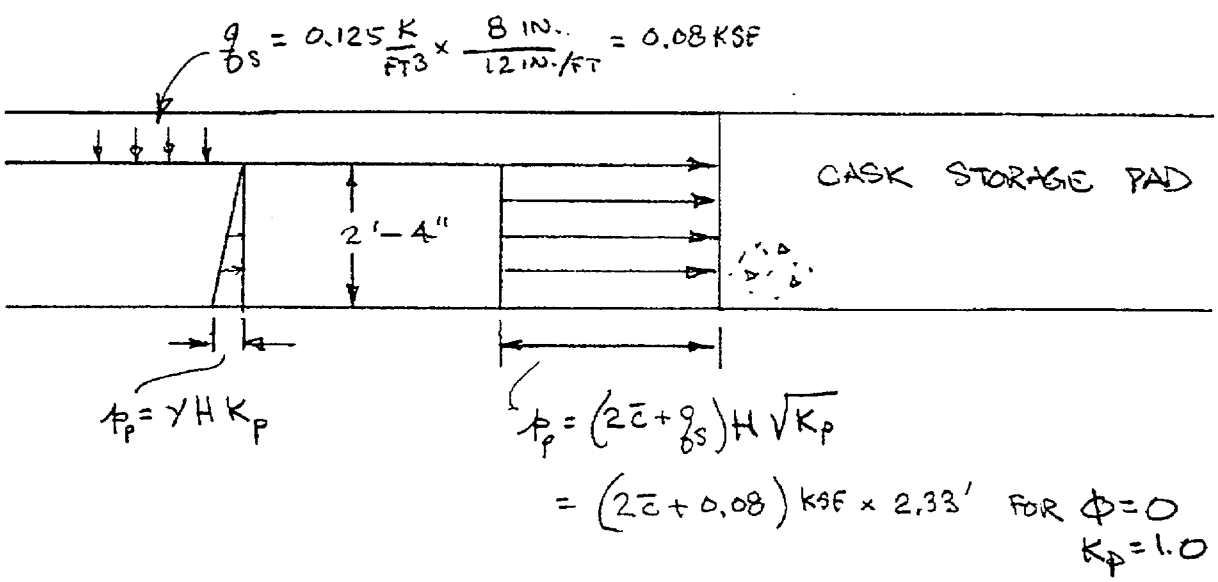


FIGURE 4

PASSIVE PRESSURE ACTING ON CASK STORAGE PADS



STONE & WEBSTER, INC.

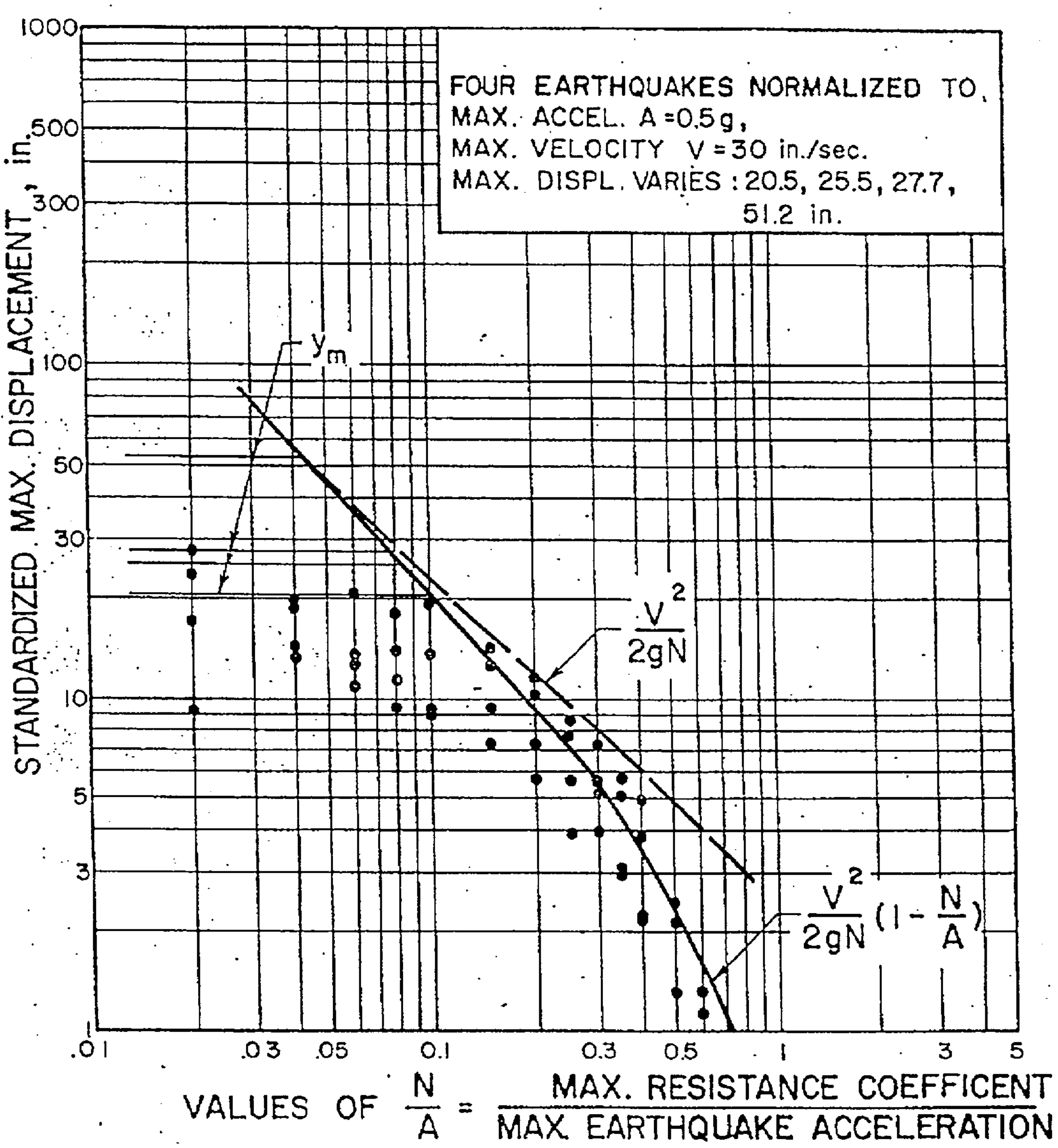
CALCULATION SHEET

5010.65

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

FIGURE 5

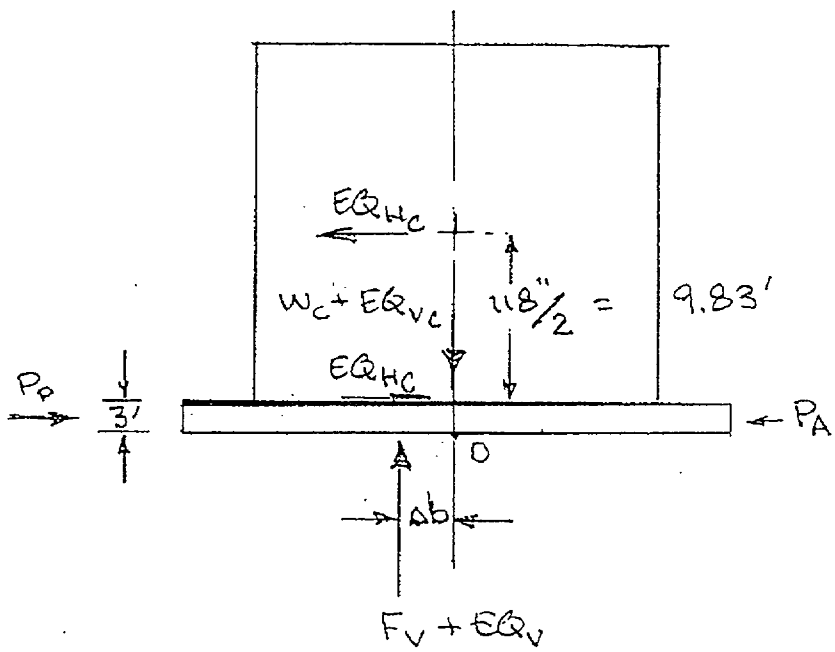
STANDARDIZED DISPLACEMENT FOR NORMALIZED EARTHQUAKES (SYMMETRICAL RESISTANCE)



From Newmark (1965)

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.

$\sum M_{@\,centerline}$ to find Δb.

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

$\sum M_{@O}$ 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

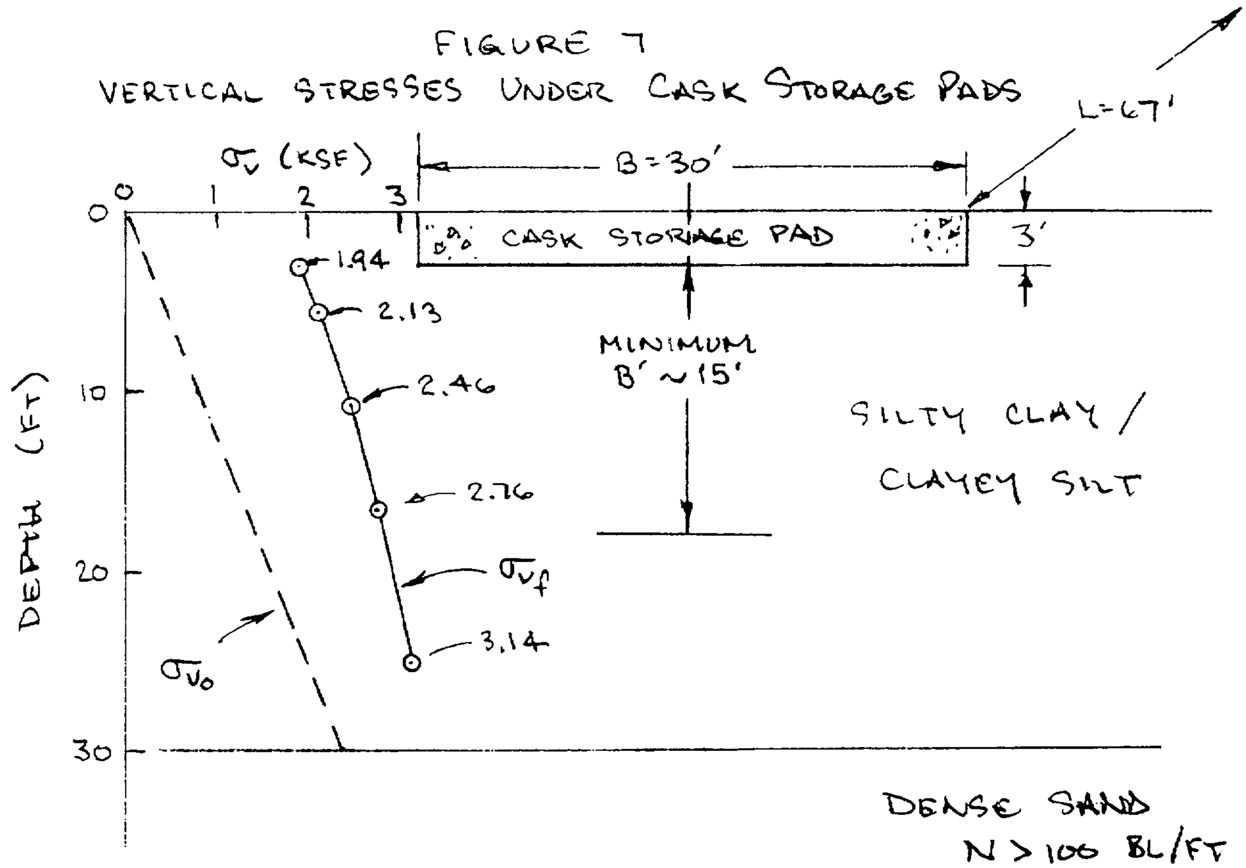
CALCULATION SHEET

▲ 5010.65

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

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:

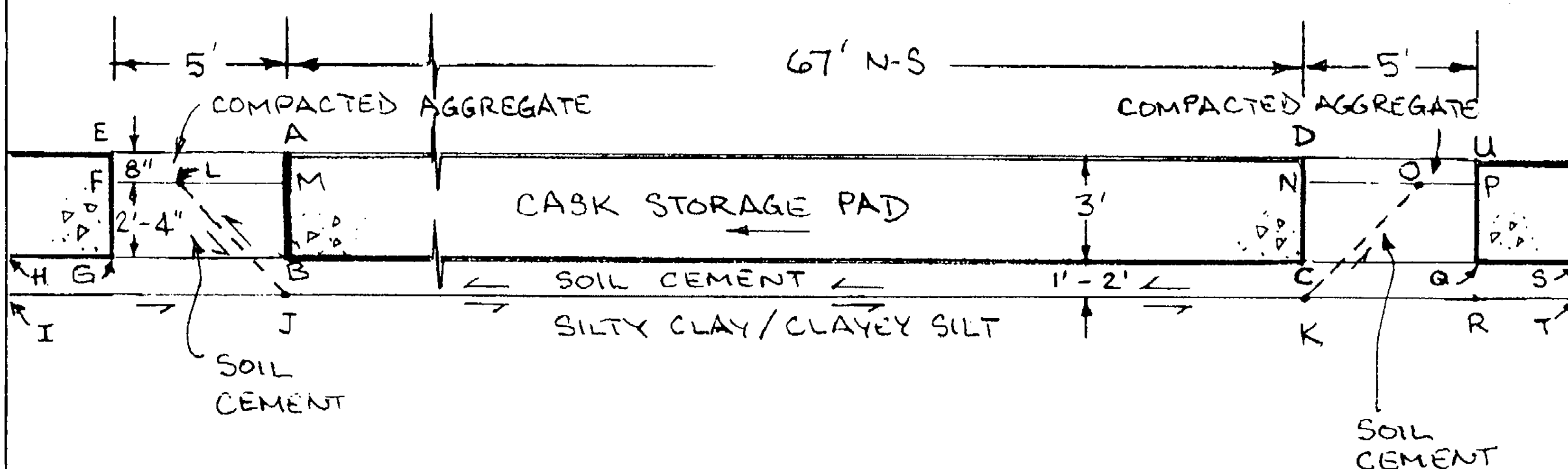
TABLE 3 => σ_{v_f} AS $f(z)$

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

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

FIGURE 8

ELEVATION VIEW OF COLUMN OF CASK STORAGE PADS - LOOKING EAST



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

NOTES OF TELEPHONE CONVERSATION

JO No. **05996.01**

PRIVATE FUEL STORAGE, LLC
PRIVATE FUEL STORAGE FACILITY

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

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

SUBJECT: DYNAMIC BEARING CAPACITY OF PAD

DISCUSSION:

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

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

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

SUPERSEDED BY ATT B 3

ACTION ITEMS:

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

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

~~00001~~

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	3
ORIGINATOR	[illegible]	DATE 3/27/01	CHECKED [illegible]	DATE	4-5-01
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	227

5.3 Soil Pressures

5.3.1 Static Soil Pressure

Calculations of static soil pressure due to dead load (DL) and cask live load (LL) are given in Table S-1 and S-2, respectively.

CEC

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	3
ORIGINATOR	W	DATE 3/27/01	CHECKED [illegible]	DATE	4-5-01
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	228

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

	k_s = 2.75 kcf	k_s = 26.2 kcf
Z_w(ft) =	0.164	0.017
q_{zw}(ksf) =	0.45	0.45

Notes:
1. Z_w = maximum vertical d splacement due to dead load (wt. of the pad only) obtained from CECSAP analysis results.
2. q_{zw} = vertical soil bearing pressure = k_s x Z_w, where k_s = subgrade modulus=2.75 and 26.2 kcf for lower-bound and upper-bound soils,respectively.

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				CALC. NO.	G(PO17)-2	REV. NO.	3
ORIGINATOR	[signature]	DATE	3/27/01	CHECKED	[signature]	DATE	4-5-01
PROJECT	Private Fuel Storage Facility					JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design					SHEET	229

Table S-2
Maximum Vertical Displacements and Soil Bearing Pressures
Live Load

Node No.	(Z_i)max ($\times 10^{-2}$ ft.)							
	subgrade modulus = 2.75 kcf				subgrade modulus = 26.2 kcf			
	2 Casks	4 Casks	8 Casks	7 Casks + OLT	2 Casks	4 Casks	8 Casks	7 Casks + OLT
1	13.06	11.29	-50.97	-57.81	0.61	1.16	-4.83	-5.30
7	13.02	11.28	-50.97	-41.84	0.59	1.14	-4.84	-4.42
13	13.06	11.29	-50.97	-25.83	0.61	1.16	-4.83	-3.50
144	-11.82	-26.36	-52.73	-78.21	-0.70	-2.89	-5.78	-7.95
150	-11.93	-26.35	-52.71	-61.06	-0.76	-2.89	-5.79	-6.31
156	-11.82	-26.36	-52.71	-43.87	-0.70	-2.89	-5.78	-4.65
287	-42.54	-62.26	-50.97	-100.20	-5.13	-5.98	-4.83	-11.81
293	-42.59	-62.25	-50.97	-80.88	-5.16	-5.98	-4.84	-8.48
299	-42.54	-62.26	-50.97	-61.84	-5.13	-5.98	-4.83	-5.47
	Maximum Soil Bearing Pressure q_{zi} (1) (ksf)							
1	0	0	-1.402	-1.590	0	0	-1.264	-1.390
7	0	0	-1.402	-1.151	0	0	-1.267	-1.159
13	0	0	-1.402	-0.710	0	0	-1.264	-0.917
144	-0.325	-0.725	-1.450	-2.151	-0.185	-0.757	-1.514	-2.082
150	-0.328	-0.725	-1.450	-1.679	-0.199	-0.758	-1.516	-1.653
156	-0.325	-0.725	-1.450	-1.206	-0.185	-0.757	-1.514	-1.219
287	-1.170	-1.712	-1.402	-2.756	-1.345	-1.567	-1.264	-3.094
293	-1.171	-1.712	-1.402	-2.224	-1.352	-1.565	-1.267	-2.222
299	-1.170	-1.712	-1.402	-1.701	-1.345	-1.567	-1.264	-1.434

Notes:
1. $q_{zi} = k_s \times Z_i$ where k_s = 2.75 and 26.2 kcf for lower-bound and upper-bound subgrade moduli, respectively, and Z_i are obtained from CECSAP analysis results (Att. A)
2. Negative displacements imply downward movements.
3. The locations of nodes listed are shown in Figure 5.1-1.
4. For snow load, the soil bearing pressures is .045 ksf (Ref. 11).

CALCULATION SHEET

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

5.3.2 Dynamic Horizontal and Vertical Soil Pressures

Calculations of lateral and vertical soil pressures due to dynamic cask loadings resulting from 2000-year event earthquake are given in the following tables:

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

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

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

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

CALCULATION SHEET

			CALC. NO.	G(PO17)-2	REV. NO. 3
ORIGINATOR		DATE 3/28/01	CHECKED		DATE 4-5-01
PROJECT	Private Fuel Storage Facility				JOB NO. 1101-000
SUBJECT	Storage Pad Analysis and Design				SHEET 231

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

Node No.	Maximum Displacement Xd ($x10^{-3}$ ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	3.512	2.409	17.160	1.624	1.177	9.076	0.798	0.547	3.597
7	3.515	2.405	17.180	1.625	1.170	9.085	0.801	0.552	3.625
13	3.512	2.409	17.190	1.624	1.177	9.060	0.799	0.550	3.618
144	4.461	9.712	17.460	2.021	4.241	9.127	1.017	2.325	3.952
150	4.461	9.729	17.470	2.021	4.242	9.156	0.999	2.294	3.951
156	4.467	9.733	17.470	2.029	4.244	9.171	0.982	2.272	3.947
287	12.800	21.490	17.510	6.201	9.504	8.860	3.345	5.306	4.514
293	12.800	21.490	17.530	6.186	9.512	8.886	3.360	5.341	4.566
299	12.800	21.470	17.530	6.173	9.516	8.886	3.381	5.349	4.565
Avg =	6.925	11.205	17.389	3.278	4.976	9.034	1.720	2.726	4.037
Kxd =	1.14E+05	1.14E+05	1.14E+05	2.33E+05	2.33E+05	2.33E+05	5.48E+05	5.48E+05	5.48E+05
Qxd =	789	1277	1982	764	1159	2105	943	1494	2212

Notes:

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

(Kxd)LB =	9.51E+06 lb/in	(Kxd)BE =	1.94E+07 lb/in	(Kxd)UB =	4.57E+07 lb/in
	1.14E+05 Kips/ft		2.33E+05 Kips/ft		5.48E+05 Kips/ft

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

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	3
ORIGINATOR		DATE 3/28/01	CHECKED	DATE	4-5-01
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	232

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

Node No.	Max. Displacement Yd (x10^{-3} ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	5.107	8.657	13.550	2.194	4.059	8.393	1.413	2.578	3.979
7	3.916	7.318	14.030	2.055	4.313	8.173	1.195	1.962	4.056
13	4.303	7.097	14.510	2.567	4.664	7.937	1.337	2.161	4.109
144	5.231	8.763	13.450	2.332	4.187	8.430	1.513	2.714	3.975
150	3.946	7.447	13.960	2.122	4.429	8.132	1.267	2.133	4.042
156	4.379	7.207	14.450	2.690	4.767	7.834	1.442	2.301	4.121
287	5.389	8.870	27.260	2.449	4.357	8.396	1.651	2.821	3.926
293	4.016	7.584	13.840	2.253	4.556	8.048	1.464	2.380	4.013
299	4.476	7.253	14.370	2.877	4.846	7.795	1.657	2.334	4.097
Avg =	4.529	7.800	15.491	2.393	4.464	8.126	1.438	2.376	4.035
Kyd =	1.08E+05	1.08E+05	1.08E+05	2.21E+05	2.21E+05	2.21E+05	5.21E+05	5.21E+05	5.21E+05
Qyd =	491	846	1680	528	986	1794	749	1237	2102

Notes:

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

(Kyd)LB =	9.04E+06 lb/in	(Kyd)BE =	1.84E+07 lb/in	(Kyd)UB =	4.34E+07 lb/in
	1.08E+05 Kips/ft		2.21E+05 Kips/ft		5.21E+05 Kips/ft

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

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	3
ORIGINATOR		DATE 3/28/01	CHECKED	DATE	4-5-01
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	273

Table D-1(c)
Summary of Total Maximum Horizontal Soil Reactions
Dynamic Load

	Max. Soil Reaction (Kips)									
	LB			BE			UB			
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	
Qxd =	789	1277	1982	764	1159	2105	943	1494	2212	E-W
Qyd =	491	846	1680	528	986	1794	749	1237	2102	N-S

Notes:
1. Qxd, and Qyd shown are obtained from Tables D-1(a), and (b), respectively.
2. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	3
ORIGINATOR	[signature]	DATE 3/28/01	CHECKED [signature]	DATE	4-5-01
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	234

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

Node No.	LB 2 Casks	LB 4 Casks	LB 8 Casks	BE 2 Casks	BE 4 Casks	BE 8 Casks	UB 2 Casks	UB 4 Casks	UB 8 Casks
	Maximum Displacement Zd ($x10^{-3}$ ft.)								
1	4.051	9.396	-31.02	1.806	4.158	-23.66	0.406	1.654	-15.92
7	3.900	7.973	-24.23	1.964	3.648	-21.18	0.439	1.024	-13.36
13	4.788	11.470	-31.22	2.115	4.636	-17.88	0.528	1.560	-15.31
144	-9.195	-22.56	-34.05	-5.939	-16.84	-22.66	-1.861	-8.34	-13.66
150	-5.063	-15.2	-12.71	-3.683	-11.13	-12.39	-1.332	-6.698	-8.016
156	-6.565	-15.9	-32.24	-2.988	-9.447	-18.42	-1.734	-5.773	-14.53
287	-29.18	-24.39	-17.51	-14.54	-15.67	-18.88	-12.72	-8.52	-8.38
293	-15.57	-16.97	-19.21	-9.019	-12.42	-12.22	-12.08	-10.68	-6.446
299	-21.85	-26.09	-28.04	-12.87	-16.35	-17.02	-9.835	-11.63	-13.12
	Maximum Soil Bearing Pressure q_{zd} (Kips/ft^2)								
1	0	0	-2.22	0	0	-3.35	0	0	-5.14
7	0	0	-1.74	0	0	-3.00	0	0	-4.32
13	0	0	-2.24	0	0	-2.53	0	0	-4.94
144	-0.66	-1.62	-2.44	-0.84	-2.38	-3.21	-0.60	-2.69	-4.41
150	-0.36	-1.09	-0.91	-0.52	-1.57	-1.75	-0.43	-2.16	-2.59
156	-0.47	-1.14	-2.31	-0.42	-1.34	-2.61	-0.56	-1.86	-4.69
287	-2.09	-1.75	-1.25	-2.06	-2.22	-2.67	-4.11	-2.75	-2.71
293	-1.12	-1.22	-1.38	-1.28	-1.76	-1.73	-3.90	-3.45	-2.08
299	-1.57	-1.87	-2.01	-1.82	-2.31	-2.41	-3.18	-3.76	-4.24

Notes:

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

(Kzd)LB =	1.20E+07 lb/in	(Kzd)BE =	2.37E+07 lb/in
	1.44.E+05 Kips/ft		2.84.E+05 Kips/ft

(Kzd)UB = 5.41E+07 lb/in; 6.49.E+05 Kips/ft

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

6.2 Vertical Soil Bearing Pressures and Horizontal Soil Shear Stresses

Vertical soil bearing pressures for individual loadings and combined loadings are Summarized in Table 4.

Horizontal soil shear stresses are shown in Tables D-1(a) and (b), and the total horizontal soil reactions (shear forces) in both the short (x) and long (y) directions of the pad are summarized in Table D-1(c).

CALCULATION SHEET

			CALC. NO.	G(PO17)-2	REV. NO. 3
ORIGINATOR		DATE 3/27/01	CHECKED		DATE 4-05-01
PROJECT	Private Fuel Storage Facility				JOB NO. 1101-000
SUBJECT	Storage Pad Analysis and Design				SHEET 26

Table 4
Summary of Vertical Soil Bearing Pressures (ksf)

		A	B	C	D	E	F	G	H	J
Loading	Point	287	293	299	144	150	156	1	7	13
2 - Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.345	1.352	1.345	0.185	0.199	0.185	0	0	0
	Pad EQ	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EQ	4.11	3.9	3.18	0.84	0.52	0.56	0	0	0
	100% Vert	6.26	6.06	5.33	1.83	1.53	1.55	0.81	0.81	0.81
4-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.712	1.712	1.712	0.757	0.758	0.757	0	0	0
	Pad EQ	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EQ	2.75	3.45	3.76	2.69	2.16	1.86	0	0	0
	100% Vert	5.27	5.97	6.28	4.25	3.73	3.42	0.81	0.81	0.81
8-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.402	1.402	1.402	1.514	1.516	1.514	1.402	1.402	1.402
	Pad EQ	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313	0.313
	Cask EQ	2.71	2.08	4.24	4.41	2.59	4.69	5.14	4.32	4.94
	100% Vert	4.92	4.29	6.45	6.73	4.91	7.01	7.35	6.53	7.15

Notes:
1. Values for Pad DL are obtained from Table S-1.
2. Values for snow LL are obtained from Table S-2.
3. Values for Cask LL are obtained from Table S-2.
4. Pad EQ pressure = (pad wt.)xa_v, where pad wt.=904.5 kips, and a_v=.695g.
5. Values for Cask EQ are obtained from Table D-1(d).
6. EQ pressures listed are the envelopes of results for all soil conditions.
7. Node numbers are shown in Figure 5.1-1.

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CALCULATION SHEET

		CALC. NO. G(PO17)-2	REV. NO. 3
ORIGINATOR	DATE 3/14/01	CHECKED	DATE 3/30/01
PROJECT Private Fuel Storage Facility			JOB NO. 1101-000
SUBJECT Storage Pad Analysis and Design			SHEET NO. 20

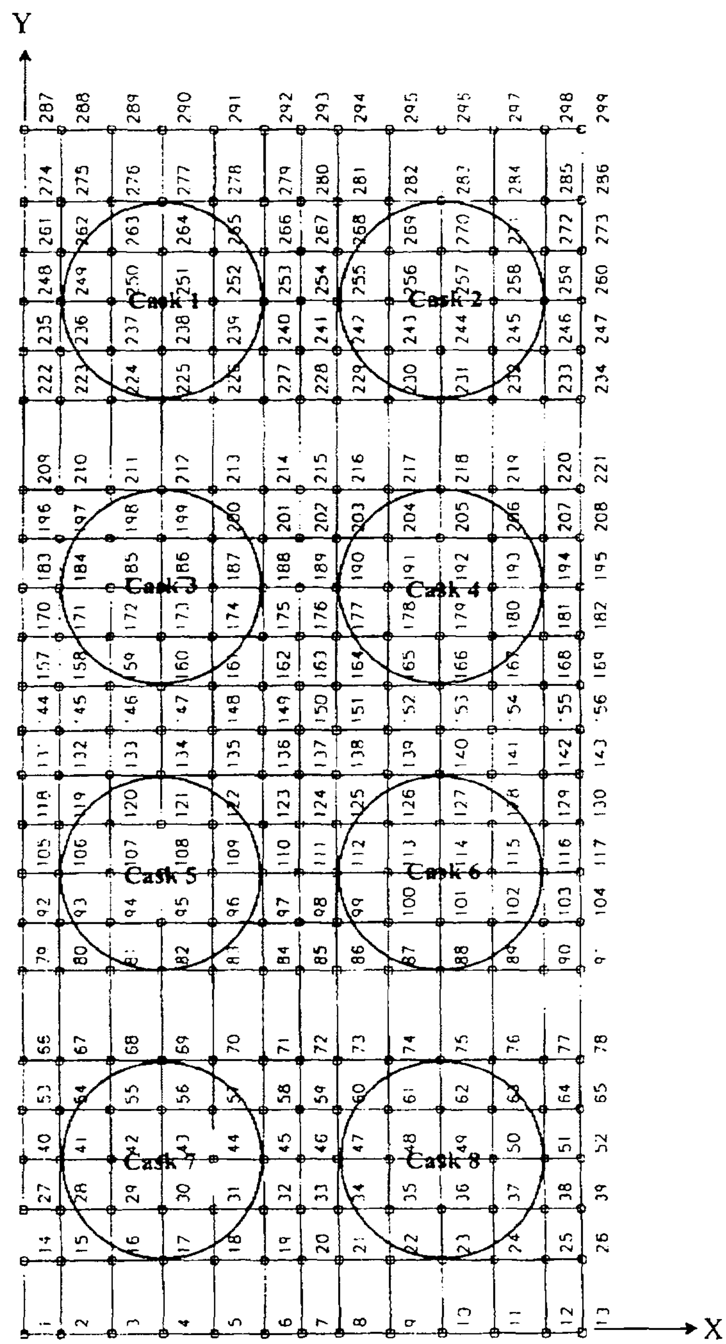

Figure 5.1-1 CECSAP Finite-Element Model with Node Numbers

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		CALC NO. G(PO17)-2	REV. NO. 3
ORIGINATOR	DATE 3/14/01	CHECKED	DATE 3/10/01
PROJECT Private Fuel Storage Facility			JOB NO. 1101-000
SUBJECT Storage Pad Analysis and Design			SHEET NO. 21

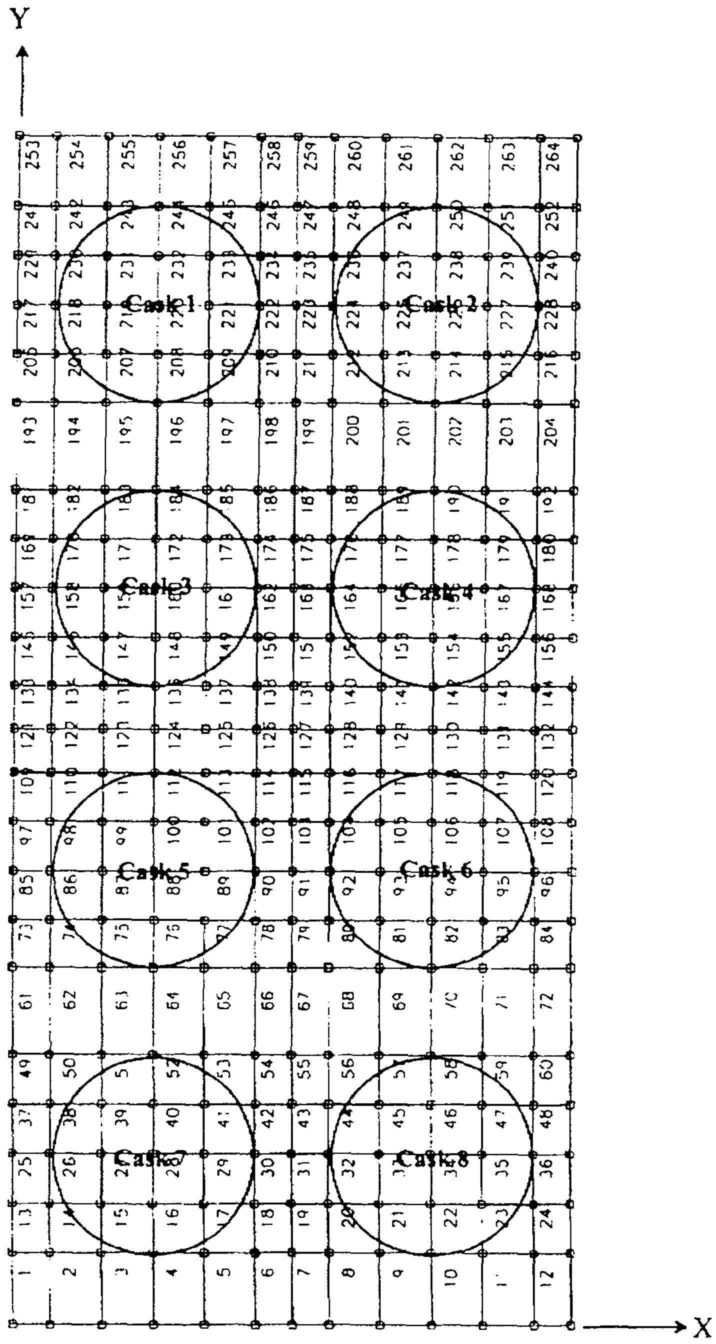


Figure 5.1-2 CECSAP Finite-Element Model with Element Numbers

CALCULATION COVER SHEET

CEC

PROJECT	Private Fuel Storage Facility (PFSF)
SUBJECT	Storage Pad Analysis and Design
JOB NO.	1101-000
FILE NO.	
CALC NO.	G(PO17)-2
NO. OF SHEETS	289

RECORD OF ISSUES

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

☒ Nuclear Quality Assurance Category ☐ Non-Nuclear Quality Assurance Category

This set of calculations documents the engineering analyses and detailed calculations required for structural design of the reinforced-concrete spent-fuel cask storage pads to be constructed at the Private Fuel Storage Facility (PFSF) project site.

This set of calculations has been prepared in accordance with CEC's quality assurance procedure for nuclear projects.

Revision 1 was made to correct (1) typographical errors on Pages 5, 29, and A-3 and (2) insert computer output file names and explanation notes on Pages 43 and 51.

Revision 2 was made to correct typographical errors and to include additional clarifications on Pages 17, 21, 28, 236, 298, and 312.

NAME		INITIAL	SIGNATURE
Anwar Mirza	(Preparer/Checker)	aum	Anwar Mirza
Donald Hamasaki	(Preparer/Checker)	DH	Donald Hamasaki
Ming S. Yang	(Preparer/Checker)	my	[signature]
Kiat Lilhanand	(Preparer/Checker)	kl	K. Lilhanand
Wen S. Tseng	(Independent Reviewer)	WT	W. S. Tseng

CALCULATION COVER SHEET

		JOB NO.	1101-000
PROJECT	Private Fuel Storage Facility (PFSF)	FILE NO.	
SUBJECT	Storage Pad Analysis and Design	CALC NO.	G(PO17)-2
		SHEET	ii

Revision 3 was made to incorporate the following: (1) PGA of 0.711g and 0.695g for horizontal and vertical components of the new design ground motions, (2) Revised dynamic soil properties for lower-bound, best-estimate, and upper-bound soils provided by Geomatrix, (3) Revised cask force time-histories provided by Holtec, (4) Revised pad size to 30 ft by 67 ft with cask spacing in the long axis of the pad changed to 16 ft and cask spacing in the short axis of the pad remained at 15 ft, (5) Pad founded in soil cement with about 3 ft under the pad and 2 ft thick on its side walls, and (6) Revised transporter weight to 145 kips.

TABLE 6

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

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

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

[geot]\05996\lab\triax-1.xls on 6/13/2000

ATTACHMENT C p C1/3
CALC 05996.02-G(B)-04-9

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

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

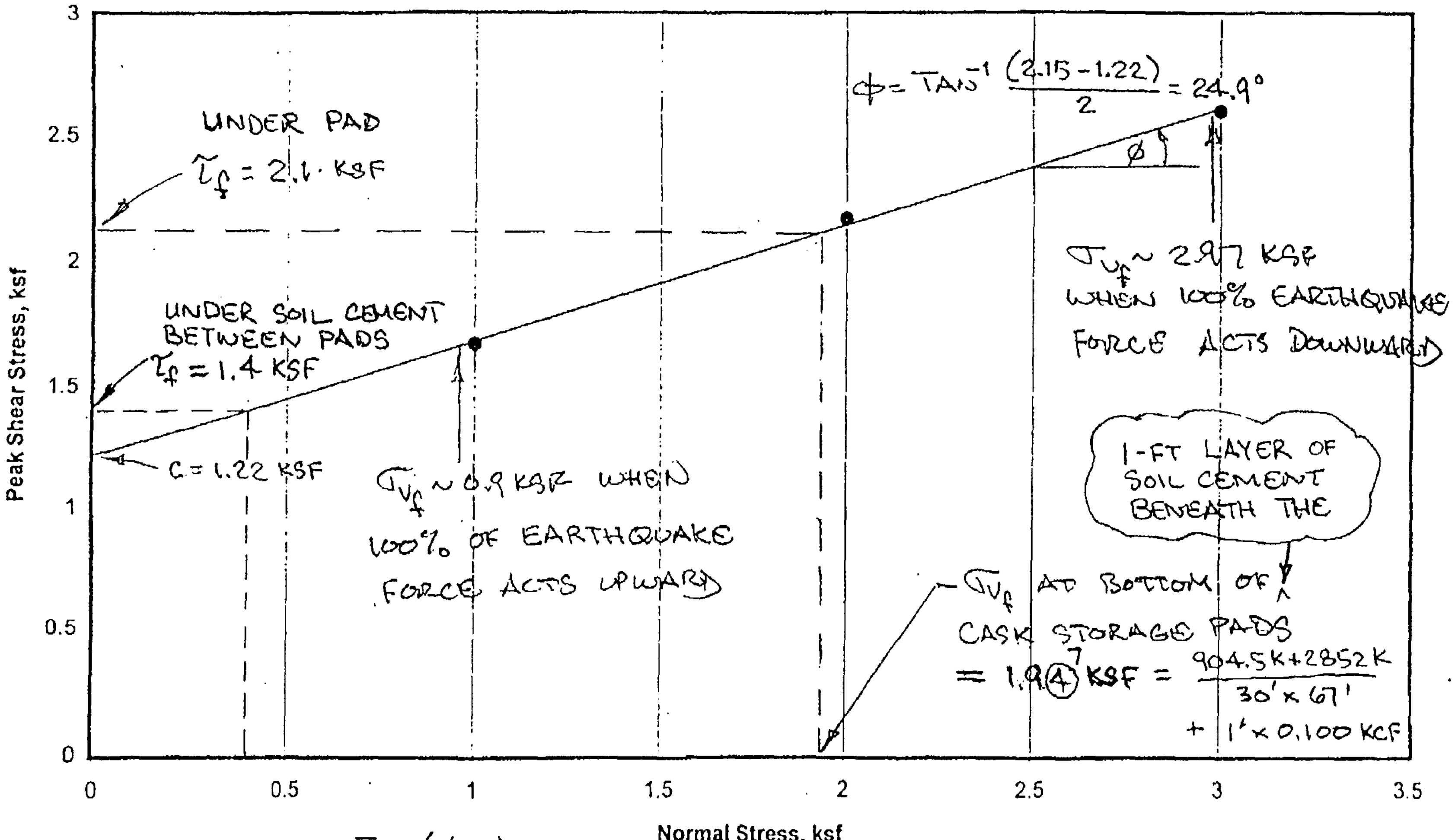
5010.65

FIGURE 7

DIRECT SHEAR TEST

Boring C-2, Sample U-1C

PAD EMPLACEMENT AREA



BETWEEN PADS $\bar{\sigma}_V \sim (3' + 1')\ 0.100\ KCF$
$= 0.4\ KSF$

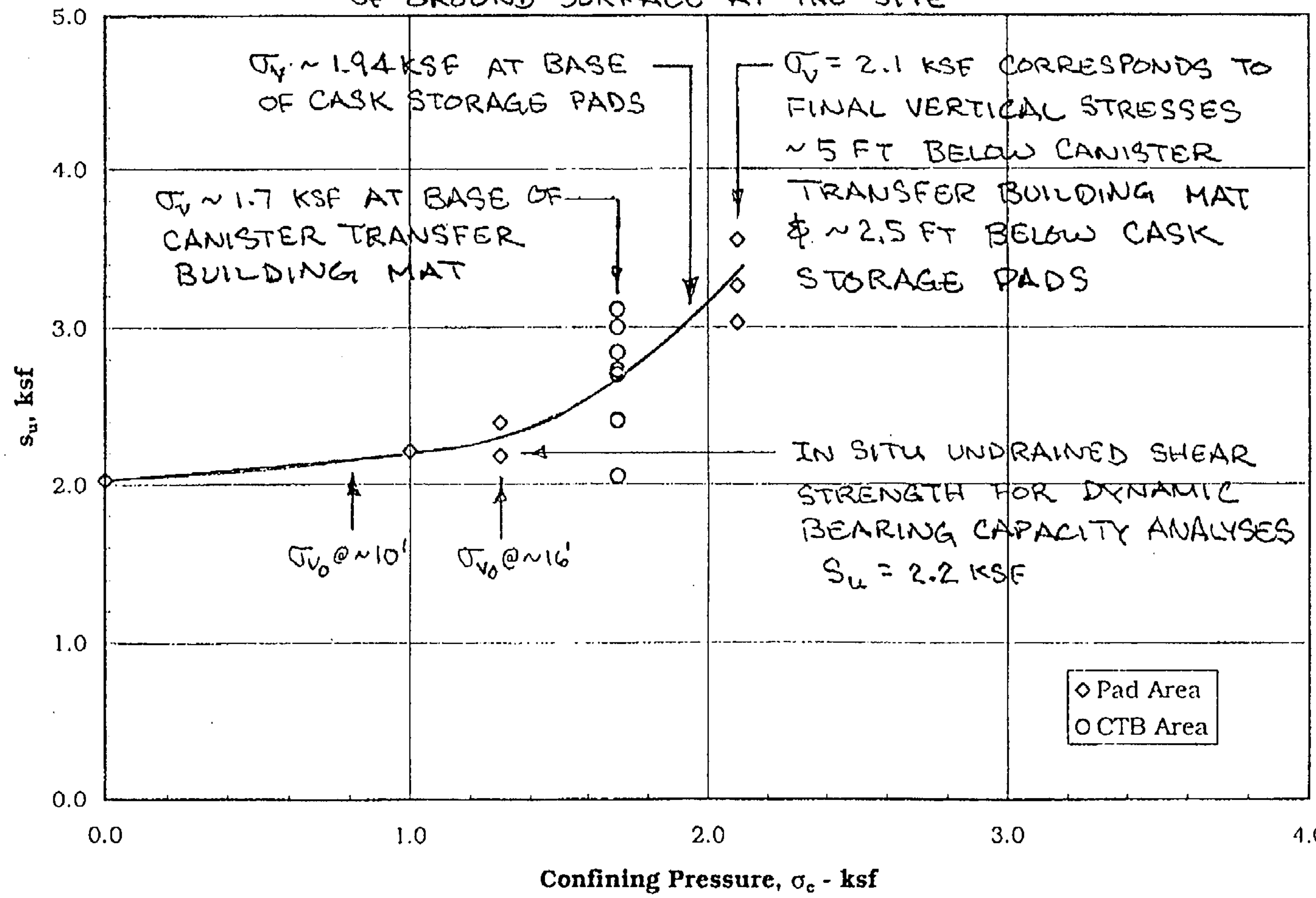
REF SAR APP 2A ATT 7

ATTACHMENT C p C2
CALC 05996.02-G(B)-04-9

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

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE
05996.02	G(B)	05-2		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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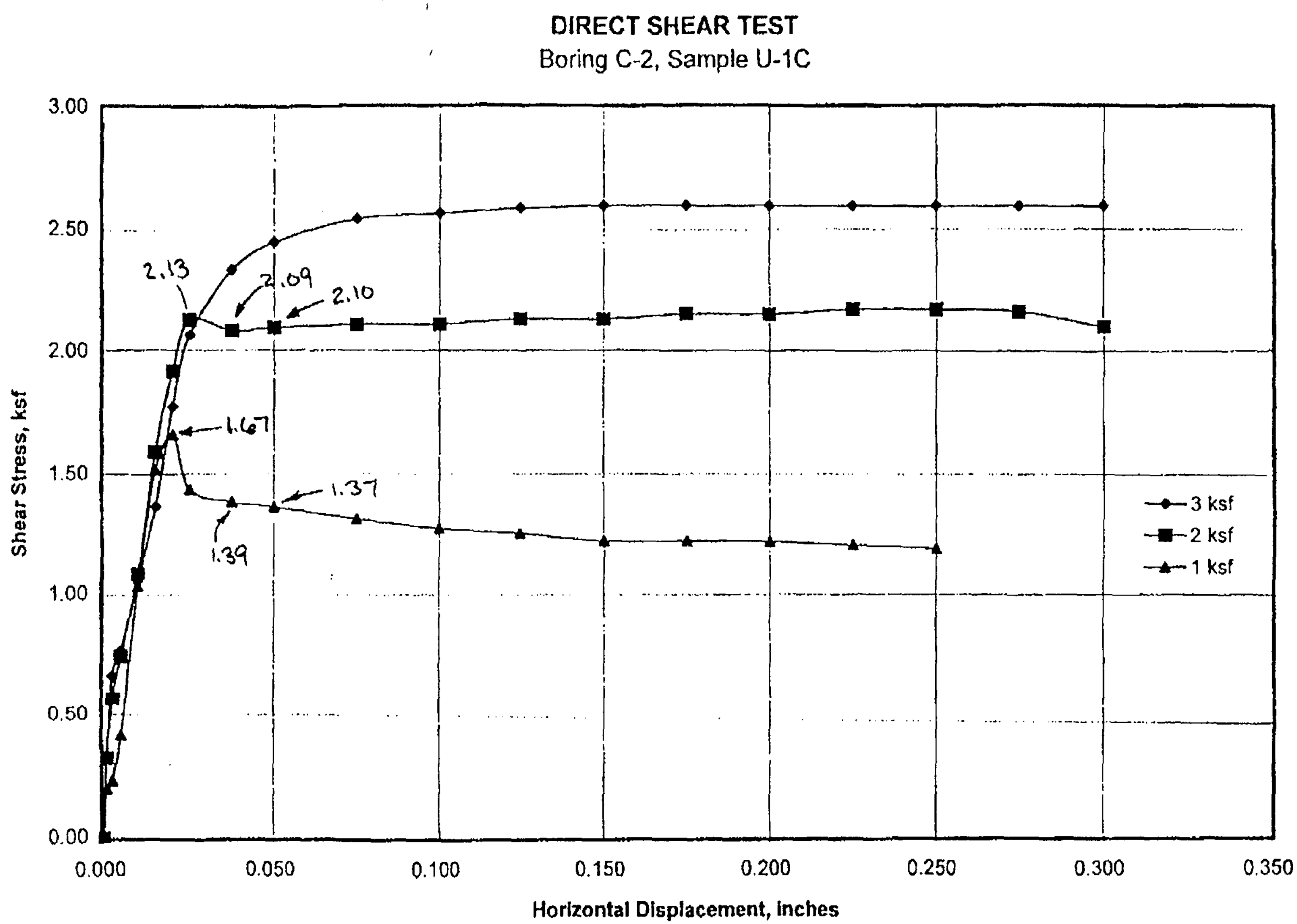
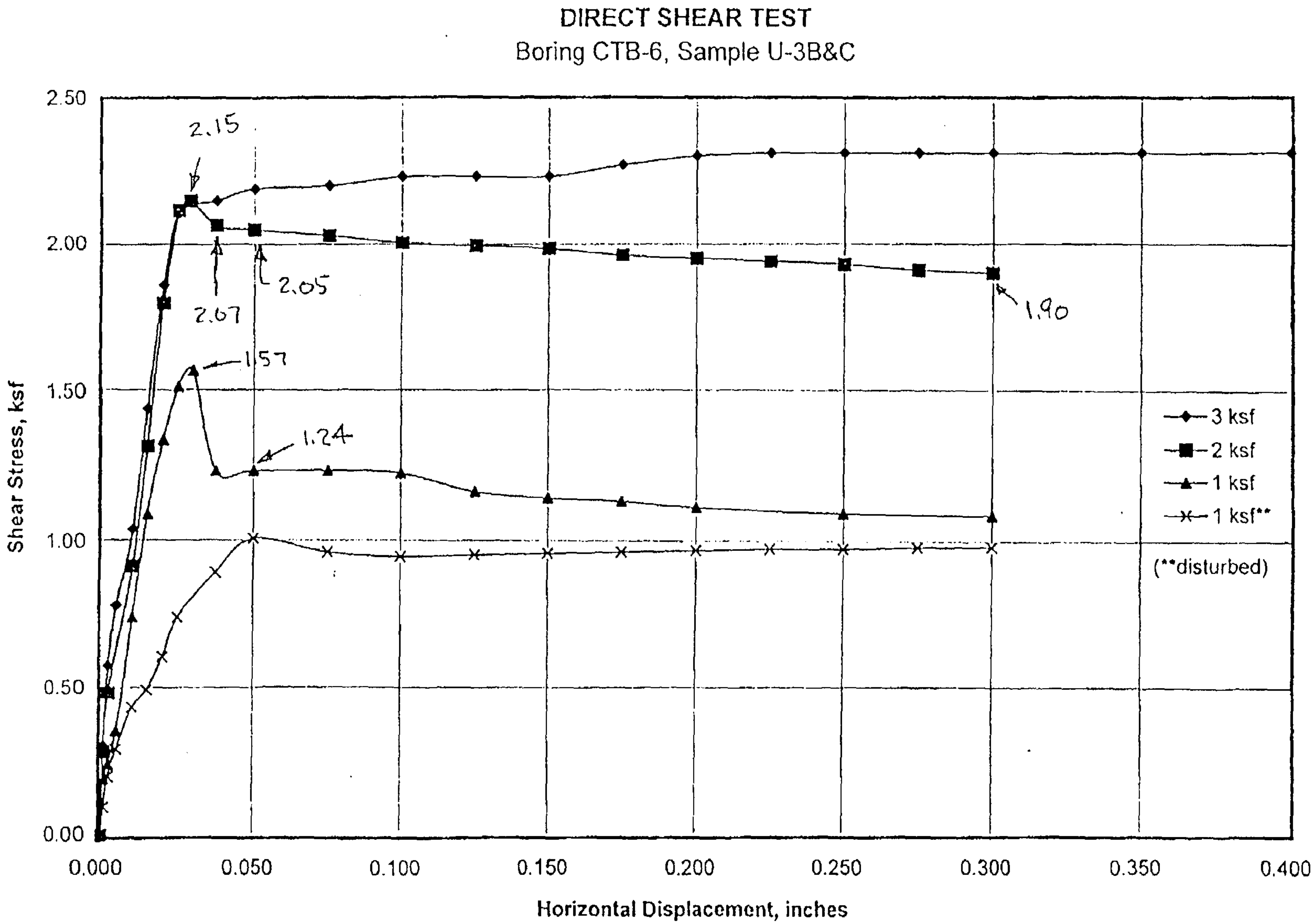
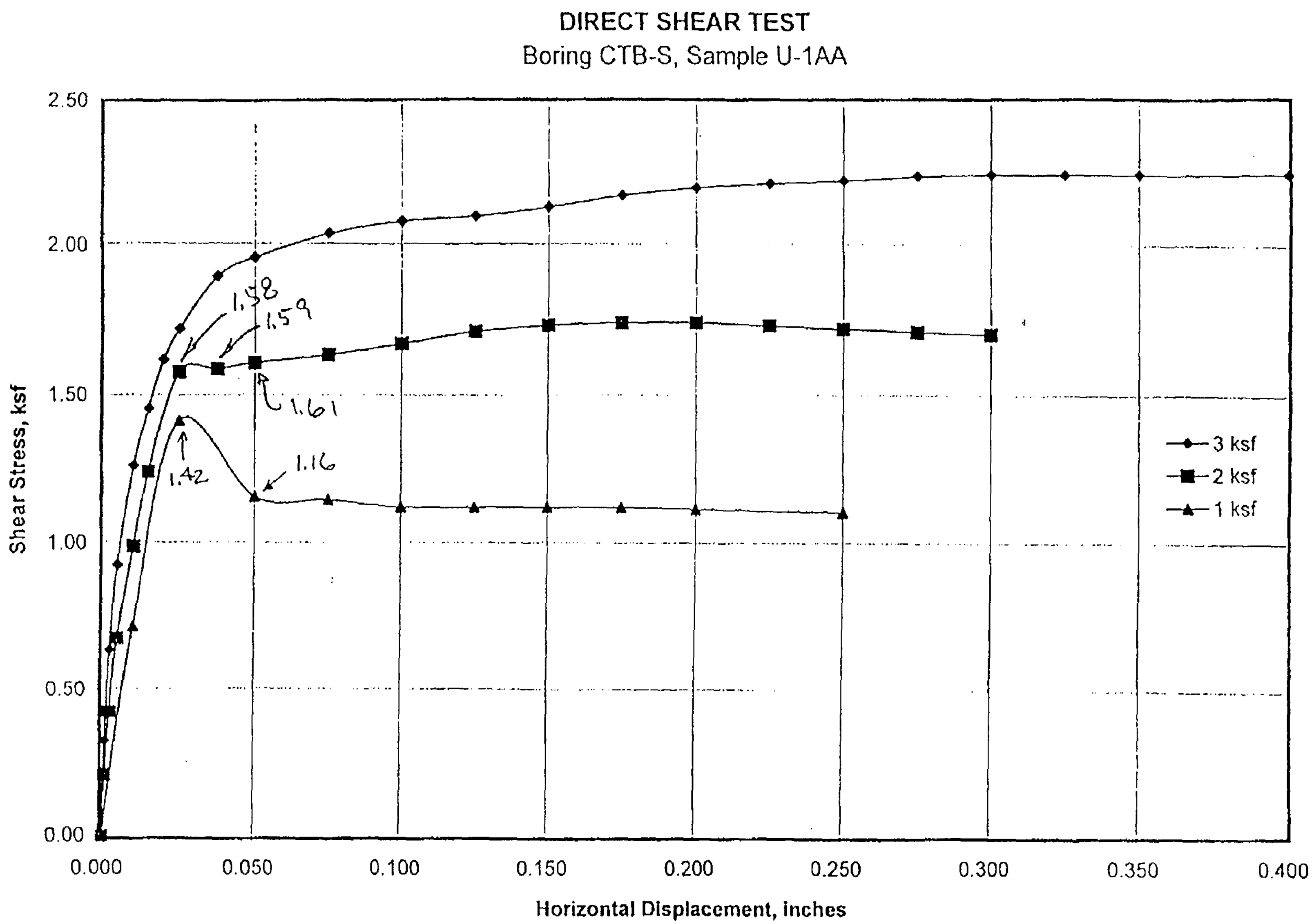
ATTACHMENT C p C3/3
CALC 05996.02-G(B)-04-9



ATTACHMENT D TO CALC 05996.02-G(8)-4-9 p D1/3

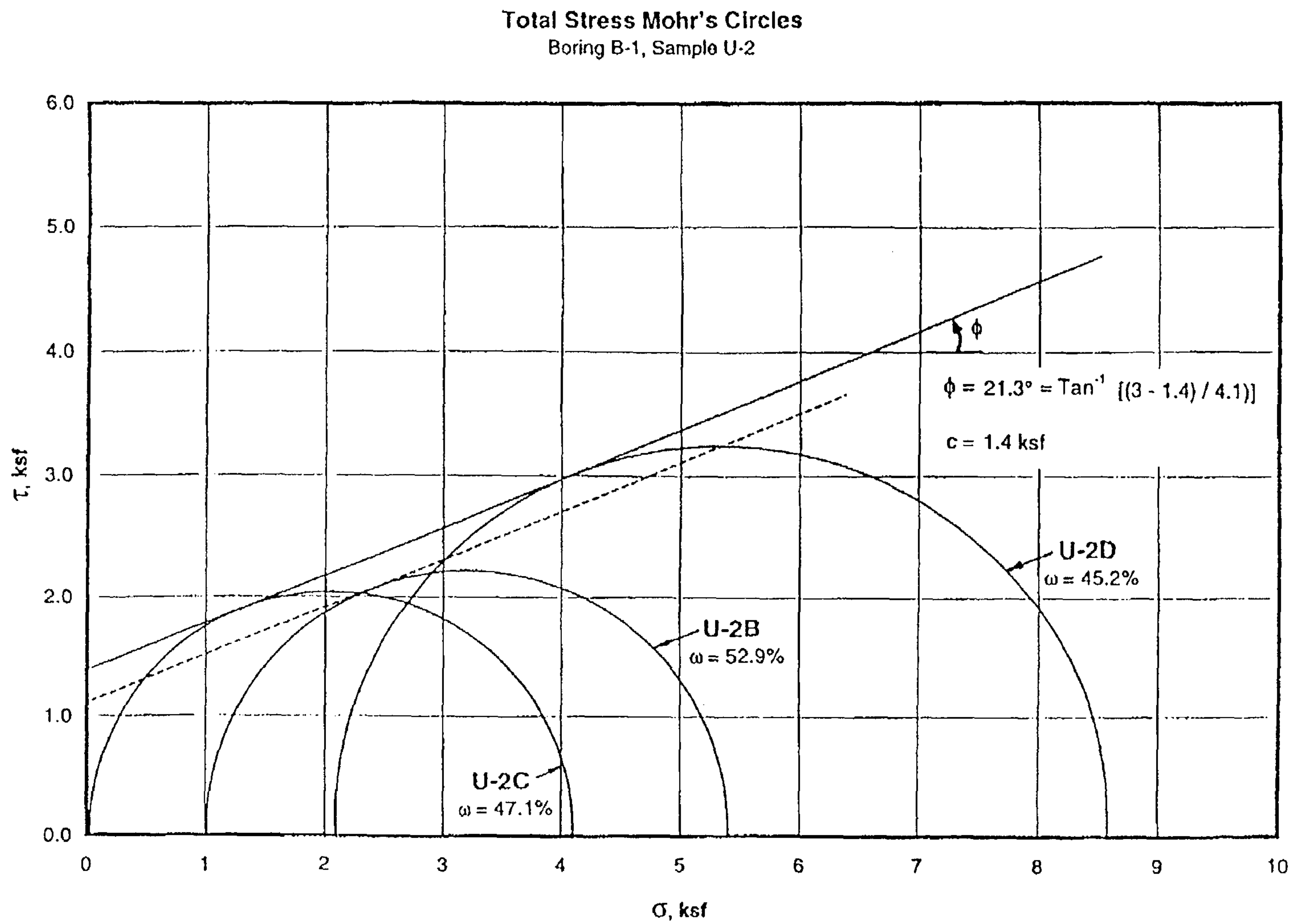


ATTACHMENT D TO CALC 05996.02-G(B)-04-9 p D2



ATTACHMENT D TO CALC 05996.02-G(B)-04-9 p D3

STONE & WEBSTER ENGINEERING CORPORATION



Private Fuel Storage, LLC
PFSF, Skull Valley, UT
SAR APP 2A ATT 8

ATTACHMENT E TO CALC 05996.02-G(B)-04-9 p E1 of 1

JO 05996.02
November 1999

STONE & WEBSTER, INC.

5010.64

CALCULATION SHEET

CLIENT & PROJECT	PAGE 1 OF 59
PRIVATE FUEL STORAGE, LLC – PFSF	+ 6 pp of ATTACHMENTS

CALCULATION TITLE	QA CATEGORY (✓)
STABILITY ANALYSES OF CANISTER TRANSFER BUILDING	☑ I NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ (other)

CALCULATION IDENTIFICATION NUMBER				
JOB ORDER NO.	DISCIPLINE	CURRENT CALC NO	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	**G(B)**	**13**		

APPROVALS - SIGNATURE & DATE			REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED ☒	
PREPARER(S)/DATE(S)	REVIEWER(S)/DATES(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
Original Signed By: LPSingh / 12-9-98	Original Signed By: DLAloysius / 12-10-98	Original Signed By: DLAloysiu / 12-10-98	0	N/A		✓
Original Signed By: DLAloysius / 9-3-99 SYBoakye / 9-3-99 *See page 2-1 for ID of*	Original Signed By: SYBoakye / 9-3-99 DLAloysius / 9-3-99 *Prepared / Reviewed By*	Original Signed By: TYChang / 9-3-99 TYChang / 9-3-99	1	G(C)-13 Rev. 0		✓
Original Signed By: PJTrudeau / 1-21-00	Original Signed By: TYChang / 1-21-00	Original Signed By: TYChang / 1-21-00	2	1		✓
Original Signed By: PJTrudeau / 6-19-00	Original Signed By: TYChang / 6-19-00	Original Signed By: TYChang / 6-19-00	3	2		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	4	3		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	5	4		✓
PJTrudeau / 7-26-01 Paul J. Trudeau	TYChang / 7-26-01 Thomas Y. Chang	TYChang / 7-26-01 Thomas Y. Chang	6	5		✓

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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 2.6-11 and incorporated revised seismic loads in calculations of overturning stability and dynamic bearing capacity.
5. Revised dynamic bearing capacity analyses to utilize only total strength parameters because these partially saturated soils will not have time to drain fully during the rapid cycling associated with the design basis ground motion. See Calculation 05996.02-G(B)-05-1 (SWEC, 1999b) for additional details.
6. Updated references to current issues of drawings.

7. Added references to foundation profiles through Canister Transfer Building area presented in SAR Figures 2.6-21 through 23.
8. Deleted analyses of bearing capacity on layered profile, as adequate factors of safety are obtained conservatively assuming that the total strengths measured for the clayey soils in the upper ~25' to 30' layer apply for the entire profile under the Canister Transfer Building and revised all of the detailed bearing capacity analyses.
9. Changed "Load Combinations" to "Load Cases" and defined these cases to be consistent throughout the various stability analyses included herein. These are the same cases as are used in the stability analyses of the cask storage pads, Calculation 05996.02-G(B)-04-5 (SWEC, 2000).
10. Added analysis of sliding on a deep plane at the top of silty sand/sandy silt layer, incorporating passive resistance acting on the block of clayey soil and the foundation mat overlying this interface.
11. Revised Conclusions to reflect results of these changes.

REVISION 3

1. Added a 1-ft deep key around the perimeter of the Canister Transfer Building mat to permit use of the cohesive strength of the in situ silty clay/clayey silt in resisting sliding due to loads from the design basis ground motion.
2. Revised shear strength used in the sliding stability analyses of the Canister Transfer Building mat supported on the in situ silty clay to be the strength measured in the direct shear tests performed on samples obtained from elevations approximately at the bottom of the 1-ft deep perimeter key. The shear strength used in this analysis equaled that measured for stresses corresponding to the vertical stresses at the bottom of the mat following completion of construction.
3. Removed static and dynamic bearing capacity analyses based on total-stress strengths.
4. The relative strength increase noted for the deeper lying soils in the cone penetration testing that was performed within the Canister Transfer Building footprint was used to determine a weighted average undrained strength of the soils in the entire upper layer for use in the bearing capacity analyses, since the soils within a depth equal to approximately the width of the foundation are effective in resisting bearing failures. This resulted in the average undrained strength for the bearing capacity analyses of the upper layer equal to 3.18 ksf.
5. Removed dynamic analyses based on increasing strengths of the cohesive soils that were measured in static tests to reflect well known phenomenon that the strength of cohesive soils increases as the rate of loading decreases.
6. Revised undrained shear strength of the clay block overlying the cohesionless layer to 2.2 ksf, based on the UU tests that were performed at confining pressures of 1.3 ksf (reported in Attachment 2 of Appendix 2A of the SAR) in the analysis of sliding of the Canister Transfer Building on deep plane of cohesionless soils.

7. Added shearing resistance available on the ends of the block of clay, since this soil must be sheared along these planes in order for the Canister Transfer Building to slide on a deep plane of cohesionless soils.
8. Revised method of calculating the inclination factor in the bearing capacity analyses to that presented by Vesic in Chapter 3 of Winterkorn and Fang (1975). Vesic's method expands upon the theory developed by Hansen for plane strain analyses of footings with inclined loads. 0Vesic's method permits a more rigorous analysis of inclined loads acting in two directions on rectangular footings, which more closely represents the conditions applicable for the Canister Transfer Building.
9. Replaced Tables 2, 2.6-9, and 2.6-10 with revised results for the changes in shear strength of the in situ soils noted above and deleted Table 3.

REVISION 4

1. Updated stability analyses to reflect revised design basis ground motions (a_H = 0.711g & a_V = 0.695g, per Table 1 of Geomatrix, 2001).
2. Resisting moment in overturning stability analysis calculated based on resultant of static and dynamic vertical forces.
3. Updated dimensions of foundation mat to 240 ft (E-W) x 279.5 ft (N-S), and changed the depth of the perimeter key to 1.5 ft, in accordance with design change identified in Figure 4.7-1 (3 sheets), "Canister Transfer Building," of SAR Revision 21 (based on S&W Drawings 0599602-EC-404A-B & 404B-B).
4. Added definition of "m" used in the inclination factors for calculating allowable bearing capacity.
5. Updated references to supporting calculations.
6. Updated discussions and conclusions to incorporate revised results.

REVISION 5

1. Shear strength of clayey soils beneath the building for resisting sliding was changed from 1.8 ksf to 1.7 ksf to reflect lower final effective stresses under the mat after changing size of mat to 240 ft x 279.5 ft.
2. Added sliding analysis that includes both shear resistance along bottom of the plane of the clayey soils enclosed within the perimeter key at the base of the mat and the full passive resistance from the soil cement placed adjacent to the mat. Used residual strength measured in the direct shear tests that were performed on these clayey soils for this case.

REVISION 6

1. Expanded description of soil cement properties.
2. Added discussion to clarify use of peak strengths measured in the direct shear tests along with one-half of passive resistance and residual strengths along with full passive resistance in sliding stability analysis.
3. Added calculation of horizontal displacement of the building due to elastic theory.
4. Expanded discussion of residual strengths of the clayey soils underlying the building.

OBJECTIVE

To determine the stability against overturning, sliding, and static and dynamic bearing capacity failure of the Canister Transfer Building supported on a mat foundation.

ASSUMPTIONS/DATA

The footprint of the Canister Transfer Building foundation mat is shown on SAR Figure 4.7-1, "Canister Transfer Building," and S&W Drawing 0599602-EC-404A-B & 404B-B, Canister Transfer Building - Conc Mat Foundation Plan, Sheets 1 & 2. The elevation view of the structure is shown on Sheets 2 & 3 of SAR Figure 4.7-1. The foundation mat is 240 ft (E-W) x 279.5 ft (N-S) x 5 ft thick, with a 6.5-ft wide x 1.5-ft deep foundation key along the perimeter of the mat.

Figure 1 presents a schematic view of the foundation and identifies the coordinate system used in these analyses. Figure 2 presents the stick model used in the structural analysis of the Canister Transfer Building.

The various static and dynamic loads and load combinations used in these analyses were obtained from Calculation 05996.02-SC-5-2 (S&W, 2001). All loads are transferred to the bottom of the mat. Moments, when transferred to the bottom of the mat, result in eccentricity of the applied load with respect to the center of gravity of the mat. Lateral loads, when combined with the vertical load, result in inclination of the vertical load, which decreases the allowable bearing capacity.

The generalized soil profile at the site is shown on Figure 3. The soil profile consists of ~30 ft of silty clay/clayey silt with sandy silt/silty sand layers (Layer 1), overlying ~30 ft of very dense fine sand (Layer 2), overlying extremely dense silt (N ≥100 blows/ft, Layer 3). SAR Figures 2.6-21 through 23 present foundation profiles showing the relationship of the Canister Transfer Building with respect to the underlying soils. These profiles, located as shown in SAR Figure 2.6-18, provide more detailed stratigraphic information, especially within the upper ~30-ft thick layer at the site.

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

GEOTECHNICAL PROPERTIES

Based on laboratory test results presented in Table 3 of Calculation 05996.02-G(B)-5-2 (SWEC, 2000a), γ_{moist} = 80 pcf above the bottom of the mat and 90 pcf below the mat.

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

The undrained shear strengths measured in the triaxial tests are used for the dynamic bearing capacity analyses because the partially saturated, fine-grained soils will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. As indicated in Figure 6, the undrained strength of the soils within ~10 ft of grade is assumed to be 2.2 ksf. This value is the lowest strength measured in the UU tests, which were performed at confining stresses of 1.3 ksf. This confining stress corresponds to the in situ vertical stress existing near the middle of the upper layer, prior to construction of these structures. It is much less than the final stresses that will exist under the cask storage pads and the Canister Transfer Building following completion of construction. Figure 6 illustrates that the undrained strength of these soils increase as the loadings of the structures are applied; therefore, 2.2 ksf is a very conservative value for use in the bearing capacity analyses of these structures.

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

In determining the bearing capacity of the foundation, the average shear strength of the soils along the anticipated bearing capacity failure slip surface should be used. This slip surface is normally confined to the zone within a depth below the footing equal to the minimum width of the footing. For the Canister Transfer Building, the effective width of the footing is decreased because of the large eccentricity of the load on the mat due to the seismic loading. As indicated in Table 2.6-10, the minimum effective width of the Canister Transfer Building occurs for Load Case IIIA, where B' = 119.5 ft. This is greater than the depth of the upper layer (~30 ft). Therefore, it is conservative to use the average strength of the soils in the upper layer in the bearing capacity analyses, since all of the soils in the upper layer will be effective in resisting failure along the anticipated bearing capacity slip surface.

The undrained strength used in the bearing capacity analyses presented herein is a weighted average strength that is applicable for the soils in the upper layer. This value is determined using the value of undrained shear strength of 2.2 ksf noted above for the soils tested at depths of ~10 ft and the relative strength increase measured for the soils below depths of ~12 ft in the cone penetration tests that were performed within the Canister Transfer Building footprint. As indicated on SAR Figure 2.6-18, these included CPT-37 and CPT-38. Similar increases in undrained strength for the deeper lying soils were also noted in all of the other CPTs performed in the pad emplacement area.

Attachment B presents copies of the plots of s_u vs depth for CPT-37 and CPT-38, which are included in Appendix D of ConeTec(1999). These plots are annotated to identify the average undrained strength of the cohesive soils measured with respect to depth. As shown by the plot of s_u for CPT-37, the weakest zone exists between depths of ~5 ft and ~12 ft. The results for CPT-38 are similar, but the bottom of the weakest zone is at a depth of ~11 ft. The underlying soils are all much stronger. The average value of s_u of the cohesive soils for the depth range from ~18 ft to ~28 ft is ~2.20 tsf, compared to s_u ~1.34 tsf for the zone between ~5 ft and ~12 ft. Therefore, the undrained strength of the deeper soils in the upper layer was ~64% (Δs_u = 100% x [(2.20 tsf - 1.34 tsf) / 1.34 tsf] higher than the strength measured for the soils within the depth range of ~5 ft to ~12 ft. The relative strength increase was even greater than this in CPT-38.

Using 2.2 ksf, as measured in the UU triaxial tests performed on specimens obtained from depths of ~10 ft, as the undrained strength applicable for the weakest soils (i.e., those in the depth range of ~5 ft to ~12 ft), the average strength for the soils in the entire upper layer is calculated as shown in Figure 4. The resulting average value, weighted as a function of the depth, is s_u ~3.18 ksf. This value would be much higher if the results from CPT-38 were used; therefore, this is considered to be a reasonable lower-bound value of the average strength applicable for the soils in the upper layer that underlie the Canister Transfer Building.

Further evidence that this is a conservative value of s_u for the soils in the upper layer is presented in Figure 6. This plot of s_u vs confining pressure illustrates that this value is slightly less than the average value of s_u measured in the CU triaxial tests that were performed on specimens obtained from depths of ~10 ft at confining stresses of 2.1 ksf. As indicated in this figure, the confining stress of 2.1 ksf used to test these specimens is comparable to the vertical stress that will exist ~7 ft [(2.1 ksf – 1.46 ksf) ÷ 0.09 kcf] below the Canister Transfer Building mat following completion of construction. Since these tests were performed on specimens of the weakest soils underlying the Canister Transfer Building mat (the deeper lying soils are stronger based on the SPT and the cone penetration test data), it is conservative to use the weighted average value of s_u of 3.18 ksf for the soils in the entire upper layer of the profile in the bearing capacity analyses.

Direct shear tests were performed on undisturbed specimens of the silty clay/clayey silt obtained from Borings CTB-6 and CTB-S, which were drilled in the locations shown in SAR Figure 2.6-18. These specimens were obtained from Elevation ~4469, approximately the elevation of the bottom of the perimeter key proposed at the base of Canister Transfer

Building mat. Note, this key is being constructed around the perimeter of the mat to ensure that the full shear strength of the clayey soils is available to resist sliding of the structure due to loads from the design basis ground motion. These direct shear tests were performed at normal stresses that ranged from 0.25 ksf to 3.0 ksf. This range of normal stresses bounds the ranges of stresses expected for static and dynamic loadings from the design basis ground motion.

The results of these tests are presented in Attachments 7 and 8 of the Appendix 2A of the SAR and they are plotted in Figures 7 and 8. Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, sliding stability analyses included below of the Canister Transfer Building constructed directly on the silty clay are performed using the average shear strength measured in these direct shear tests for a normal stress equal to the vertical stress under the building following completion of construction, but prior to imposition of the dynamic loading due to the earthquake. As shown in Figures 7 and 8, this average shear strength is 1.7 ksf and the friction angle is set equal to 0°.

Effective-stress strength parameters are estimated to be ϕ = 30° and c = 0 ksf, even though these soils may be somewhat cemented. This value of ϕ is based on the PI values for these soils, which ranged between 5% and 23% (SWEC, 2000a), and the relationship between ϕ and PI presented in Figure 18.1 of Terzaghi & Peck (1967).

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

Case IA Static using undrained strength parameters: ϕ = 0° & c = 3.18 ksf.

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

and dynamic bearing capacity analyses are performed using ϕ = 0° & c = 3.18 ksf.

Soil Cement Properties:

The unit weight of the soil cement is assumed to be 100 pcf in the analyses included herein and the unconfined compressive strength is 250 psi. (Initial results of the soil-cement testing indicate that 110 pcf is a reasonable lower-bound value for the total unit weight of the soil cement adjacent to the Canister Transfer Building foundation.) This strength is consistent with the soil-cement mix proposed for use within the frost zone adjacent to the cask storage pads and is based on the assumption that the strength will be at least this value to obtain a soil cement mix design that will satisfy the durability requirements of the ASTM wet/dry and freeze/thaw tests.

PFS is developing the soil-cement mix design using standard industry practice, in accordance with the criteria specified by the Portland Cement Association. This effort includes performing laboratory testing of soils obtained from the site. This on-going laboratory testing is being performed in accordance with the requirements of Engineering Services Scope of Work (ESSOW) for Laboratory Testing of Soil-Cement Mixes, ESSOW 05996.02-G010, Rev. 0. This program includes measuring gradations and Atterberg limits of samples of the near-surface soils obtained from the site. It includes testing of mixtures of these soils with varying amounts of cement and the testing of compacted specimens of soil-cement to determine moisture-density relationships, freeze/thaw and wet/dry characteristics, compressive and tensile strengths, and permeability of compacted soil-cement specimens. The entire laboratory testing program is being conducted in full compliance with the Quality Assurance (QA) Category I requirements of the ESSOW.

As part of this effort, PFS is performing so-called durability testing. These tests are performed in accordance with ASTM D559 and D560 to measure the durability of soil cement specimens exposed to 12 cycles of wet/dry and freeze/thaw conditions. As indicated on p. 16 of PFS Calculation 05996.02-G(B)-04-8:

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

PFS is performing these tests to determine the amounts of cement and water that must be added to the site soils and to determine the compaction requirements to ensure that the soil cement will be durable and will withstand exposure to the elements. As indicated on p. 8 of PCA[1]:

> *"The freeze-thaw and wet-dry tests were designed to determine whether the soil-cement would stay hard or whether expansion and contraction on alternate freezing-and-thawing and moisture changes would cause the soil-cement to soften."*

And on p. 32:

> *"The principle requirement of a hardened soil-cement mixture is that it withstand exposure to the elements. Thus the primary basis of comparison of soil-cement mixtures is the cement content required to produce a mixture that will withstand the stresses induced by the wet-dry and freeze-thaw tests. The service record of projects in use proves the reliability both of the results based on these tests and of the criteria given below.*
>
> *The following criteria are based on considerable laboratory test data, on the performance of many projects in service, and on information obtained from the outdoor exposure of several thousand specimens. The use of these criteria will provide the minimum cement content required to produce hard, durable soil-cement, suitable for base-course construction of the highest quality.*
>
> *1. Soil-cement losses during 12 cycles of either the wet-dry test or freeze-thaw test shall conform to the following limits:*
>
> *Soil Groups A-1, A-2-4, A-2-5, and A-3, not over 14 percent;*
>
> *Soil Groups A-2-6, A-2-7, A-4, and A-5, not over 10 percent;*
>
> *Soil Groups A-6 and A-7, not over 7 percent.*

[1] Portland Cement Association, "Soil-Cement Laboratory Handbook," Skokie, IL, 1971.

2. *Compressive strengths should increase both with age and with increases in cement content in the ranges of cement content producing results that meet requirement 1."*

The on-going laboratory testing program will also include additional tests to confirm that the bond at the interfaces between lifts of soil-cement and soil-cement and the site soils will exceed the strength of the in situ clayey soils. These tests will include direct shear tests, performed on specimens prepared from the site soils at various cement and moisture contents, in a manner similar to that used by DeGroot[2] in his testing of bond along soil-cement interfaces. This testing will include direct shear tests to be performed in the laboratory in the near-term (pre-construction) during the soil-cement mix development to demonstrate that the required interface strengths can be achieved (p. 2.6-113 of SAR) and during construction to demonstrate that the required interface strengths are achieved (p. 2.6-114 of SAR). In addition, PFS has committed to augmenting this field testing program by performing additional site-specific testing of the strengths achieved at the interface between the bottom of the soil cement and the underlying soils.

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

METHOD OF ANALYSIS

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

The following load combinations are analyzed:

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

For Case II, 100% of the dynamic lateral forces in both X and Z directions are combined. For Cases III and IV, 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the dynamic loading acts in the other two directions. For these cases, the suffix "A" is used to designate 40% in the X direction (N-S for the Canister Transfer Building, as shown in Figure 1), 100% in the Y direction (vertical), and 40% in the Z direction (E-W). Similarly, the suffix "B" is used to designate 40% in the X direction, 40% in the Y, and 100% in the Z, and the suffix "C" is used to designate 100% in the X direction and 40% in the other two directions. Thus,

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

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

Combining the effects of the three components of the design basis ground motion in this manner is in accordance with ASCE-4 (1986).

ANALYSIS OF OVERTURNING STABILITY

The factor of safety against overturning is defined as:

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

The overturning stability of the Canister Transfer Building is determined using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads are listed in Table 2.6-11, and they were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (S&W, 2001) and described in SAR Section 4.7.1.5.3. The masses and accelerations of the joints (see Figure 2 for locations of the joints) used in the model of the Canister Transfer Building in Calculation 05996.02-SC-5 are listed on the left side of Table 2.6-11, and the resulting inertial forces and associated moments are listed on the right. Based on building geometry shown schematically in Figure 1 and the forces and moments shown in Table 2.6-11, overturning is more critical about the N-S axis (279.5 ft) than about the E-W axis (240 ft). Page 37 of Calculation 05996.02-SC-5 indicates that the moment due to angular (rotational) acceleration of the structure is 465,729 ft-K about the N-S axis and 1,004,332 ft-K about the E-W axis.

The vertical force due to the earthquake can act upward or downward. However, when it acts downward, it acts in the same direction as the weight, tending to stabilize the structure with respect to overturning stability. The minimum factor of safety against overturning will occur when the maximum dynamic vertical force acts in the upward direction, tending to unload the mat and reduce the resisting moment. Therefore, calculate the factor of safety for Case III.

CHECKING OVERTURNING ABOUT THE N-S AXIS

For Case IIIA, where 40% of the horizontal force due to the earthquake act in the N-S and E-W directions and 100% acts vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 79,779 K, (i.e., Weight – Total $F_{V\,Dyn}$), as shown in Table 2.6-11. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of 240 ft, or 120 ft. Therefore,

$$\Sigma M_{Resisting} = (97{,}749 - 79{,}779)\ K \times 120\ ft = 2{,}156{,}400\ ft\text{-}K.$$

This ignores the eccentricities of the vertical masses with respect to the center of the mat. Incorporating these eccentricities, which are included in Attachment A of Calc 05996.02-SC-5, Rev. 2, the resulting resisting moment is calculated as follows:

JOINT	EL.	MASS Y k-sec²/ft	AY g's	Z (E-W) ft	Moment Arm E-W ft	$SM_{@N\text{-}S}$ ft-K
0	94.25	260.1	0.783	0	120.00	218,002
1	95	1,908.0	0.783	-0.73	119.27	1,589,353
2	130	420.4	0.821	-2.02	117.98	285,292
3	170	304.3	0.913	-3.14	116.86	99,412
4	190	117.1	0.928	0	120.00	32,638
5	190	27.6	1.840	0	120.00	-89,478
6	170	1.0	0	0	120.00	3,860
					Total =	**2,139,080**

The driving moments include 40% of the ΣM acting about the N-S axis, $\Sigma M_{@X}$ in Table 2.6-11, which is 0.4 x 2,706,961.4 = 1,082,785 ft-K, and 40% of the moment about the N-S axis due to angular (rotational) acceleration of the structure, which is 0.4 x 465,729 = 186,292 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the E-W axis do not contribute to overturning about the N-S axis; therefore,

$$\sum M_{Driving} = \sqrt{1{,}082{,}785^2 + (186{,}292)^2} = 1{,}098{,}694 \quad \text{ft} - \text{K}$$

and **FS_{OT}** = 2,156,400 ÷ 1,098,694 = **1.96**

about the N-S axis for Case IIIA without including eccentricities of vertical masses.

Including the effect of the eccentricities of the vertical masses, the resulting factor of safety against overturning is:

FS_{OT} = **2,139,080** ÷ 1,098,694 = **1.95** **(Minimum)**

For Case IIIB, where 100% of the horizontal force due to the earthquake acts in the E-W direction and 40% acts in the N-S direction and vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 40% of 79,779 K, (i.e., Weight – Total $F_{V\ Dyn}$), as shown in Table 2.6-11. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of 240 ft, or 120 ft. Therefore,

$\Sigma M_{Resisting}$ = (97,749 – 0.4 x 79,779) K x 120 ft = 7,900,488 ft-K.

The driving moments include 100% of the ΣM acting about the N-S axis, $\Sigma M_{\theta X}$ in Table 2.6-11, which is 2,706,961.4 ft-K, and 100% of the moment about the N-S axis due to angular (rotational) acceleration of the structure, which is 465,729 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the E-W axis do not contribute to overturning about the N-S axis; therefore,

$$\sum M_{Driving} = \sqrt{2{,}706{,}961.4^2 + 465{,}729^2} = 2{,}746{,}733 \quad ft-K$$

and FS_{OT} = 7,900,488 ÷ 2,746,733 = **2.88** about the N-S axis for Case IIIB.

Case IIIC, where 100% of the horizontal force due to the earthquake acts in the N-S direction and 40% acts in the E-W direction and vertically upward, **is less critical** for overturning about the N-S axis than Case IIIB.

CHECKING OVERTURNING ABOUT THE E-W AXIS

For Case IIIA, where 40% of the horizontal force due to the earthquake act in the N-S and E-W directions and 100% acts vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 79,779 K, (i.e., Weight – Total $F_{V\,Dyn}$), as shown in Table 2.6-11. For overturning about the E-W axis, the moment arm for the resisting moment equals ½ of 279.5 ft, or 139.75 ft. Therefore,

$\Sigma M_{Resisting}$ = (97,749 – 79,779) K x 139.75 ft = 2,511,308 ft-K.

This ignores the eccentricities of the vertical masses with respect to the center of the mat. Incorporating these eccentricities, the resulting resisting moment is calculated as follows:

JOINT	EL.	MASS Y k-sec²/ft	AY g's	Moment Arm N-S ft	$SM_{\theta E-W}$ ft-K
0	94.25	260.1	0.783	139.75	253,882
1	95	1,908.0	0.783	138.08	1,840,009
2	130	420.4	0.821	131.46	317,889
3	170	304.3	0.913	143.18	121,802
4	190	117.1	0.928	139.75	38,010
5	190	27.6	1.840	139.75	-104,205
6	170	1.0	0	139.75	4,496
				Total =	**2,471,883**

The driving moments include 40% of the ΣM acting about the E-W axis, $\Sigma M_{\theta z}$ in Table 2.6-11, which is 0.4 x 2,849,703 = 1,139,881 ft-K, and 40% of the moment about the E-W axis due to angular (rotational) acceleration of the structure, which is 0.4 x 1,004,322 = 401,729 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the N-S axis do not contribute to overturning about the E-W axis; therefore,

$$\sum M_{Driving} = \sqrt{1{,}139{,}881^2 + 401{,}729^2} = 1{,}208{,}601 \quad \text{ft} - \text{K}$$

and $\quad FS_{OT} = 2{,}511{,}308 \div 1{,}208{,}601 = \mathbf{2.07}$

about the E-W axis for Case IIIA without including eccentricities of vertical masses.

Including the effect of the eccentricities of the vertical masses, the resulting factor of safety against overturning is:

FS_{OT} = **2,471,883** ÷ 1,208,601 = **2.05 (Minimum @ E-W Axis)**

For Case IIIC, where 100% of the horizontal force due to the earthquake acts in the N-S direction and 40% acts in the E-W direction and vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 40% of 79,779 K, (i.e., Weight – Total $F_{V\,Dyn}$), as shown in Table 2.6-11. For overturning about the E-W axis, the moment arm for the resisting moment equals ½ of 279.5 ft, or 139.75 ft. Therefore,

$\Sigma M_{Resisting}$ = (97,749 – 0.4 x 79,779) K x 139.75 ft = 9,200,777 ft-K.

The driving moments include 100% of the ΣM acting about the E-W axis, $\Sigma M_{\theta z}$ in Table 2.6-11, which is 2,849,703.4 ft-K, and 100% of the moment about the E-W axis due to angular (rotational) acceleration of the structure, which is 1,004,322 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the N-S axis do not contribute to overturning about the E-W axis; therefore,

$$\sum M_{Driving} = \sqrt{2{,}849{,}703^2 + 1{,}004{,}322^2} = 3{,}021{,}501 \quad \text{ft} - \text{K}$$

and $\quad FS_{OT} = 9{,}200{,}777 \div 3{,}021{,}501 = \mathbf{3.05}$ about the E-W axis for Case IIIC.

Case IIIB is less critical for overturning about the N-S axis than Case IIIC.

ANALYSIS OF SLIDING STABILITY

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

$$FS = \text{Resisting Force} \div \text{Driving Force} = T \div V$$

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

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

where, N (normal force) = $\Sigma F_v = F_{v\,Static} + F_{v\,Eqk}$

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

c = 1.7 ksf, as discussed above under "Geotechnical Properties."

B = 240 feet

L = 279.5 feet

The driving force, V, is calculated as follows:

$$V = \sqrt{F_{HN-S}^2 + F_{HE-W}^2}$$

SLIDING STABILITY OF THE CANISTER TRANSFER BUILDING ON IN SITU CLAYEY SOILS

Based on Half of the Passive Resistance of the Soil Cement and the Peak Strength of the Clayey Soils Under the Building

The sliding stability of the CTB was evaluated using the foundation loadings developed in the soil-structure interaction analyses (Calculation 05996.02-SC-5, S&W, 2001). In this case, the strength of the clayey soils at the bottom of the 1.5-ft deep key around the CTB mat was based on the average of the two sets of direct shear tests performed on samples of soils obtained from beneath the CTB, approximately at the elevation proposed for founding the structure. The results of these tests are included in Attachments 7 and 8 of Appendix 2A of the SAR, and Figures 7 and 8 present plots of peak shear stress vs normal stress measured in these tests. As discussed above under Geotechnical Properties, $\phi = 0°$ and a shear strength of 1.7 ksf were used for the clayey soils underlying the Canister Transfer Building in determining resisting forces for the earthquake loading combinations.

The unconfined compressive strength of the soil cement adjacent to the Canister Transfer Building will be at least 250 psi. These analyses assume that the peak shear strength of the clayey soils under the Canister Transfer Building are available to resist sliding along with up to half of the passive resistance of the soil cement.

The backfill to be placed around the Canister Transfer Building mat and 1.5-ft deep key will be soil cement, constructed from the eolian silt and silty clay that was excavated from the area. For soil cement constructed using these soils, it is reasonable to assume the lower bound value of γ is 100 pcf, $\phi = 0°$ & c = 125 psi.

For the soil cement, $P_p = 2c \times D_f \times (B \text{ or } L)$

For 5' of soil cement, using a factor of safety of 2 applied to the passive resistance,

$$P_p = \frac{2 \times c \times D_f \times w}{FS} = \frac{2 \times 125 \frac{\#}{in.^2} \times \frac{144 \cdot in.^2}{ft^2} \times \frac{K}{1{,}000\#} \times 5\text{ ft} \times 1 \frac{ft}{LF}}{2} = 90 \frac{K}{LF}$$

The CTB mat is 240' wide in the E-W direction and 279.5' long in the N-S direction; therefore, the passive force available to resist sliding is at least 240' x 90 K/LF = 21,600 K acting in the N-S direction in the analyses that use half of the passive resistance of the soil cement adjacent to the mat.

The effects of wall movement on wall pressure are defined in DM-7[3] (p. 7.2-60) as the ratio of horizontal displacement to the height of the wall. For stiff cohesive soils, the wall rotation or yield ratio, y/H, required to fully mobilize passive resistance is 0.02, or 2%. For dense cohesionless soils, even less movement is required to reach full passive, ~0.2%. Lambe & Whitman (1969, p 166) also indicates that little horizontal compression, ~0.5%, is required to reach half of full passive resistance for dense sands. The soil cement will be compacted to a dense state, and once it cures, it is expected to be stiffer than dense sand, requiring less displacement to reach full passive resistance. Therefore, it is conservative to assume that half of the total passive resistance is available to resist sliding of the building.

Note, if we assume that the soil cement is comparable in stiffness to stiff cohesive soil, the figure from DM-7 cited above indicates that yield ratio, y/H, required to fully mobilize passive resistance is 2%. It is reasonable to use a yield ratio of half of this, or ~1% of the 5 ft height of the mat + 1.5-ft deep key, to reach half of passive resistance for the soil cement adjacent to the mat. This indicates that a horizontal displacement of the mat = 0.01 x 6.5 ft x 12 in./ft = 0.78 in. would be sufficient to reach half of the passive resistance. Since there are no safety-related systems that would be severed or otherwise impacted by movements of this small magnitude, it is reasonable to use this passive thrust to resist sliding. The following analysis demonstrates that it is also reasonable to use the resistance provided by the peak shear strength of the clayey soils enclosed within the perimeter key at the base of the mat to resist sliding in this case, because this amount of horizontal displacement can be obtained from elastic deformation of the clayey soils underlying the building.

The horizontal displacement of the Canister Transfer Building is estimated using elastic theory, as described in Section 4.3, "Rectangles Subjected to Shear Loading," of Poulos and Davis[4].

$$\rho = \frac{q \times a \times I}{E}$$ Eq. 4.9 Poulos & Davis

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

4 Poulos, H. G., and Davis, E. H., Elastic Solutions for Soil and Rock Mechanics, John Wiley & Sons, New York, NY, 1974.

$$G_s = \rho \times V_s^2 = \frac{80\text{ pcf}}{32.2\,\frac{\text{ft./sec}^2}{g}} \times (540\text{ ft/sec})^2 = 724{,}472\text{ psf} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 = 5{,}031\text{ psi}$$

$$E_s = 2 \times (1+\nu) \times G_s = 2 \times (1+0.4) \times 5{,}031\text{ psi} = 14{,}087\text{ psi}$$

In the E-W direction (See Table 2.6-11 for horizontal shear values):

$$q = \frac{99{,}997\text{ K}}{240\text{ ft} \times 279.5\text{ ft}} = 1.49\text{ ksf} \times \frac{1{,}000\text{ lbs}}{\text{K}} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 = 10.4\text{ psi}$$

$$\frac{h}{b} = \frac{6.5\text{ ft}}{279.5\text{ ft}} = 0.023$$

$$\frac{b}{a} = \frac{279.5\text{ ft}}{240\text{ ft}} = 1.17$$

In the N-S direction:

$$q = \frac{111{,}108\text{ K}}{240\text{ ft} \times 279.5\text{ ft}} = 1.66\text{ ksf} \times \frac{1{,}000\text{ lbs}}{\text{K}} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 = 11.5\text{ psi}$$

$$\frac{h}{b} = \frac{6.5\text{ ft}}{240\text{ ft}} = 0.027$$

$$\frac{b}{a} = \frac{240\text{ ft}}{279.5\text{ ft}} = 0.859$$

From Figure 4.17 of Poulos & Davis, estimate the horizontal displacement factor for the corners for horizontal shear of a horizontal rectangle. For the h/b and b/a values shown above, $I_{E\text{-}W} = 0.62$ and $I_{N\text{-}S} = 0.59$.

$$\rho_{E-W} = \frac{10.4\text{ psi} \times 240\text{ ft} \times 12\,\frac{\text{in.}}{\text{ft}} \times 0.62}{14{,}087\text{ psi}} = 1.32\text{ inches} \quad \text{Eq. 4.9 Poulos \& Davis}$$

$$\text{Yield Ratio} = \frac{\rho}{H} = \frac{1.32\text{ in.}}{6.5\text{ ft} \times 12\,\frac{\text{in.}}{\text{ft}}} = 0.017\text{, or }1.7\%$$

$$\rho_{N-S} = \frac{11.5\text{ psi} \times 279.5\text{ ft} \times 12\frac{\text{in.}}{\text{ft}} \times 0.59}{14{,}087\text{ psi}} = 1.62\text{ inches} \quad \text{Eq. 4.9 Poulos \& Davis}$$

$$\text{Yield Ratio} = \frac{\rho}{H} = \frac{1.62\text{ in.}}{6.5\text{ ft} \times 12\frac{\text{in.}}{\text{ft}}} = 0.021,\text{ or } 2.1\%$$

Thus, based on the shear modulus estimated from the shear wave velocity of the surficial silty clay/clayey silt, the horizontal displacement of the CTB subjected to the full horizontal earthquake load is calculated to be about 1.3 to 1.6 inches using the elastic solution of a buried horizontal rectangle subjected to shear in an elastic half-space. This horizontal displacement corresponds to a yield ratio, defined as horizontal displacement ÷ height of wall, of 2% from translation of the 6.5 ft height of the CTB foundation mat adjacent to the soil cement. This yield ratio is larger than the yield ratio required to mobilize one half of full passive resistance for dense sand or stiff cohesive soils. This displacement is sufficient to develop full passive resistance in the soil cement adjacent tot he mat; therefore, it is conservative to use one-half of the passive resistance in these analyses

The results of the sliding stability analysis of the Canister Transfer Building for this case are presented in Table 2.6-13. In this table, the components of the driving and resisting forces are combined using the SRSS rule. All of these factors of safety are greater than 1.1, the minimum required value. These results indicate that the factors of safety are acceptable for all load combinations examined. The lowest factor of safety is 1.15, which applies for Cases IIIC and IVC, where 100% of the dynamic earthquake forces act in the N-S direction and 40% act in the other two directions.

These results are conservative, because they assume that only one-half of the passive pressures are available to resist sliding and no credit is taken for the fact that the strength of cohesive soils increases as the rate of loading increases. Note, Newmark and Rosenblueth (1973) indicate:

> *"In all cohesive soils reported to date, strength and stiffness increase markedly with strain rate (Figs. 13.6 and 13.7). An increase of the order of 40 percent is common for the usual strain rates of earthquakes, above the strength and stiffness of static tests."*

Schimming et al, (1966), Casagrande and Shannon (1948, and Das (1993) all report similar increases in strength of cohesive soils due to rapid loading. Therefore, since these results are based on static shear strengths, they represent conservative lower-bound values of the factor of safety against sliding of the Canister Transfer Building founded on in situ silty clay/clayey silt with soil-cement backfill around the mat.

Based on the Full Passive Resistance of the Soil Cement and the Residual Strength of the Clayey Soils Under the Building

Before a complete sliding failure can occur, the full passive resistance of the soil cement must be engaged. Because the horizontal displacements associated with reaching the full passive state typically are large for soils, in the analyses where the full passive resistance of the soil cement adjacent to the mat is used, the shear strength of the clayey soils under the building is reduced to a conservative estimate of the residual shear strength based on the results of the direct shear tests.

The results of the direct shear tests, presented as plots of shear stress vs horizontal displacement in Attachment 7 of Appendix 2A of the SAR (annotated copies are included in Attachment C of this calculation), illustrate that the residual strength of these soils is nearly equal to the peak strength for those specimens that were tested at confining stresses of 2 ksf. For example, for Sample U-1C from Boring C-2, at horizontal displacements of ~0.025" past the peak strength, there is ~1.5% reduction in the shear strength indicated. The results for Sample U-1AA from Boring CTB-S showed no decrease in shear strength following the peak at ~0.025" horizontal displacement, and Samples U-3B&C from Boring CTB-6 showed a decrease of ~5%. The specimens that were tested at confining stresses of 1 ksf all show reductions of ~20% at horizontal displacements of ~0.025" past the peak.

The final effective vertical stresses at the base of the Canister Transfer Building, σ'_v, are ~1.5 ksf, now that the mat has been changed to 240 ft x 279.5 ft. This value is approximately half-way between the confining stresses of 1 and 2 ksf used for several of the direct shear tests. The residual strength of the clayey soils beneath the building are expected to show reductions from the peak strength of ~10% to ~12.5%; i.e., approximately half-way between the reductions observed for the specimens tested at confining stresses of 1 ksf and 2 ksf, since the final effective stresses under the building are ~1.5 ksf; i.e., approximately half-way between confining stresses used in these tests (1 ksf and 2 ksf). Therefore, it is reasonable to assume that the peak strength of the clayey soils enclosed within the perimeter key at the base of the Canister Transfer Building mat should be reduced to account for horizontal displacement required to reach full passive resistance of the soil cement adjacent to the mat. Based of the results of the direct shear tests performed on samples of the site soils, it would be reasonable to use a reduction of ~10% to ~12.5% to obtain the residual strength applicable for the final vertical stresses at the base of the Canister Transfer Building. The analyses that follow, however, reduce the peak strength even more than this, by a total of 20%, to provide additional conservatism.

The following table illustrates further that using a reduction of the peak strength equal to 20% provides a conservative estimation of the residual strength of these soils. This table presents the peak strengths measured in the direct shear tests at normal stresses of 1 ksf and 2 ksf. It also lists the final shear strengths measured in these tests, which were generally obtained at horizontal displacements of 0.25 inches or 0.30 inches. The table also lists the calculated post-peak strength reduction for these test results, as well as the average post-peak strength reduction for normal stress of 1.5 ksf, which is applicable for the state of stress existing under the Canister Transfer Building mat. Note, that the average post-peak strength reduction for normal stress of 1.5 ksf for the three direct shear tests is only 15.6% for these very high shear displacements in the direct shear tests. The maximum value of the average the post-peak strength reductions for normal stress of 1.5 ksf occurred for Sample U-3B&C in CTB-6, and it equaled 20.8%. If the results of this test were used to define the residual strength of these soils, the analyses would be performed at c = 1.5 ksf, the average of the post-peak strengths measured at the maximum shear displacements in these tests for normal stresses of 1 ksf and 2 ksf. This would result in higher factors of safety than are calculated and presented in Table 2.6-14, based on c = 1.36 ksf.

CALCULATION OF AVERAGE POST-PEAK STRENGTH REDUCTION FOR NORMAL STRESS APPLICABLE TO FINAL TRESSES UNDER THE CANISTER TRANSFER BUILDING

Boring	Sample	Normal Stress = 1 ksf			Normal Stress = 2 ksf			Average Post-Peak Strength Reduction for Normal Stress = 1.5 ksf
		Peak Strength	Strength at Maximum Shear Displace-ment	Post-Peak Strength Reduction	Peak Strength	Strength at Maximum Shear Displace-ment	Post-Peak Strength Reduction	
		ksf	ksf	%	ksf	ksf	%	%
C-2	**U-1C**	1.67	1.2	28.1	2.13	2.1	1.4	**14.8**
CTB-6	**U-3B&C**	1.57	1.1	29.9	2.15	1.9	11.6	**20.8**
CTB-S	**U-1AA**	1.42	1.1	22.5	1.58	1.7	~0.0	**11.3**

Average = 15.6

The results of the sliding stability analysis of the Canister Transfer Building for this case are presented in Table 2.6-14. In this table, the components of the driving and resisting forces are combined using the SRSS rule. All of these factors of safety are greater than 1.1, the minimum required value. These results indicate that the factors of safety are acceptable for all load combinations examined. The lowest factor of safety is 1.26, which applies for Cases IIIC and IVC, where 100% of the dynamic earthquake forces act in the N-S direction and 40% act in the other two directions. These results demonstrate that there is additional margin available to resist sliding of the building due to the earthquake loads, even when very conservative estimates of the residual shear strength of the clayey soils are used.

SLIDING STABILITY OF THE CANISTER TRANSFER BUILDING ON COHESIONLESS SOILS

The Canister Transfer Building will be founded on clayey soils that have an adequate amount of cohesive strength to resist sliding due to the dynamic forces from the design basis ground motion. As shown in SAR Figures 2.6-21 through 2.6-23, however, some of the soils underlying the building are cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. Analyses presented on the next six pages address the possibility that sliding may occur along a deeper slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

The resistance to sliding is greatly reduced for frictional materials when the dynamic forces due to the earthquake act upward. The normal forces act downward for Case IV loadings and, hence, the resisting forces will be much greater than those for Case III. Therefore, these analyses are performed only for Load Cases IIIA, IIIB, and IIIC. As described above, these load cases are defined as follows:

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

As shown in SAR Figures 2.6-21 through 2.6-23, the top of the cohesionless layer varies from about 5 ft to about 9 ft below the mat, and it generally is at a depth of about 6 ft below the mat. These analyses include the passive resistance acting on a plane extending from grade down to the top of the cohesionless layer, plus the shear strength available at the ends of the silty clay block under the mat, plus the frictional resistance available along the top of the cohesionless layer. The weight of the clayey soils existing between the top of the cohesionless soils and the bottom of the mat is included in the normal force used to calculate the frictional resistance acting along the top of the cohesionless layer.

A review of the cone penetration test results (ConeTec, 1999) obtained within the top 2 ft of the layer of nonplastic silt/silty sand/sandy silt underlying the Canister Transfer Building indicated that $\phi = 38°$ is a reasonable minimum value for these soils. This review is presented on the next page.

The next five pages illustrate that the factor of safety against sliding along the top of this layer is >1.1 for all load cases (i.e., Load Cases IIIA, IIIB, and IIIC). These analyses include several conservative assumptions. They are based on static strengths of the silty clay block under the Canister Transfer Building mat, even though, as reported in Das (1993), experimental results indicate that the strength of cohesive soils increases as the rate of loading increases. For rates of strain applicable for the cyclic loading due to the design basis ground motion, Das indicates that for most practical cases, one can assume that $c_{u\ dynamic} \sim 1.5 \times c_{u\ static}$. In addition, the silty sand/sandy silt layer is not continuous under the Canister Transfer Building mat, and this analysis neglects cementation of these soils that was observed in the samples obtained in the borings. Therefore, sliding is not expected to occur along the surface of the cohesionless soils underlying the Canister Transfer Building.

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CALCULATION SHEET

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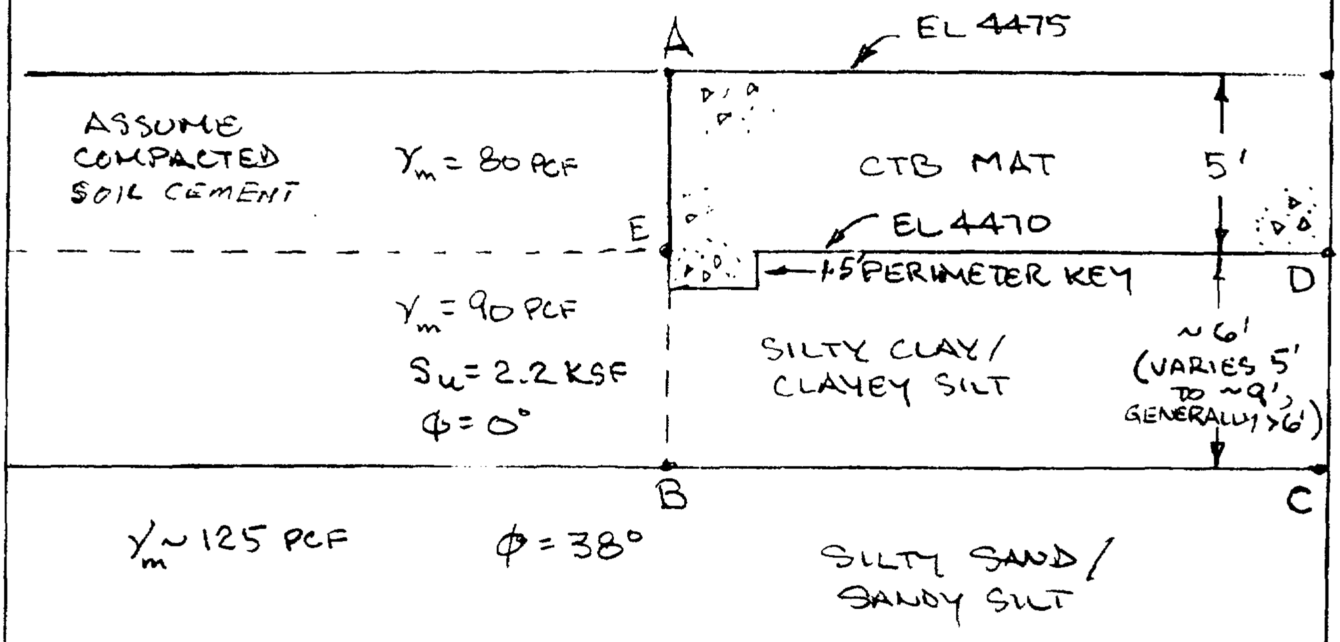
CALCULATION IDENTIFICATION NUMBER				PAGE 25
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6		

SLIDING ON DEEP PLANE AT TOP OF SILTY SAND/ SANDY SILT LAYER



NOTE: VALUE OF ϕ BASED ON ϕ DATA FROM CPT-37 & 38. PRESENTED IN CONETEC (1999)

ID	~DEPTH OF SILTY SAND	MIN ϕ	MAX ϕ	AVG ϕ	MEDIAN ϕ	ϕ IN TOP 2'
CPT-37	~11.6' TO ~18.7'	36*	44	40	40	~38
CPT-38	~11' TO ~18'	38	46	43	44	~38

PASSIVE PRESSURES ACTING ON PLANE AB WILL INCREASE AS B GETS DEEPER IN THE SILTY SAND/SANDY SILT LAYER; ∴ USE ϕ NEAR THE TOP OF THE LAYER. ⇒ $\phi = 38°$.

N VALUES ARE HIGH, GENERALLY ≫ 20 BL/FT; ∴ $\phi = 38°$ IS REASONABLE

* EXCLUDING SINGLE VALUE OF $\phi = 34°$ AT $z = 13.8'$

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CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 13-6	OPTIONAL TASK CODE	PAGE 26

SLIDING ON DEEP COHESIONLESS PLANE

$$FS_{SLIDING} = \frac{\Sigma \text{ RESISTING FORCES}}{\Sigma \text{ DRIVING FORCES}}$$

RESISTING FORCES INCLUDE PASSIVE RESISTANCE AVAILABLE ALONG AB + SHEAR RESISTANCE ALONG ENDS OF BLOCK BCDE + FRICTION ALONG BC.

(1) PASSIVE RESISTANCE AVAILABLE ALONG AB

INCLUDES $(2 \times 5 \times 125 \times .144 \frac{K}{FT^2}) \times (5') = 180$ K/LF FOR COMPACTED 5' SOIL-CEMENT ADJACENT TO 5' MAT

$+ \frac{1}{2} \gamma H^2 K_P + q_s H K_P + 2CH\sqrt{K_P}$ FOR 5' BLOCK OF SILTY CLAY UNDERLYING THE COMPACTED SOIL-CEMENT

$$\frac{1}{2}\left(0.090 \frac{K}{FT^3}\right) \times (5\ FT)^2 \times \overset{K_P}{1.0} + \overset{H}{6\ FT} \times \overset{\gamma}{0.080 \frac{K}{FT^3}} \times 5\ FT \times \overset{K_P}{1.0}$$

$$+ 2 \times \overset{C}{2.2} \frac{K}{FT^2} \times 5\ FT \times \sqrt{1.0} = 1.125 + 2.40 + 22.0 = 25.52 \frac{K}{FT}$$

$\therefore$ TOTAL PASSIVE RESISTANCE AVAILABLE ALONG AB

$= 180 + 25.52 = 205.52$ K/LF

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 27
05996.02	G(B)	13-6		

(2) ESTIMATE ADDITIONAL RESISTANCE TO SLIDING AVAILABLE AT THE ENDS OF THE BLOCK OF SILTY CLAY THAT MUST SHEARED BEFORE THE CTB CAN SLIDE. INCLUDE ONLY THE PORTION BELOW THE CTB MAT; i.e., BCDE SHOWN ON PAGE 19.

$S_u = 2.2$ KSF, = MINIMUM S_u MEASURED IN UU TRIAXIAL TESTS AT $\sigma_c = 1.3$ KSF

$$\text{AREA BCDE} = 6\,FT \times 240\,FT_{E\text{-}W} = 1440\,\frac{FT^2}{END}$$

$$\therefore\ \Delta T_{ENDS_{E\text{-}W}} = 2\text{ ENDS} \times 1440\,\frac{FT^2}{END} \times 2.2\,\frac{K}{FT^2} = 6{,}336\,K_{E\text{-}W}$$

$$\Delta T_{END_{N\text{-}S}} = 2\text{ ENDS} \times 6' \times \underbrace{279.5'}_{L_{N\text{-}S}} \times 2.2\,\frac{K}{FT^2} = 7{,}379\,K_{N\text{-}S}$$

(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.

$$\Delta N_{CLAY} = \underbrace{6'}_{\Delta H} \times \underbrace{0.090\,\frac{K}{FT^3}}_{\gamma} \times \underbrace{240' \times 279.5'}_{B \times L} = 36{,}223\,K$$

(4) SINCE THE MATERIAL FROM GROUND SURFACE TO THE TOP COHESIONLESS SILTY SAND/SANDY SILT ARE ALL COHESIVE (SOIL CEMENT, SILTY CLAY), THE ACTIVE EARTHQUAKE PRESSURE IS SMALL AND NEGLIBLE.

NOTE: FRICTIONAL RESISTANCE WILL BE LOWER WHEN VERT EARTHQUAKE FORCES ACT UPWARD. $\therefore$ CHECK CASES III A, B & C

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CALCULATION SHEET

▲ 5010.65

3 CALCULATION IDENTIFICATION NUMBER				PAGE 28
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 13-6	OPTIONAL TASK CODE	

SLIDING ON DEEP PLANE

	N-S	VERT	E-W
CASE IIIA:	40% IN X	−100% IN Y	40% IN Z
FROM TABLE 1	$0.4\times 111{,}108^k$	$-79{,}779^k$	$\underbrace{0.4\times 99{,}997}_{L=V_{E\text{-}W}} = 39{,}999^k$

$$\therefore N = \underset{CTB\ DL}{97{,}749} - \underset{F_{VD}\text{ or }EQ_V}{79{,}779^k} + \underset{\Delta N_{CLAY}}{36{,}223^k} = 54{,}193^k$$

$$N \tan\phi = 54{,}193^k \tan 38° = 42{,}340^k$$

$$\therefore FS_{SLIDING_{N\text{-}S}} = \frac{205.52^k/_{LF}\times 240' + \underset{\sigma_{E\text{-}W}}{7{,}379^k} + 42{,}340^k}{0.4\times 111{,}108^k} = 1.78$$

$$FS_{SLIDING_{E\text{-}W}} = \frac{205.52\frac{k}{LF}\times \overset{L}{279.5} + 6336^k + 42{,}340^k}{0.4\times 99{,}997^k} = 1.65 > 1.1$$

∴ OK

	N-S	VERT	E-W
CASE IIIB	40% IN X	−40% IN Y	100% IN Z
FROM TABLE 1	$0.4\times 111{,}108^k$	$-0.4\times 79{,}779^k$	$99{,}997^k$
			$L = V_{E\text{-}W}$

$$\therefore N = \underset{CTB\ DL}{97{,}749^k} - \underset{F_{VD}}{0.4\times 79{,}779^k} + \underset{\Delta N_{CLAY}}{36{,}223^k} = 102{,}060^k$$

$$\Rightarrow T = \underbrace{\left(\overset{P_P}{180\frac{k}{L.F.}} + 25.52\frac{k}{L.F.}\right)\times \overset{L_{N\text{-}S}}{279.5'}}_{57{,}443^k} + 6{,}336^k_{E\text{-}W}$$

$$+ \underbrace{\overset{N}{102{,}060^k} \tan \overset{\phi}{38°}}_{79{,}738} = 143{,}517^k$$

$$FS = \frac{RESISTING}{DRIVING} = \frac{143{,}517^k}{99{,}997^k} = 1.44 > 1.1$$

∴ O.K

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CALCULATION SHEET

5010.65

4 CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 13-6	OPTIONAL TASK CODE	PAGE 29

SLIDING ON DEEP PLANE

	N-S	VERT	E-W
CASE III C	100% IN X	-40% IN Y	40% IN Z
FROM TABLE 1	111,108 K	-0.4 × 79,779 K	0.4 × 99,997

$$\therefore N = \overset{CTB\ LL}{97,749} - \underset{31,912}{\overset{F_{VD}}{0.4 \times 79,779\,K}} + \overset{\Delta N_{CLAY}}{36,223} = 102,060\,K$$

$$T_{N\text{-}S} = 205.52\,\frac{K}{LF} \times \overset{B}{240'} + \overset{\Delta T_{N\text{-}S}}{7,379\,K} + 102,060 \tan 38° = 136,442\,K$$

$$FS_{SLIDING} = \frac{T}{V_{N\text{-}S}} = \frac{136,442\,K}{111,108\,K} = 1.23 \quad > 1.1 \;\therefore OK$$

THE FACTOR OF SAFETY AGAINST SLIDING ON A DEEP PLANE OF COHESIONLESS SOIL IS > 1.1 FOR LOAD CASES III A, III B, & III C. THEREFORE THERE IS NO SLIDING ON A DEEP PLANE OF COHESIONLESS SOIL.

Allowable Bearing Capacity

Bearing capacity calculations are performed using the method for determining general bearing capacity failure, as presented in Winterkorn and Fang (1975). Local bearing capacity (punching shear) failure is ruled out due to the large size of the mat, 240' x 279.5'.

The general bearing capacity equation is a modification of Terzaghi's bearing capacity equation, which was developed for strip footings and which indicates that $q_{ult} = cN_c+qN_q+1/2\ \gamma BN_\gamma$. For this relationship, the ultimate bearing capacity of soil consists of three components: 1) cohesion, 2) surcharge, and 3) friction, which are represented by bearing capacity factors N_c, N_q, and N_γ. Terzaghi's bearing capacity equation has been enhanced by various investigators to incorporate shape, depth, and load inclination factors for different foundation geometries and loads as follows:

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

where

q_{ult} = ultimate bearing capacity

c = cohesion or undrained strength

q = effective surcharge at bottom of foundation, = γD_f

γ = unit weight of soil

B = foundation width

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

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

i_c, i_q, i_γ = load inclination factors

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

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

Bearing Capacity Factors

Bearing capacity factors computed based on relationships proposed by Vesic (1973), which are presented in Chapter 3 of Winterkorn and Fang (1975).

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

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

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

SHAPE FACTORS

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

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

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

DEPTH FACTORS

For $\frac{D_f}{B} \leq 1$:

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

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

$$d_\gamma = 1$$

INCLINATION FACTORS

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

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

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

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

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

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

Static Bearing Capacity of the Canister Transfer Building

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

Case IA Static using undrained strength parameters (ϕ = 0° & c = 3.18 ksf).

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

Table 2.6-9 presents the results of the bearing capacity analyses for these static load cases. The minimum factor of safety required for static load cases is 3.

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 6.5 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume ϕ = 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 ϕ = 30° and c = 0 results in higher allowable bearing pressures. As shown in Table 2.6-9, the gross allowable bearing capacity of the Canister Transfer Building for static loads for these soil strengths is 56.6 ksf.

CALCULATION IDENTIFICATION NUMBER				PAGE 33
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Static Analysis: **Case** **IA - Static** | **0 % in N-S, 0 % in Vert 0 % in E-W**

Soil Properties:
- s_u = **3,180** Average undrained strength (psf) in upper ~30' layer
- ϕ = **0** Friction Angle (degrees)
- γ = **90** Unit weight of soil (pcf)
- γ_{surch} = **80** Unit weight of surcharge (pcf)

Foundation Properties:
- B' = **240.0** Footing Width - ft (E-W) L' = **279.5** Length - ft (N-S)
- D_f = **5** Depth of Footing (ft)
- β = **0.0** Angle of load inclination from vertical (degrees)
- FS = **3** Factor of Safety required for $q_{allowable}$.
- F_V = **97,749** k EQ_V = 0 k
- $EQ_{H\ E\text{-}W}$ = **0** k + $EQ_{H\ N\text{-}S}$ = 0 k = 0 k for F_H

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

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

Equation		Value	Reference
$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.17	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.66	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.01	Eq 3.27

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

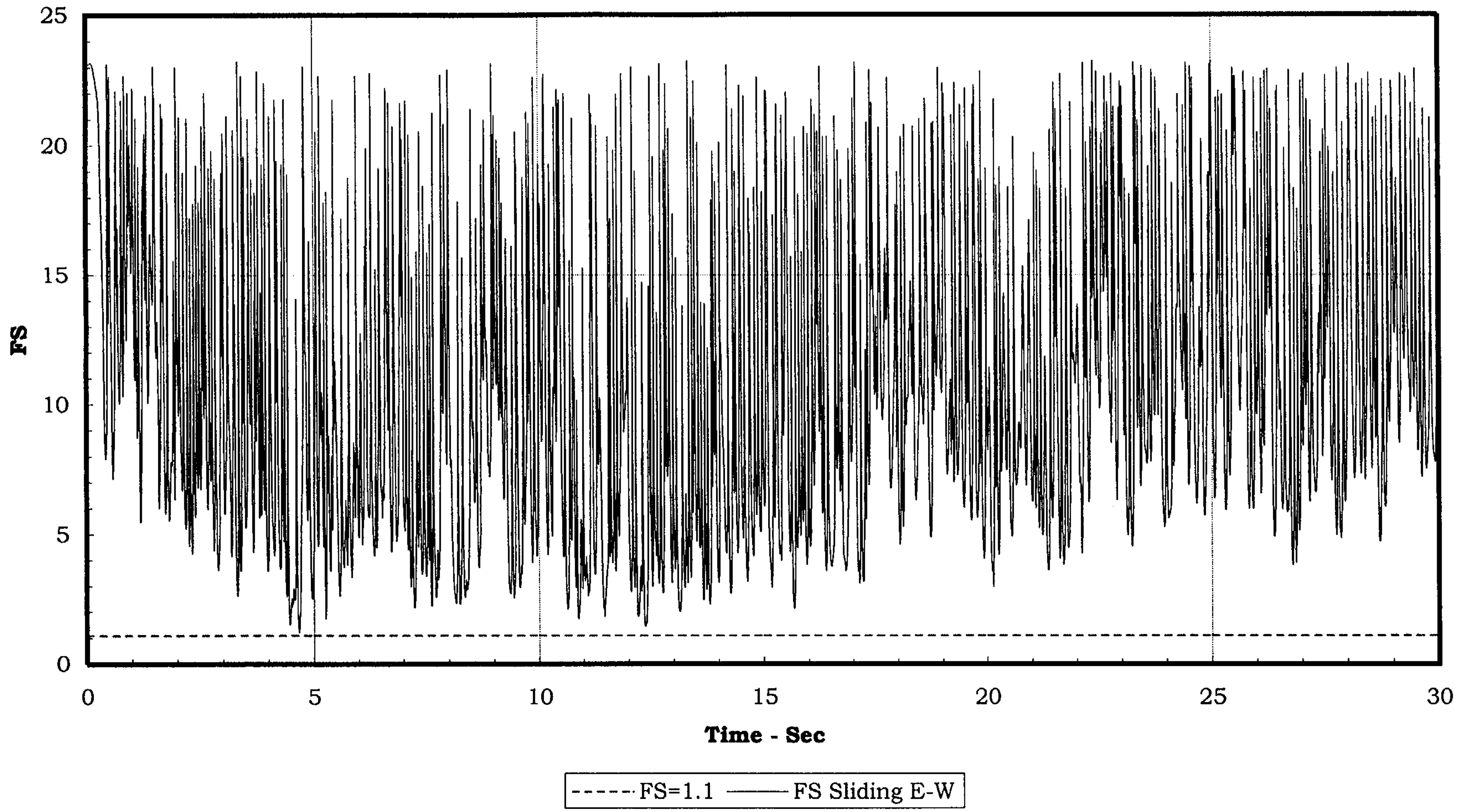
			N_c term		N_q term		N_γ term
Gross q_{ult} =	19,635	psf =	19,235	+	400	+	0

q_{all} = 6,540 psf = q_{ult} / FS

q_{actual} = 1,457 psf = $(F_v + EQ_v) / (B' \times L')$

FS_{actual} = 13.47 = q_{ult} / q_{actual} > 3 **Hence OK**

Sliding Stability - Dynamic Loads from Holtec for 2,000-Yr Earthquake for Pad Loaded with 8 Casks, μ = 0.8, and Best-Estimate Soil Properties c = 2.1 ksf and ϕ = 0 at Base of Concrete Pad Includes Dynamic Active, but No Passive Pressure



[geot]\05996\calc\brng_cap\pad\SSI.xls "Exhibit 3"

SEISMIC DESIGN

ASCE 4-86

ASCE STANDARD

Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures

September 1986

ASCE 1852

American Society of Civil Engineers

rock-like beneath the foundation. A rock-like foundation is defined by a shear-wave velocity of 3,500 ft/sec (1,100 m/sec) or greater at a shear strain of 10^{-3} percent or smaller when considering preloaded soil conditions due to the structure.

3.3.1.2 Spatial Variations of Free-Field Motion-- (a) Vertically propagating shear and compressional waves may be assumed for an SSI analysis provided that torsional effects due to nonvertically propagating waves are considered.

(b) Variation of amplitude and frequency content with depth may be considered for partially embedded structures. The spectral amplitude of the acceleration response spectra in the free field at the foundation depth shall be not less than ~~60% of the corresponding design response spectra at the finish grade in the free field.~~

3.3.1.3 Three-Dimensional Effects-- The three-dimensional phenomenon of radiation damping and layering effects of foundation soil shall be considered in SSI analysis.

3.3.1.4 Nonlinear Behavior of Soil-- The nonlinear behavior of soil shall be considered and may be approximated by equivalent linear material properties. Two types of nonlinear behavior may be identified: primary and secondary nonlinearities. "Primary nonlinearity" denotes nonlinear material behavior induced in the soil due to the excitation alone, i.e., ignoring structure response. "Secondary nonlinearity" denotes nonlinear material behavior induced in the soil due to structural response as a result of SSI. Primary nonlinearities shall be considered in the SSI analysis. Except for the provisions of 3.3.1.9, secondary nonlinearities including local nonlinear behavior in the vicinity of the soil-structure interface need not be considered.

3.3.1.5 Structure-to-Structure Interaction-- Structure-to-structure interaction may be generally neglected for overall structural response but shall be considered for local effects due to one structure on another, such as required in 3.5.3 for walls.

✓ **3.3.1.6 Effect of Mat and Lateral Wall Flexibility**-- The effect of mat flexibility for mat foundations and the effect of wall flexibility for embedded walls need not be considered in the SSI analysis.

3.3.1.7 Uncertainties in SSI Analysis- The uncertainties in the SSI analysis shall be considered. In lieu of a probabilistic evaluation of uncertainties, an acceptable method to account for uncertainties in SSI analysis is to vary the soil shear modulus. Soil shear modulus shall be varied between the best estimate value times $(1 + C_v)$ and the best estimate value divided by $(1 + C_v)$, where C_v is a factor that accounts for uncertainties in the SSI analysis and soil properties. The minimum value of C_v shall be 0.5.

$(1+C_v)\cdot G$

$G/(1+C_v)$

3.3.1.8 Model of Structure-- (a) Structural models defined in 3.1 may be simplified for the SSI analysis. Simplified models may be used provided they adequately represent the mass and stiffness effects of the structure and adequately match the predominant frequencies, related mode shapes, and participation factors of the more detailed structure model.

(b) When a simplified model is used to generate in-structure response spectra, representative in-structure response spectra also shall be adequately matched for fixed-base conditions in both the detailed and simplified models.

3.3.1.9 Embedment Effects-- The potential for reduced lateral soil support of the structure should be considered when accounting for embedment effects. One method to comply with this requirement is to assume no connectivity between structure and lateral soil over the upper half of the embedment or 20 ft (6 m), whichever is less. However, full connection between the structure and lateral soil elements may be assumed if adjacent structures founded at a higher elevation produce a surcharge equivalent to at least 20 ft (6 m) of soil.

20 ft

?

3.3.2 Subsurface Material Properties

3.3.2.1 General Requirements-- Subsurface material properties shall be determined by field and laboratory testing, supplemented as appropriate by experience, empirical relationships, and published data for similar materials. The following material properties shall be determined for use in equivalent-linear analyses: shear modulus, G; damping ratio, D; Poisson's ratio, ν; and total unit weight, γ.

3.3.2.2 Shear Modulus-- The shear modulus, G, defined as shown in Fig. 3300-1, shall be determined as a function of shear strain level.

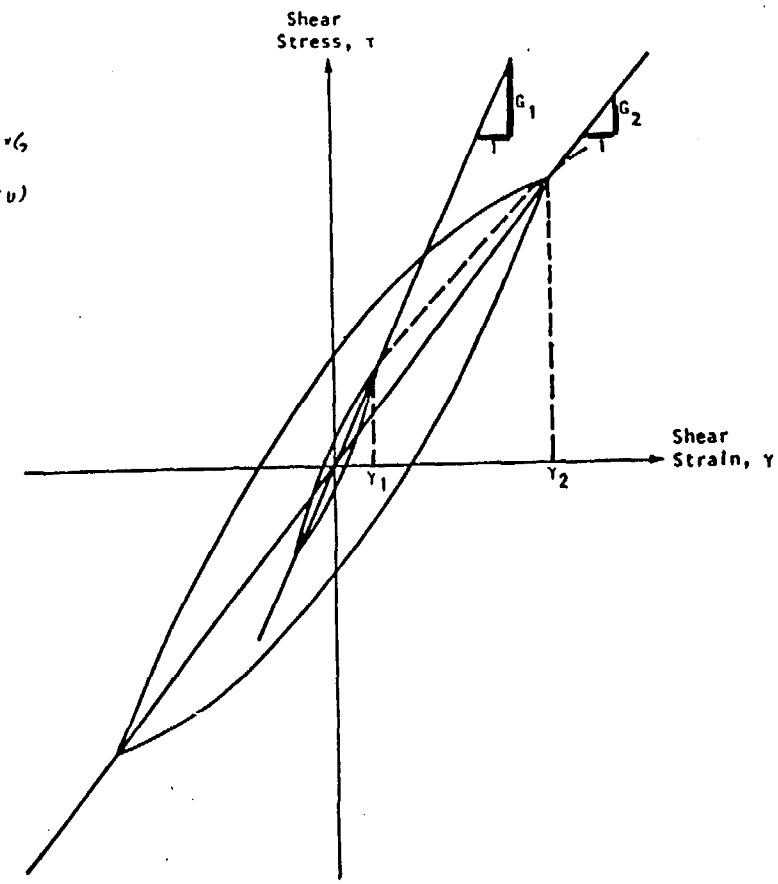


FIGURE 3300-1 DEFINITION DIAGRAM FOR SHEAR MODULUS, G

3.3.2.3 Material (Hysteretic) Damping Ratio-- (a) The material (hysteretic) damping ratio, D, defined as shown in Fig. 3300-2, shall be determined as a function of shear strain level.

(b) At very small strains ($\leq 10^{-4}$ percent), the ~~material (hysteretic) damping ratio shall not exceed [illegible] of~~ critical.

3.3.2.4 Poisson's Ratio-- Poisson's ratio, ν, in combination with shear modulus, G, defines the Young's modulus of the material in accordance with the theory of elasticity. For saturated soils, the behavior of the water phase shall be considered in evaluating Young's modulus and selecting values of ν.

3.3.3 Direct Method

SSI analysis by the direct method shall consist of the following steps:

1. Locate the bottom and lateral boundaries of the soil-structure model.
2. Establish input motion to be applied at the boundaries.
3. Establish soil model, properties, and layer boundaries to be used for the foundation.
4. Perform SSI analyses in one or two steps, as discussed in 3.1.1.2, using structural models as discussed in 3.3.1.8.

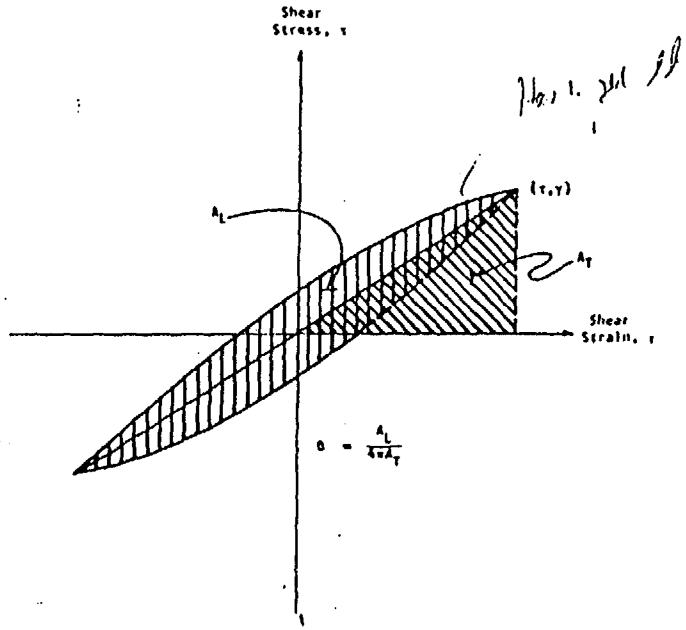


FIGURE 3300-2 DEFINITION DIAGRAM FOR HYSTERETIC DAMPING RATIO, D

3.3.3.1 Seismic Input for Model Boundaries-- (a) Boundary motion input to the soil model shall be compatible with the design earthquake specified at the finish grade in the free field.

(b) The motions shall be established as a function of the soil properties, the type of waves propagating during the earthquake, and the type of boundary assumed.

(c) The analyses to establish boundary motions shall be performed using mathematical models and procedures compatible with those used in the SSI analysis.

3.3.3.2 Lower Boundary-- The lower boundary shall be located far enough from the structure that the seismic response at points of interest is not significantly affected. The lower boundary of the model may be placed at a layer at which the shear-wave velocity equals or exceeds 3,500 ft/sec (1,100 m/sec) or at a soil layer that has a modulus 10 times or more larger than the modulus of the layer immediately below the structure foundation level. The lower boundary need not be placed more than 3 times the maximum foundation dimension below the foundation. The lower boundary may be assumed to be rigid.

3.3.3.3 Selection of Lateral Boundaries-- The location and type of lateral boundaries shall be selected so as not to significantly affect the structural response at points of interest. Elementary, viscous, or transmitting boundaries may be used.

3.3.3.4 Soil Element Size-- Soil discretization (elements or zones) shall be established to adequately reproduce static and dynamic effects. When using simple quadrilateral finite elements, at least eight horizontal discretizations over the foundation width shall be used, immediately beneath the foundation, to adequately reproduce the static stress distribution beneath the foundation. The discretization adjacent to the foundation shall be fine enough to adequately model rocking, if significant. The soil elements shall be fine enough to ensure frequency-transmitting characteristics up to a frequency of at least 25 Hz, which requires an element vertical dimension smaller than or equal to one-fifth of the smallest wavelength of interest. Larger element sizes may be used when justified.

3.3.3.5 Time Step and Frequency Increment-- (a) For solution of the SSI analysis in the time domain, the integration time step shall be selected to be small enough to ensure accuracy and stability of the solution.

(b) For solution of the SSI analysis in the frequency domain, the frequency increment shall be selected to be small enough to ensure accuracy of the solution. A quiet period shall be added to the excitation to damp out structural vibrations. The transfer functions shall be established using a sufficient number of points. ~~The cutoff frequency shall be at least 25 Hz, except a lower frequency cutoff may be used when justified.~~

3.3.4 Impedance Method

SSI analysis by the impedance function approach shall consist of the following steps:

1. Determine the input motion to the massless rigid foundation.
2. Determine the foundation impedance functions.
3. Analyze coupled soil-structure system.

3.3.4.1 Determination of Input Motion-- The control motion defined at the free-field surface may be input to the massless rigid foundation. When the control motion is used as the input, rotational input due to embedment or wave passage effects need not be considered. Alternatively, the input motion to the massless rigid foundation may be modified from the control motion at the free-field surface to incorporate embedment or wave passage effects, provided the corresponding computed rotational inputs are also used in the analysis.

3.3.4.2 Determination of Foundation Impedance Functions

3.3.4.2.1 *Equivalent Foundation Dimensions*-- For impedance function calculations, all mat foundations may be approximated by equivalent rectangular or circular shapes. The equivalent rectangular or circular dimensions shall be computed by equating the basemat soil contact area for translational modes of excitation and by equating the contact area moment of inertia with respect to the reference axis of rotation for rotational modes of excitation. The equivalent embedment depth shall be determined by equating the volume of soil displaced by the embedded structure.

3.3.4.2.2 *Uniform Soil Sites*-- When the soil below the foundation basemat is relatively uniform to a depth equal to the largest foundation dimension, frequency-independent soil spring and dashpot constants, as shown in Table 3300-1 for circular foundations and Table 3300-2 for rectangular foundations, may be used. Frequency-dependent impedance functions for a viscoelastic half-space using the integral equation formulation may also be used.

3.3.4.2.3 *Layered Soil Sites*-- Where the soil deposit can be approximated by a number of horizontal layers of uniform soil, or where the uniform soil deposit is underlain by bedrock at a depth less than the largest equivalent foundation dimensions, frequency-dependent impedance functions shall be developed. An integral equation formulation is acceptable for computing the impedance functions. The use of finite-element or finite-difference formulations is also acceptable.

3.3.4.2.4 *Embedded Foundations*-- (a) ~~For shallow embedments (depth-to-equivalent-radius ratio less than 0.3), the effect of embedment may be neglected in obtaining the~~ impedance functions, provided the soil profile and properties below the basemat elevation are used for the impedance calculations.

(b) When the effect of embedment is considered, a simplified formulation may be used that assumes that the soil reactions at the base of the foundation are equal to those of a foundation placed on the soil surface assumed at the foundation elevation and uses lateral soil reactions calculated independently using soil properties of the side soil. More accurate formulations using integral equations, finite-element methods, finite-difference methods, or a combination of these methods may also be used.

3.3.4.3 Analysis of Coupled Soil-Structure System-- (a) The coupled soil-structure system shall include the structure, or its modal representation, and the soil spring and dashpots anchored at the foundation level. The dynamic characteristics of the soil shall be defined by impedance functions computed in accordance with 3.3.4.2. The coupled soil-structure dependent. Foundation impedances depend on the soil configuration and material behavior, the frequency of the excitation, and the geometry of the foundation.

- Analysis of the coupled soil-structure system by solving the appropriate equations of motion.

The impedance-function approach is limited to linear or equivalent linear analysis, since it is based on the principle of superposition. It is typically applied to general, three-dimensional environments.

3.3.1.1 Fixed-Base Analysis-- A fixed-base condition may be assumed for soil-structure systems when the site soil conditions behave in a rock-like manner to reduce computational efforts. However, SSI analysis may always be performed.

3.3.1.2 Spatial Variations of Free-Field Motion-- The earthquake ground motion at the site is a function of the location and source mechanism of the earthquake, the transmission path, and the local site conditions. Describing the free-field ground motion entails specifying the point at which the motion is applied (the control point), the amplitude and frequency characteristics of the motion, and the spatial variations of the motion. In terms of SSI, the variation of motion over the depth and width of the foundation is the key factor. For surface foundations, the variation of motion on the surface of the soil is important; for embedded foundations, the variation of motion over both the embedment depth and the foundation width should be known. Specification of the control motion is discussed in Section 2 of the standard. Spatial variation of the free-field ground motion is discussed here.

To perform SSI analysis by either the direct method or the impedance-function approach, an assumption as to the wave-propagation characteristics of this ground motion must be made (3.3-1). The direct method requires a compatible seismic excitation on the boundaries of the model. The impedance-function approach requires determination of the motions of a massless foundation bonded to the soil. It is common to assume a horizontally stratified soil and vertically propagating trains of waves. In this case, vertically propagating shear waves produce only horizontal translations, and vertically propagating dilatational waves produce only vertical motions in the free-field soil deposit. This assumption reduces the free-field wave-propagation problem to one dimension.

In general, the pattern of wave propagation due to an earthquake is extremely complex and very uncertain. The assumption of trains of waves incident to the soil deposit free surface at angles other than vertical produces effects which can increase or decrease the structural response depending on the specific situation. Consider a massless foundation bonded to the free surface of a soil deposit for illustrative purposes. Vertically propagating shear and dilatational waves will produce only a resultant horizontal and vertical motion, respectively, of the foundation. Trains of waves incident to the surface at varying angles will produce a coupling of horizontal and torsional motion and vertical and rocking motion. The resultant effect may be a net increase or decrease in foundation motion depending on the site specificity, assumed wave trains, the foundation characteristics, and the frequency range of interest.

Refs. 3.3-4, -6, and -17 contain specific examples quantifying the effect of non-vertically incident seismic waves on in-structure response. These results span the range of increases and decreases in response. For realistic angles of incidence, the one quantity which requires consideration is the induced torsional response due to nonvertically incident waves. For design purposes, an accidental eccentricity of 5% of the structure's plan dimension accounts for this phenomenon. It is the judgment of the Committee that vertically propagating waves may be assumed for design when an accidental eccentricity is included.

For the direct method, a consistent seismic motion on the boundaries of the model must be known, assumed, or computed corresponding to the design ground motion specified at the control point. For the common assumption of vertically propagating trains of waves, a one-dimensional iterative linear wave-propagation analysis may be performed. Variations in soil material properties with strain level may be treated in an equivalent linear sense, i.e., iterate on the linear material properties to converge on a measure of the strain level over the duration of the excitation. The analysis may be either convolution or deconvolution. In the former, an excitation is specified along the boundary of the model, and the computed motion on the free-surface of the soil deposit is compared with the design specification. This is a trial-and-error process if a specified surface motion is to be matched. In the latter case, the free-surface motion is deconvolved to determine the boundary motion. In either case, the computed motions within the soil deposit exhibit amplifications and reductions in frequency content dependent on the location in the deposit and the assumed soil model.

A comparison of the design ground response spectra with the computed in-soil response spectra at the foundation depth in the free field should be made. The reduction of the in-soil response spectra at the foundation depth should be limited for design purposes to 60% of the corresponding design ground response spectra at all frequencies. When soil properties are varied in accordance with 3.3.1.7, the 60% limitation may be satisfied using the envelope of the three spectra corresponding to the three soil properties. This limitation reflects engineering judgment to account for the uncertainties in the assumptions leading to the reduction, e.g., assumed wave types, angles of incidence, soil material behavior, etc. The recording and analysis of earthquake motions at depth will assist in reducing these uncertainties in the future.

3.3.1.3 Three-Dimensional Effects-- SSI is a three-dimensional phenomenon—the soil and structure exhibit three-dimensional dynamic characteristics. The structure's supporting medium (soil or rock) is infinite in extent in two horizontal directions and the vertical direction. The dynamic behavior of this three-dimensional medium should be adequately represented in the analysis. For example, radiation damping, the geometric dispersion of energy away from the structure, is an important three-dimensional phenomenon to be included in the analysis. If two-dimensional, plane strain, approximations are made, special consideration should be given to the three-dimensional effects. In general, for deep soil sites, the plane strain approximation to the three-dimensional dynamic behavior cannot adequately represent both the stiffness and damping characteristics. The nonuniform character of the soil in the neighborhood of the site should also be considered.

Structures of a nuclear power plant facility exhibit three-dimensional dynamic behavior. Coupling between horizontal translations and torsional rotations exist even in structures nearly axisymmetric such as typical reactor buildings. This coupling should be treated in the analysis and design.

3.3.1.4 Nonlinear Behavior of Soil-- The constitutive behavior of soil with varying strain levels is clearly nonlinear as described in 3.3.2. For discussion purposes, this nonlinear behavior can be separated into two parts: Primary and secondary nonlinearities. The term "primary nonlinearity" denotes the nonlinear material behavior induced in the soil due to the excitation alone, i.e., ignoring structure response. The term "secondary nonlinearity" denotes the nonlinear material behavior induced in the soil due to the structural response as a result of SSI. The nonlinear behavior of soil should be taken into account for the SSI analysis. However, to perform rigorous nonlinear analysis of a typical nuclear power plant structure would require a fully three-dimensional model and an appropriate set of constitutive equations for soil. This is currently beyond the state of the art for design. Nonlinear soil behavior may be treated by:

- Using equivalent linear soil material properties typically determined from an iterative linear analysis of the free-field soil deposit. This accounts for the primary nonlinearity.
- Performing an iterative linear analysis of the coupled soil-structure system. This accounts for the primary and secondary nonlinearities.

Either technique is acceptable for structural response determination.

In view of the large uncertainties in describing the material behavior of soil and the SSI phenomenon, engineering judgment dictates consideration of a range of material properties for design.

3.3.1.5 Structure-to-Structure Interaction-- Structure-to-structure interaction

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

DRAFT

▲5010.64 (FRONT)

CLIENT & PROJECT	PAGE 1 OF 65
Private Fuel Storage, LLC / PFSF at Skull Valley	Plus 16 Attachment Pgs.

CALCULATION TITLE (Indicative of the Objective):	QA CATEGORY (✓)
FINITE ELEMENT ANALYSIS OF CANISTER TRANSFER BUILDING	☑ 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	Structural	SC-6	–	–

* 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
T.M. Snyder 11/25/98	William Dykstra WILLIAM DYKSTRA 12/4/98	Sean Chen Sean Chen 12/4/98	0	NA	✓ See pg. 7	
T.M. Snyder	DRAFT	DRAFT	1	0		

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CALCULATION ATTACHMENT	J.O./W.O./CALCULATION NO. 05996.02-SC-6	REVISION 1	ATTACH 6 PAGE 1

PREPARER/DATE	REVIEWER/CHECKER/DATE	INDEPENDENT REVIEWER
B. E. Ebbeson 4/01/02	T.M.Snyder 4/01/2002	Pares Datta 4/01/2002

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
PFSF / Skull Valley / Finite Element Analysis of Canister Transfer Building	I

ATTACHMENT No. 6

The purpose of this Attachment is to find the differential vertical displacement of the CTB base mat caused by vertical earthquake loads. Results will be used in the testimony of Bruce E. Ebbeson on Section D of Unified Contention L/QQ before the Atomic Safety and Licensing Board.

The load combination with the full vertical earthquake is LC 1. This combination also includes 40% of the maximum N-S and E-W seismic loads, as well as dead and live loads. Displacement along the building centerline in the N-S direction (along column line D), and in the E-W direction along column line 6 will be plotted, and difference between the maximum and minimum displacements calculated. See pages 6-2 and 6-3 for these plots.

N-S Direction:

Maximum vertical displacement = .033094 feet

Minimum vertical displacement = .019479 feet

Differential vertical displacement = (0.033094-0.019479)(12 in/ft) = 0.163 inches

E-W Direction:

Maximum vertical displacement = .035367 feet

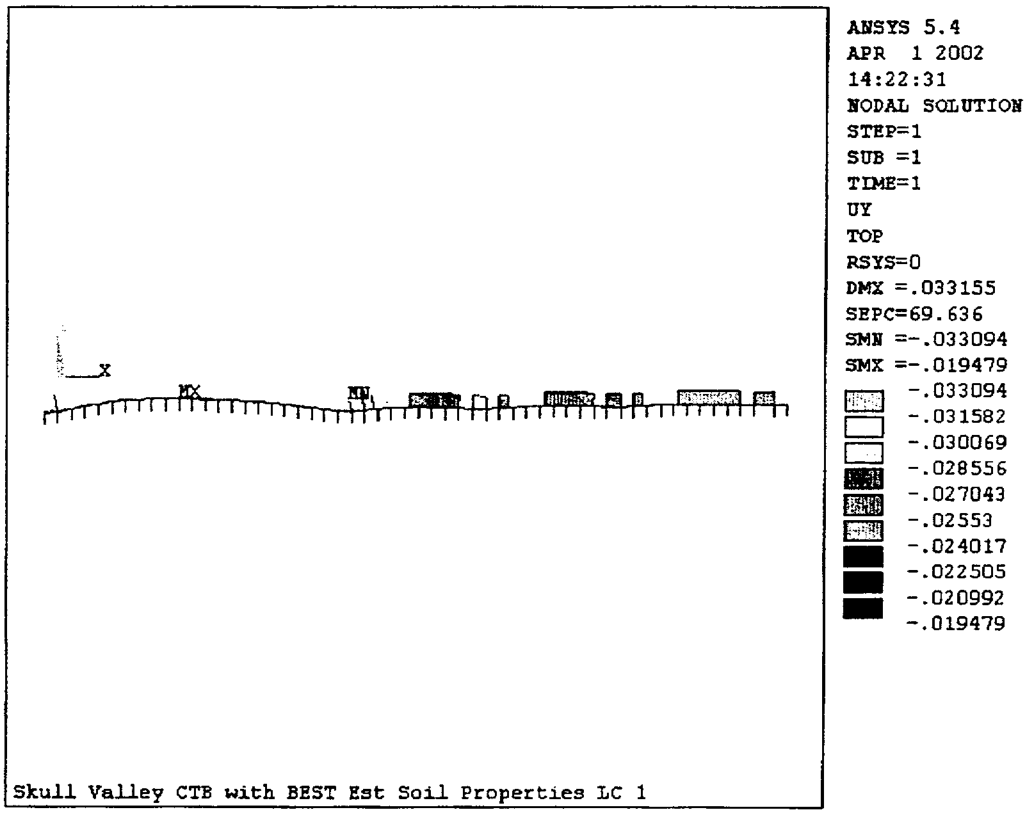
Minimum vertical displacement = .007579 feet

Differential vertical displacement = (0.035367-0.007579)(12 in/ft) = 0.333 inches.

It should be noted that these values are conservative because:

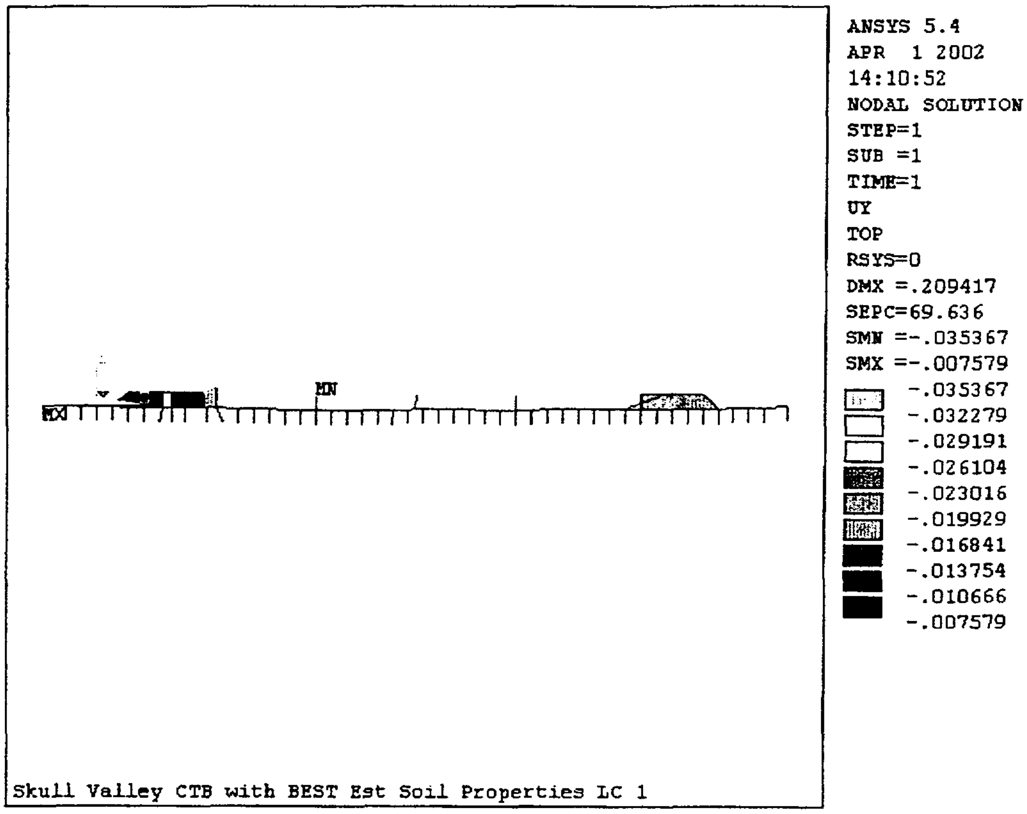
- They contain contribution from the dead and live loads
- They contain rigid body rotations caused by the horizontal seismic loads.

ATTACHMENT No. 6



SECTION CUT OF BASE MAT ALONG D-LINE
VIEW FACING WEST
(99' < Z < 103')
(-6' < Y < 6')

ATTACHMENT No. 6



SECTION CUT OF BASE MAT ALONG 6-LINE
VIEW FACING NORTH
(-6' < Y < 6')
(X = 135')

TABLE 5.1-1
(Sheet 1 of 2)

ANTICIPATED TIME AND PERSONNEL REQUIREMENTS FOR HI-STORM CANISTER TRANSFER OPERATIONS

OPERATION	NO. OF PERSONNEL[1]	TASK DURATION (HOURS)
1. Receive and inspect shipment. Measure dose rates.	3	0.5
2. Move shipment into Canister Transfer Building.	4	0.5
3. Remove personnel barrier, measure cask dose rates, and perform contamination survey.	3	1.6
4. Remove impact limiters and tiedowns.	3	1.5
5. Attach lifting yoke to crane and HI-STAR shipping cask. Upright HI-STAR cask and move to transfer cell. Connect support struts.	3	1.0
6. Sample enclosed cask gas and vent.	2	0.5
7. Remove HI-STAR closure plate bolts.	3	1.0
8. Remove HI-STAR closure plate (lid).	3	0.2
9. Prep HI-STAR to mate with HI-TRAC transfer cask.	3	0.2
10. Install canister lift cleats and attach slings.	3	1.0
11. Attach lifting yoke to crane and HI-TRAC.	3	0.5
12. Mount HI-TRAC on top of HI-STAR. Connect support struts to HI-TRAC.[2]	3	0.5
13. Open HI-TRAC transfer cask doors.	3	0.2
14. Attach slings to canister downloader hoist and raise canister.	3	0.5
15. Close HI-TRAC doors and install pins.	3	0.2
16. Lower canister onto HI-TRAC doors.	3	0.2
17. Prep HI-STORM storage cask to mate with HI-TRAC transfer cask. Disconnect support struts.[2]	3	0.2
18. Move HI-TRAC from HI-STAR to HI-STORM. Attach support struts to HI-TRAC.[2]	3	0.7
19. Raise canister and open HI-TRAC doors.	3	0.5
20. Lower canister into HI-STORM storage cask.	3	0.5

TABLE 5.1-1
(Sheet 2 of 2)

ANTICIPATED TIME AND PERSONNEL REQUIREMENTS FOR HI-STORM CANISTER TRANSFER OPERATIONS

OPERATION	NO. OF PERSONNEL[1]	TASK DURATION (HOURS)
21. Disconnect lifting slings.	3	0.2
22. Close transfer cask doors.	3	0.2
23. Disconnect support struts.[2] Remove HI-TRAC from HI-STORM	3	0.5
24. Remove canister lift cleats.	3	0.5
25. Install HI-STORM lid and lid bolts.	3	1.0
26. Perform dose survey and install HI-STORM lifting eyes.	3	0.5
27. Drive cask transporter in transfer cell.	2	0.3
28. Connect HI-STORM to cask transporter.	3	0.5
29. Raise HI-STORM storage cask.	3	0.2
30. Transport HI-STORM cask to storage pad.	3	2.0
31. Position and lower HI-STORM cask on pad.	3	0.5
32. Disconnect HI-STORM cask from transporter and remove cask lifting eyes.	3	1.0
33. Connect cask temperature instrumentation.	3	0.5
34. Perform cask operability tests.	2	48
Total Hours	-	19.9[3]

Notes

1. Number of personnel typically includes 2 to 3 operators and 1 HP technician.
2. While the HI-TRAC transfer cask is connected to the crane, it is not necessary to attach the seismic support struts to the transfer cask, since connection of the crane to the transfer cask provides assurance that the transfer cask cannot topple in the event of an earthquake. However, prior to disconnecting the crane from the transfer cask, the support struts must be connected to the transfer cask.
3. Total does not reflect 48 hour duration in Step 34, which is time required for cask temperature to reach equilibrium. Personnel time required to monitor temperatures during the equilibrium phase is minimal.

Table 10.3.3a

MPC TRANSFER INTO THE HI-STORM 100 SYSTEM DIRECTLY FROM TRANSPORT USING THE 125-TON HI-TRAC TRANSFER CASK

ESTIMATED OPERATIONAL EXPOSURES† (45,000 MWD/MTU, 9-YEAR COOLED PWR FUEL)

ACTION	CHAPTER 8 STEP	DURATION (MINUTES)	OPERATOR LOCATION (FIGURE 10.3.1)	NUMBER OF OPERATORS	DOSE RATE AT OPERATOR LOCATION (MREM/HR)	DOSE TO INDIVIDUAL (MREM/HR)	TOTAL DOSE (PERSON-MREM)	ASSUMPTIONS
Section 8.5.2								
MEASURE HI-STAR DOSE RATES	1	16	17A	2	14.1	3.8	7.5	16 POINTS@1 POINT/MIN
REMOVE PERSONNEL BARRIER	2	10	17C	2	21.5	3.6	7.2	ATTACH SLING REMOVE 8 LOCKS
PERFORM REMOVABLE CONTAMINATION SURVEYS	3	1	17C	1	21.5	0.4	0.4	10 SMEARS@10 SMEARS/MINUTE
REMOVE IMPACT LIMITERS	4	16	17A	2	14.1	3.8	7.5	ATTACH FRAME REMOVE 22 BOLTS IMPACT TOOLS
REMOVE TIE-DOWN	5	6	17A	2	14.1	1.4	2.8	ATTACH 2-LEGGED SLING REMOVE 4 BOLTS
PERFORM A VISUAL INSPECTION OF OVERPACK	6	10	17B	1	9	1.5	1.5	CHECKSHEET USED
REMOVE REMOVABLE SHEAR RING SEGMENTS	7	4	17A	1	14.1	0.9	0.9	4 BOLTS EACH @2/MIN X 2 SEGMENTS
UPEND HI-STAR OVERPACK	8	20	17B	2	9	3.0	6.0	DISCONNECT LIFT YOKE
INSTALL TEMPORARY SHIELD RING SEGMENTS	9	16	18A	1	7.9	2.1	2.1	8 SEGMENTS @ 2 MIN/SEGMENT
FILL TEMPORARY SHIELD RING SEGMENTS	9	25	18A	1	7.9	3.3	3.3	230 GAL @10GPM, LONG HANDLED SPRAYER
REMOVE OVERPACK VENT PORT COVER PLATE	10.a	2	18A	1	7.9	0.3	0.3	4 BOLTS @2/MIN
ATTACH BACKFILL TOOL	10.a	2	18A	1	7.9	0.3	0.3	4 BOLTS @2/MIN
OPEN/CLOSE VENT PORT PLUG	10.c	0.5	18A	1	7.9	0.1	0.1	SINGLE TURN BY HAND NO TOOLS
REMOVE CLOSURE PLATE BOLTS	12	39	18A	2	7.9	5.1	10.3	52 BOLTS@4/MIN X 3 PASSES

† See notes at bottom of Table 10.3.4.

Table 10.3.3a (Continued)
MPC TRANSFER INTO THE HI-STORM 100 SYSTEM DIRECTLY FROM TRANSPORT USING THE 125-TON HI-TRAC TRANSFER CASK
ESTIMATED OPERATIONAL EXPOSURES† (45,000 MWD/MTU, 9-YEAR COOLED PWR FUEL)

ACTION	CHAPTER 8 STEP	DURATION (MINUTES)	OPERATOR LOCATION (FIGURE 10.3.1)	NUMBER OF OPERATORS	DOSE RATE AT OPERATOR LOCATION (MREM/HR)	DOSE TO INDIVIDUAL (MREM/HR)	TOTAL DOSE (PERSON-MREM)	ASSUMPTIONS
REMOVE OVERPACK CLOSURE PLATE	12	2	18A	1	7.9	0.3	0.3	4 SHACKLES@2/MIN
INSTALL HI-STAR SEAL SURFACE PROTECTOR	13	2	19B	1	7.9	0.3	0.3	PLACED BY HAND NO TOOLS
INSTALL TRANSFER COLLAR ON HI-STAR	14	10	19B	2	7.9	1.3	2.6	ALIGN AND POSITION REMOVE 4 SHACKLES
REMOVE MPC LIFT CLEAT HOLE PLUGS	15	2	19A	1	150.9	5.0	5.0	4 PLUGS AT 2/MIN NO TORQUING
INSTALL MPC LIFT CLEATS AND LIFT SLING	16	25	19A	2	150.9	62.9	125.8	INSTALL CLEATS AND HYDRO TORQUE 4 BOLTS
MATE OVERPACKS	21	10	20B	2	27.4	4.6	9.1	ALIGNMENT GUIDES USED
REMOVE DOOR LOCKING PINS AND OPEN DOORS	22	4	20B	2	27.4	1.8	3.7	2 PINS@2/MIN
INSTALL TRIM PLATES	23	4	20B	2	27.4	1.8	3.7	INSTALLED BY HAND NO FASTENERS
REMOVE TRIM PLATES	26	4	20B	2	27.4	1.8	3.7	INSTALLED BY HAND NO FASTENERS
CLOSE HI-TRAC DOORS AND INSTALL DOOR LOCKING PINS	27	4	20B	2	27.4	1.8	3.7	2 PINS@2/MIN
MATE OVERPACKS	30	10	13B	2	27.4	4.6	9.1	ALIGNMENT GUIDES USED
ATTACH MPC LIFT SLINGS TO MPC LIFT CLEATS	30	10	13A	2	52.3	8.7	17.4	2 SLINGS@5MIN/SLING NO TOOLS
REMOVE TRANSFER LID DOOR LOCKING PINS AND OPEN DOORS	30	4	13B	2	27.4	1.8	3.7	2 PINS@2/MIN
INSTALL TRIM PLATES	30	4	13B	2	27.4	1.8	3.7	INSTALLED BY HAND NO FASTENERS

† See notes at bottom of Table 10.3.4.

Table 10.3.3a (Continued)
MPC TRANSFER INTO THE HI-STORM 100 SYSTEM DIRECTLY FROM TRANSPORT USING THE 125-TON HI-TRAC TRANSFER CASK
ESTIMATED OPERATIONAL EXPOSURES† (45,000 MWD/MTU, 9-YEAR COOLED PWR FUEL)

ACTION	CHAPTER 8 STEP	DURATION (MINUTES)	OPERATOR LOCATION (FIGURE 10.3.1)	NUMBER OF OPERATORS	DOSE RATE AT OPERATOR LOCATION (MREM/HR)	DOSE TO INDIVIDUAL (MREM/HR)	TOTAL DOSE (PERSON-MREM)	ASSUMPTIONS
DISCONNECT SLINGS FROM MPC LIFTING DEVICE	30	10	13A	2	52.3	8.7	17.4	2 SLINGS@5/MIN
REMOVE TRIM PLATES	30	4	13B	2	27.4	1.8	3.7	INSTALLED BY HAND NO FASTENERS
REMOVE MPC LIFT CLEATS AND MPC LIFT SLINGS	30	10	14A	1	150.9	25.2	25.2	4 BOLTS,NO TORQUING
INSTALL HOLE PLUGS IN EMPTY MPC BOLT HOLES	30	2	14A	1	150.9	5.0	5.0	4 PLUGS AT 2/MIN NO TORQUING
REMOVE HI-STORM VENT DUCT SHIELD INSERTS	30	2	15A	1	6.3	0.2	0.2	4 SHACKLES@2/MIN
REMOVE ALIGNMENT DEVICE	30	4	15A	1	6.3	0.4	0.4	REMOVED BY HAND NO TOOLS (4 PCS@1/MIN)
INSTALL HI-STORM LID AND INSTALL LID STUDS/NUTS	30	25	16A	2	2.4	1.0	2.0	INSTALL LID AND HYDRO TORQUE 4 BOLTS
INSTALL HI-STORM EXIT VENT GAMMA SHIELD CROSS PLATES	30	4	16B	1	19.1	1.3	1.3	4 PCS @ 1/MIN INSTALL BY HAND NO TOOLS
INSTALL THERMOCOUPLES	30	20	16B	1	19.1	6.4	6.4	4@5MIN/THERMOCOUPLE
INSTALL EXIT VENT SCREENS	30	20	16B	1	19.1	6.4	6.4	4 SCREENS@5MIN/SCREEN
REMOVE HI-STORM LID LIFTING DEVICE	30	2	16A	1	2.4	0.1	0.1	4 SHACKLES@2/MIN
INSTALL HOLE PLUGS IN EMPTY HOLES	30	2	16A	1	2.4	0.1	0.1	4 PLUGS AT 2/MIN NO TORQUING
PERFORM SHIELDING EFFECTIVENESS TESTING	31	16	16D	1	9.6	2.6	2.6	16POINTS@1 MIN

† See notes at bottom of Table 10.3.4.

Table 10.3.3a (Continued)
MPC TRANSFER INTO THE HI-STORM 100 SYSTEM DIRECTLY FROM TRANSPORT USING THE 125-TON HI-TRAC TRANSFER CASK
ESTIMATED OPERATIONAL EXPOSURES† (45,000 MWD/MTU, 9-YEAR COOLED PWR FUEL)

ACTION	CHAPTER 8 STEP	DURATION (MINUTES)	OPERATOR LOCATION (FIGURE 10.3.1)	NUMBER OF OPERATORS	DOSE RATE AT OPERATOR LOCATION (MREM/HR)	DOSE TO INDIVIDUAL (MREM/HR)	TOTAL DOSE (PERSON-MREM)	ASSUMPTIONS
SECURE HI-STORM TO TRANSPORT DEVICE	30	10	16A	1	2.4	0.4	0.4	ASSUMES AIR PAD
TRANSFER HI-STORM TO ITS DESIGNATED STORAGE LOCATION	30	40	16C	1	6.6	4.4	4.4	200 FEET @ 4FT/MIN
INSERT HI-STORM LIFTING JACKS	30	4	16D	1	9.6	0.6	0.6	4 JACKS@1/MIN
REMOVE AIR PAD	30	5	16D	1	9.6	0.8	0.8	1 PAD MOVED BY HAND
REMOVE HI-STORM LIFTING JACKS	30	4	16D	1	9.6	0.6	0.6	4 JACKS@1/MIN
INSTALL INLET VENT SCREENS	30	20	16D	1	9.6	3.2	3.2	4 SCREENS@5MIN/SCREEN
PERFORM AIR TEMPERATURE RISE TEST	32	8	16B	1	19.1	2.5	2.5	8 MEASMT@1/MIN
TOTAL							324.9 PERSON-MREM	

† See notes at bottom of Table 10.3.4.

TABLE 3.4-1

QUALITY ASSURANCE CLASSIFICATION OF STRUCTURES, SYSTEMS, AND COMPONENTS

IMPORTANT TO SAFETY	NOT IMPORTANT TO SAFETY
Classification Category A	Storage Facility Infrastructure
Spent Fuel Canister	Security and Heath Physics Building
	Administration Building
Classification Category B	Operations and Maintenance Building
Storage Cask	Intrusion Detection System
Transfer Cask	CCTV System
Associated Lifting Devices	Restricted Area Lighting
Canister Transfer Building	Security Alarm Stations
Canister Transfer Overhead Bridge Crane	Electrical Power - UPS
Canister Transfer Semi-gantry Crane	Electrical Power - Backup Diesel Generator
Seismic Support Struts	Electrical Power - Normal
	Yard/Building Lighting
Classification Category C	Cask Transporter
Cask Storage Pads	Radiation Monitors
	Temperature Monitoring System
	Communication Systems
	Fire Detection/Suppression
	Water Supply Systems
	Septic Systems
	Access Road
	Road Transport Components
	Railroad Line Components

Classification Category A - Critical to Safe Operation

Category A items include SSCs whose failure or malfunction could directly result in a condition adversely affecting public health and safety. The failure of a single item could cause loss of primary containment leading to release of radioactive material, loss of shielding, or unsafe geometry compromising criticality control.

Classification Category B - Major Impact on Safety

Category B items include SSCs whose failure or malfunction could indirectly result in a condition adversely affecting public health and safety. The failure of a Category B item, in conjunction with the failure of an additional item, could result in an unsafe condition.

Classification Category C - Minor Impact on Safety

Category C items include SSCs whose failure or malfunction would not significantly reduce the packaging effectiveness and would not be likely to create a situation adversely affecting public health and safety.

The QA determination for the SSCs that are classified as Important to Safety are discussed in the following sections. A QA classification for these SSCs establishes the requirements that satisfy 10 CFR 72.122(a) general design criteria, which specifies SSCs Important to Safety be designed, fabricated, erected, and tested to quality standards.

INCH-POUND

DOE-STD-1020-94
April 1994
Change Notice #1
January 1996

DOE STANDARD

NATURAL PHENOMENA HAZARDS DESIGN AND EVALUATION CRITERIA FOR DEPARTMENT OF ENERGY FACILITIES

U.S. Department of Energy
Washington, D.C. 20585

AREA FACR

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40521

Foreword

Change notice #1 has been included in this standard to provide information to help meet the requirements in DOE Order 420.1 and its associated implementation guides, accounting for the cancellation of DOE Order 6430.1A, correcting errors in the prevoius standard, and updating this standard to the most current references.

This DOE standard is approved for use by all departments and contractors of the Department of Energy (DOE). This Standard will still apply when DOE Order 420.1 is converted to a rule. In addition, this Standard will still apply when other referenced Orders such as 5480.23, the SAR Order, 5480.22, the TSR Order, etc. are converted to rules.

There is an established hierarchy in the set of documents that specify NPH requirements. In this hierarchy, DOE Order 420.1 is the highest authority. The next set of controlling documents are the associated implementation guides followed by the set of NPH standards. In the event of conflicts in the information provided by these documents, the information provided in the document of higher authority should be utilized (e.g., the definitions provided in the implementation guides should be utilized even though corresponding definitions are provided in the NPH standards).

The Department of Energy (DOE) has issued an Order 420.1 which establishes policy for its facilities in the event of natural phenomena hazards (NPH) along with associated NPH mitigation requirements. This DOE Standard gives design and evaluation criteria for NPH effects as guidance for implementing the NPH mitigation requirements of DOE Order 420.1 and the associated implementation Guides. These are intended to be consistent design and evaluation criteria for protection against natural phenomena hazards at DOE sites throughout the United States. The goal of these criteria is to assure that DOE facilities can withstand the effects of natural phenomena such as earthquakes, extreme winds, tornadoes, and flooding. These criteria apply to the design of new facilities and the evaluation of existing facilities. They may also be used for modification and upgrading of existing facilities as appropriate. It is recognized that it is likely not cost-effective to upgrade existing facilities which do not meet these criteria by a small margin. Hence, flexibility in the criteria for existing facilities is provided by permitting limited relief from the criteria for new design. The intended audience is primarily the civil/structural or mechanical engineers familiar with building code methods who are conducting the design or evaluation of DOE facilities.

The design and evaluation criteria presented herein control the level of conservatism introduced in the design/evaluation process such that earthquake, wind, and flood hazards are treated on a consistent basis. These criteria also employ a graded approach to ensure that the level of conservatism and rigor in design/evaluation is appropriate for facility characteristics such as importance, hazards to people on and off site, and threat to the environment. For each natural phenomena hazard covered, these criteria consist of the following:

1. Performance Categories and target performance goals as specified in the DOE Order 420.1 NPH Implementation Guide, and DOE-STD-1021.

2. Specified probability levels from which natural phenomena hazard loading on structures, equipment, and systems is developed.

3. Design and evaluation procedures to evaluate response to NPH loads and criteria to assess whether or not computed response is permissible.

Table of Contents

40526

40527

Chapter 1
Introduction

1.1 Overview of DOE Natural Phenomena Hazards Order, Standards, and Guidance

It is the policy of the Department of Energy (DOE) to design, construct, and operate DOE facilities so that workers, the general public, and the environment are protected from the impacts of natural phenomena hazards on DOE facilities. DOE Order 420.1, "Facility Safety" (Ref. 1-1) and the associated Implementation Guides, "Implementation Guide for the Mitigation of Natural Phenomena Hazards for DOE Nuclear Facilities and Non-nuclear Facilities" (Ref. 1-2), "Implementation Guide for Nonreactor Nuclear Safety Design Criteria and Explosives Safety Criteria" (Ref. 1-3), and "Implementation Guide for use with DOE Orders 420 and 470 Fire Safety Program" (Ref. 1-4) identify the responsibilities and requirements to execute this policy in a consistent manner throughout DOE which includes: (1) providing safe work places; (2) protecting against property loss and damage; (3) maintaining operation of essential facilities; and (4) protecting against exposure to hazardous materials during and after occurrences of natural phenomena hazards. There is an established hierarchy in the set of documents that specify NPH requirements. In this hierarchy, DOE Order 420.1 is the highest authority. The next set of controlling documents are the associated Implementation Guides followed by the set of NPH standards. The NPH requirements have been developed to provide the necessary information that assess the NPH safety basis for DOE facilities, which is documented in Safety Analysis Reports (SARs), if available. DOE 5480.23 (Ref. 1-5) and the guidance provided in the associated Standard, DOE-STD-3009-94 (Ref. 1-6) prescribed the use of a graded approach for the effort expended in safety analysis and the level of detail presented in associated documentation. DOE NPH mitigation requirements are also consistent with the National Earthquake Hazards Reduction Program and Executive Orders 12699 (Ref. 1-7) and 12941 (Ref. 1-8).

The overall approach for NPH mitigation shall be consistent with the graded approach embodied in the SAR. The application of NPH design requirements to structures, systems, and components (SSCs) shall be based on the life-safety or the safety classifications for the SSCs as established by safety analysis. The application of the most rigorous design requirements should be limited to those SSCs classified by safety analysis as Safety-Class or Safety-Significant consistent with DOE-STD-3009-94. Although DOE-STD-3009-94 is specifically

40530

applicable to non-reactor nuclear facilities, it is DOE's intention to apply DOE-STD-3009-94 definitions for "Safety-Class" and "Safety-Significant" to all nuclear reactor and other hazardous facilities, and this broader approach is applied here. Mission importance and economic considerations should also be used to categorize SSCs which require NPH design. Once the SSCs have been classified, DOE Order 420.1 and the associated Implementation Guides specifies the NPH requirements to ensure that the SSCs are adequately designed to resist NPH. The NPH requirements utilize a graded approach in order to provide a reasonable level of NPH protection for the wide variety of DOE facilities. A graded approach is one in which various levels of NPH design, evaluation and construction requirements of varying conservatism and rigor are established ranging from common practice for conventional facilities to practices used for more hazardous critical facilities.

Five DOE Standards have been developed to provide specific acceptance criteria for various aspects of NPH to meet the requirements of DOE Order 420.1 and the associated Implementation Guides. These requirements should be used in conjunction with the NPH Implementation Guide and other pertinent documents which provide more detailed methods on specific NPH design and evaluation subjects such as DOE guidance documents, consensus national standards, model building codes, and industry accepted codes and specifications. Figure 1-1 presents a conceptual NPH design framework which identifies how the DOE NPH standards are used to assess NPH design requirements.

The following national consensus codes and standards have been referred to in this standard:

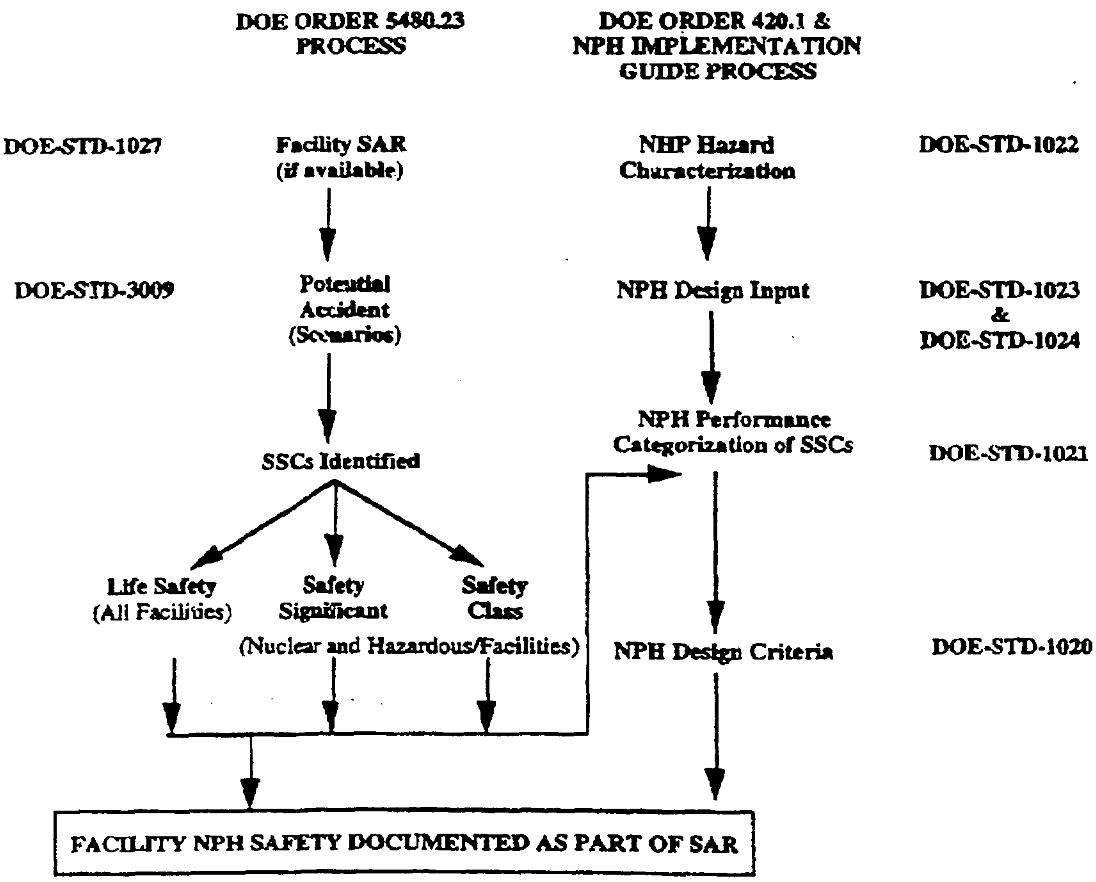
ACI 318	—	Building Code Requirements for Reinforced Concrete
ACI 349	—	Code Requirements for Nuclear Safety-Related Concrete Structures
AISC N690	—	Nuclear Facilities - Steel Safety Related Structures for Design, Fabrication, and Erection
AISC (LRFD)	—	Manual of Steel Construction, Load & Resistance Factor Design
AISC (ASD)	—	Manual of Steel Construction, Allowable Stress Design
ASCE 4	—	Seismic Analysis of Safety-Related Nuclear Structures
ASCE 7	—	Minimum Design Loads for Buildings and Other Structures
ASME	—	Boiler and Pressure Vessel Code
ATC-14	—	Evaluating the Seismic Resistance of Existing Buildings
ATC-22	—	A Handbook for Seismic Evaluation of Existing Buildings
IEEE 344	—	IEEE Recommended Practice for Seismic Qualification of Class IE Equipment for Nuclear Power Generating Stations
UBC	—	Uniform Building Code

NBC	—	National Building Code
SBC	—	Standard Building Code
FEMA 222A	—	NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings
ICSSC RP3	—	Guidelines for Identification and Mitigation of Seismically Hazardous of Existing Federal Buildings
ICSSC RP4	—	Standards of Seismic Safety for Existing Federally Owned or Leased Buildings
ICSSC RP5	—	ICSSC Guidance on Implementing Executive Order 12941 on Seismic Safety of Existing Federally Owned or Leased Buildings

Figure 1-1

Natural Phenomena Design Input

Conceptual Framework



40532

The NPH Implementation Guide of DOE Order 420.1 has established Performance Categories and target probabilistic performance goals for each category. Performance goals are expressed as the mean annual probability of exceedance of acceptable behavior limits of structures and equipment due to the effects of natural phenomena. Five Performance Categories (PC) have been established in the NPH Implementation Guide of DOE Order 420.1. Performance Categories and performance goals range from those for conventional buildings to those for facilities with hazardous materials for operations. The selection of NPH Performance Categories for SSCs is dependent on several factors including the overall risk of facility operation and the assigned function to the SSC. An SSC's safety classification is based on its function in accident prevention or mitigation as determined by safety analysis. The safety classification should be applied to specific SSCs on a case-by-case basis and need not apply to an entire facility. Experience to date has demonstrated that only a few nuclear facilities are likely to contain Safety-Class SSCs. This indicates that most SSCs in nuclear facilities should be assigned to NPH Performance Category 3 and lower. DOE is revisiting the approach used to assign NPH Performance Categories, and is likely to develop a direct link between NPH Performance Categories and accident dose (radiological or toxicological) criteria. Once this is completed, DOE-STD-1021 will be revised as necessary. The use of NPH Performance Category 4 should be reserved for those facilities whose accident dose potential is similar to that of commercial nuclear reactors.

1.2 Overview of the NPH Design and Evaluation Criteria

This natural phenomena hazard standard (DOE-STD-1020), developed from UCRL-15910 (Ref. 1-9), provides criteria for design of new structures, systems, and components (SSCs) and for evaluation, modification, or upgrade of existing SSCs so that Department of Energy (DOE) facilities safely withstand the effects of natural phenomena hazards (NPHs) such as earthquakes, extreme winds, and flooding. DOE-STD-1020 provides consistent criteria for all DOE sites across the United States. These criteria are provided as the means of implementing DOE Order 420.1 and the associated Implementation Guides, and Executive Orders 12699 and 12941 for earthquakes.

The design and evaluation criteria presented in this document provide relatively straightforward procedures to evaluate, modify, or upgrade existing facilities or to design new facilities for the effects of NPHs. The intent is to control the level of conservatism in the design/evaluation process such that: (1) the hazards are treated consistently; and (2) the level of conservatism is appropriate for structure, system, and component (SSC) characteristics related to safety, environmental protection, importance, and cost. The requirements for each hazard are presented in subsequent chapters. Terminology, guidelines, and commentary material are included in appendices which follow the requirement chapters.

Prior to applying these criteria, SSCs will have been placed in one of five Performance Categories ranging from PC-0 to PC-4. No special considerations for NPH are needed for PC-0; therefore, no guidance is provided. Different criteria are provided for the remaining four Performance Categories, each with a specified performance goal. Design and evaluation criteria aimed at target probabilistic performance goals require probabilistic natural phenomena hazard assessments. NPH loads are developed from such assessments by specifying natural phenomena hazard mean annual probabilities of exceedance. Performance goals may then be achieved by using the resulting loads combined with deterministic design and evaluation procedures that provide a consistent and appropriate level of conservatism. Design/Evaluation procedures conform closely to industry practices using national consensus codes and standards so that the procedures will be easily understood by most engineers. Structures, systems, and components comprising a DOE facility are to be assigned to a Performance Category utilizing the approach described in the DOE performance categorization standard (Ref. 1-10). These design and evaluation criteria (DOE-STD-1020) are the specific provisions to be followed such that the performance goal associated with the Performance Category of the SSC under consideration is achieved. For each category, the criteria include the following steps:

1. NPH loads are determined at specified NPH probabilities as per DOE-STD-1023 (Ref. 1-11).
2. Design and evaluation procedures are used to evaluate SSC response to NPH loads.
3. Criteria are used to assess whether or not computed response in combination with other design loads is permissible.
4. Design detailing provisions are implemented so that the expected performance during a potential NPH occurrence will be achieved.
5. Quality assurance and peer review are applied using a graded approach.

For each Performance Category, target performance goals are provided in the NPH Implementation Guide of DOE Order 420.1 in terms of mean annual probability of exceedance of acceptable behavior limits. In Item 1, the annual probability of exceedance of an NPH parameter such as ground acceleration, wind speed, or water elevation is specified. The level of conservatism in Items 2, 3, 4, and 5 above is controlled such that sufficient risk reduction from the specified NPH probability is achieved so that the target performance goal probability is met. DOE-STD-1020 provides an integrated approach combining definition of loading due to natural phenomena hazards, response evaluation methods, acceptance criteria, and design detailing requirements.

Performance goals and NPH levels are expressed in probabilistic terms; design and evaluation procedures are presented deterministically. Design/evaluation procedures specified in this document conform closely to common standard practices so that most engineers will readily understand them. The intended audience for these criteria is the civil/structural or mechanical engineer conducting the design or evaluation of facilities. These NPH design and evaluation criteria do not preclude the use of probabilistic or alternative design or evaluation approaches if these approaches meet the specified performance goals.

1.3 Evaluation of Existing Facilities

Evaluations of existing SSCs must follow or, at least, be measured against the NPH criteria provided in this document. For SSCs not meeting these criteria and which cannot be easily remedied, budgets and schedule for required strengthening must be established on a prioritized basis. A back-fit analysis should be conducted. Prioritization criteria for evaluation and upgrade of existing DOE facilities are currently being developed. Priorities should be established on the basis of Performance Category, cost of strengthening, and margin between as-is SSC capacity and the capacity required by the criteria. For SSCs which are close to meeting criteria, it is probably not cost effective to strengthen the SSC in order to obtain a small reduction in risk. As a result, some relief in the criteria is allowed for evaluation of existing SSCs. It is permissible to perform such evaluations using natural phenomena hazard exceedance probability of twice the value specified for new design. For example, if the natural phenomena hazard annual probability of exceedance for the SSC under consideration was 10^{-4}, it would be acceptable to reconsider the SSC at hazard annual probability of exceedance of $2x10^{-4}$. This would have the effect of slightly reducing the seismic, wind, and flood loads in the SSC evaluation by about 10% to 20%. This amount of relief is within the tolerance of meeting the target performance goals and is only a minor adjustment of the corresponding NPH design and evaluation criteria. In addition, it is consistent with the intent of the Federal Program (Ref. 1-8) being developed by the Interagency Committee on Seismic Safety in Construction. The Implementation Guide provides guidance for facilities with a remaining service life of less than 5 years.

1.4 Quality Assurance and Peer Review

All DOE structures, systems, and components must be designed or evaluated utilizing a formal quality assurance plan as required by 10 CFR 830.120 (Ref. 1-12). The QA and peer review should be conducted within the framework of a graded approach with increasing level of rigor employed from Performance Category 1 to 4. Specific details about a formal quality assurance plan for NPH design and evaluation should be similar to the seismic plan described in the Commentary, Appendix C. The major features of a thorough quality assurance plan for design or evaluation for natural phenomena hazards are described below.

In general, it is good practice for a formal quality assurance plan to include the following requirements. On the design drawings or evaluation calculations, the engineer must describe the NPH design basis including (1) description of the system resisting NPH effects and (2) definition of the NPH loading used for the design or evaluation. Design or evaluation calculations should be checked for numerical accuracy and for theory and assumptions. For new construction, the engineer should specify a program to test materials and inspect construction. In addition, the engineer should review all testing and inspection reports and visit the site periodically to observe compliance with plans and specifications.

For Performance Categories 2, 3, and 4, NPH design or evaluation must include independent peer review. The peer review is to be performed by independent, qualified personnel. The peer reviewer must not have been involved in the original design or evaluation. If the peer reviewer is from the same company/organization as the designer/evaluator, he must not be part of the same program where he could be influenced by cost and schedule consideration. Individuals performing peer reviews must be degreed civil/mechanical engineers with 5 or more years of experience in NPH evaluation.

For more information concerning the implementation of a formal engineering quality assurance program and peer review, Chapter 19 of Reference 1-9 should be consulted. This reference should also be consulted for information on a construction quality assurance program consistent with the implementation of the engineering quality assurance program.

1.5 References

1-1. U. S. Department of Energy, **Facility Safety**, DOE Order 420.1, Washington, DC, October 13, 1995.

1-2. U. S. Department of Energy, **Implementation Guide for the Mitigation of Natural Phenomena Hazards for DOE Nuclear Facilities and Non-Nuclear Facilities** (draft for interim use), Washington, DC, November 13, 1995.

1-3. U. S. Department of Energy, **Implementation Guide for Nonreactor Nuclear Safety Design Criteria and Explosives Safety Criteria** (draft for interim use), Washington, DC, November 13, 1995.

1-4. U. S. Department of Energy, **Implementation Guide for use with DOE Orders 420.1 and 440.1 Fire Safety Program**, Washington, DC, November 13, 1995.

1-5. U. S. Department of Energy, **Nuclear Safety Analysis Reports**, DOE Order 5480.23, Washington, DC, April 30, 1992.

1-6. U. S. Department of Energy, **Preparation Guide For U. S. Department of Energy Nonreactor Nuclear Facility Safety Analysis Reports**, DOE-STD-3009-94, Washington, DC, July 1994.

1-7. **Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction**, Executive Order 12699, Washington, DC, January 5, 1990.

1-8. **Seismic Safety of Existing Federally Owned or Leased Buildings**, Executive Order 12941, Washington, DC, December 1, 1994.

1-9. Kennedy, R.P., S.A. Short, J.R. McDonald, M.W. McCann, R.C. Murray, J.R. Hill, **Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards**, UCRL-15910, Lawrence Livermore National Laboratory, Livermore, CA, June 1990. (Superceded)

1-10. U.S. Department of Energy, **Performance Categorization Criteria for Structures, Systems, and Components at DOE Facilities Subjected to Natural Phenomena Hazards**, DOE-STD-1021-93, Washington, DC, July 1993.

1-11. U. S. Department of Energy, **Natural Phenomena Hazards Assessment Criteria**, DOE-STD-1023-95, Washington, DC, September 1995.

1-12. U. S. Department of Energy, **Quality Assurance Requirements**, U.S. Government Printing Office, Washington, DC, 10 CFR 830.120.

1-13. American Society of Civil Engineers, Manual and Reports on Engineering Practice No. 73, **Quality in the Constructed Project for Trial Use and Comment**, 1990.

1-14. U. S. Department of Energy, **Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites**, DOE-STD-1024-92, Department of Energy Seismic Working Group, December 1992.

1-15. U. S. Department of Energy, **Natural Phenomena Hazards Site Characterization Criteria**, DOE-STD-1022-94, Washington, DC, March 1994.

Chapter 2
Earthquake Design and Evaluation Criteria

2.1 Introduction

This chapter describes requirements for the design or evaluation of all classes of structures, systems, and components (SSCs) comprising DOE facilities for earthquake ground shaking. These classes of SSCs include safety class and safety significant SSCs per DOE-STD-3009-94 (ref. 1-6) and life-safety SSCs per Uniformed Building Codes. This material deals with how to establish Design/Evaluation Basis Earthquake (DBE) loads on various classes of SSCs; how to evaluate the response of SSCs to these loads; and how to determine whether that response is acceptable. This chapter also covers the importance of design details and quality assurance to earthquake safety. These earthquake design and evaluation provisions are equally applicable to buildings and to items contained within the building, such as equipment and distribution systems. These provisions are intended to cover all classes of SSCs for both new construction and existing facilities. These design and evaluation criteria have been developed such that the target performance goals of the NPH Implementation Guide are achieved. For more explanation see the Commentary (Appendix C) herein and the Basis Document (Ref. 2-1).

2.2 General Approach for Seismic Design and Evaluation

This section presents the approach upon which the specific seismic force and story drift provisions for seismic design and evaluation of structures, systems, and components in each Performance Category (as described in Section 2.3) is based. These provisions include the following steps:

1. Selection of earthquake loading
2. Evaluation of earthquake response
3. Specification of seismic capacity and drift limits, (acceptance criteria)
4. Ductile detailing requirements

It is important to note that the above four elements taken together comprise seismic design and evaluation criteria. Acceptable performance (i.e., achieving performance goals) can only be reached by consistent specification of all design criteria elements as shown in Figure 2-1. In order to achieve the target performance goals, these seismic design and evaluation criteria specify seismic loading in probabilistic terms. The remaining elements of the criteria (see Fig. 2-1) are deterministic design rules which are familiar to design engineers and

40538

which have a controlled level of conservatism. This level of conservatism combined with the specification of seismic loading, leads to performance goal achievement.

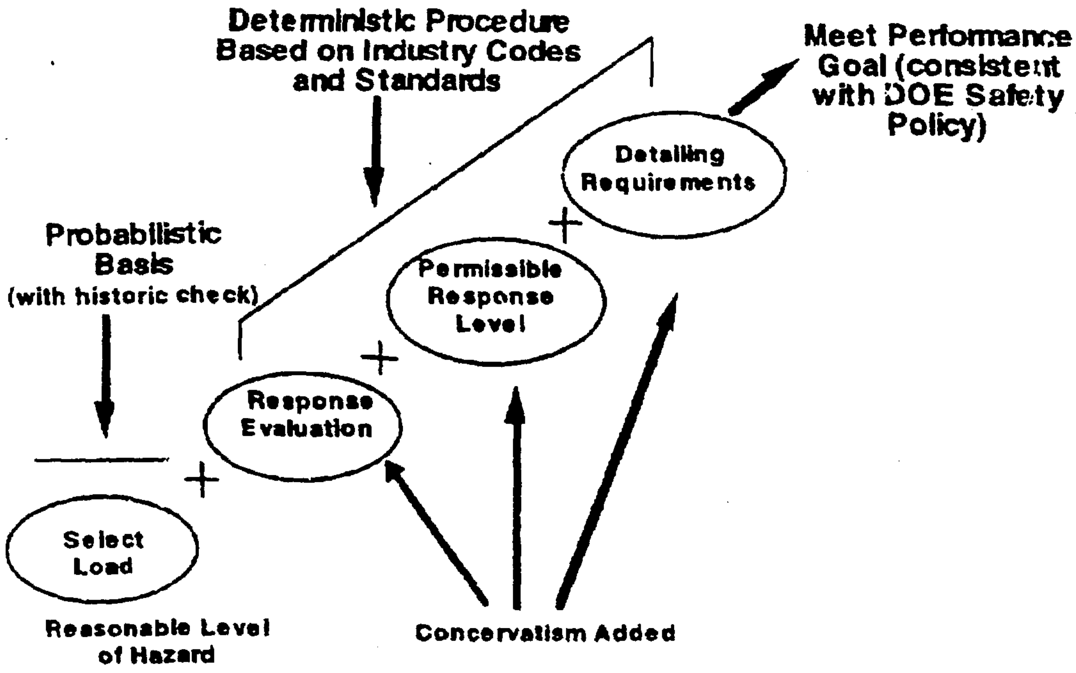


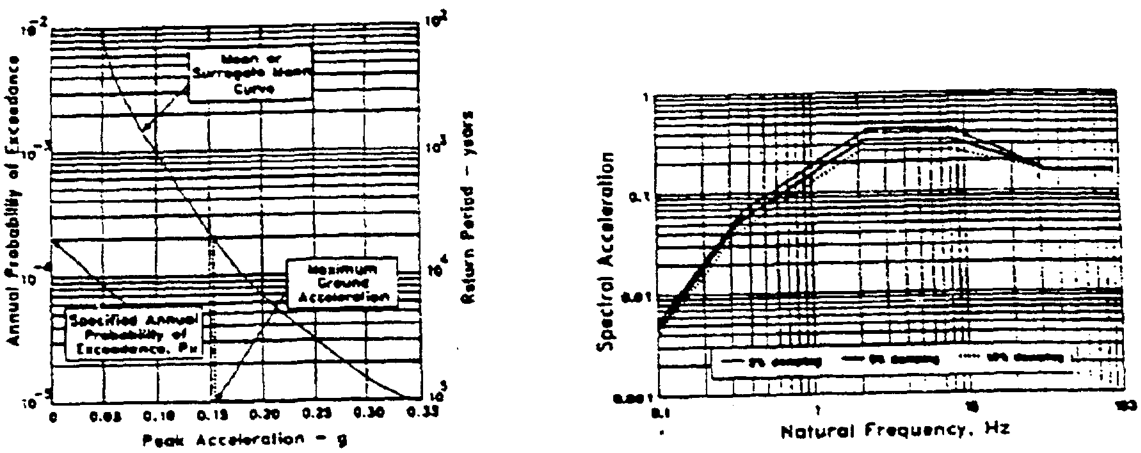
Figure 2-1. DOE-STD-1020 Combines Various Steps to Achieve Performance Goals

Criteria are provided for each of the four Performance Categories 1 to 4 as defined in the NPH Implementation Guide of DOE Order 420.1 and DOE-STD-1021 (Ref. 1-6). The criteria for Performance Categories 1 and 2 are similar to those from model building codes, with the exception that DOE requirements specify a 1000 year return period in the case of PC-2. Criteria for PC-3 are similar to those for Department of Defense Essential Facilities (Ref. C-5) Tri-Services Manual. Criteria for PC-4 approach the provisions for commercial nuclear power plants.

Seismic loading is defined in terms of a site-specified design response spectrum (the Design/Evaluation Basis Earthquake, [DBE]). Either a site-specific design response spectrum specifically developed for the site, or a generic design response spectrum that is appropriate or conservative for the site may be used. Seismic hazard estimates are used to establish the DBE per DOE-STD-1023 (REF. 2-22).

For each Performance Category, a mean annual exceedance probability for the DBE, P_H is specified from which the maximum ground acceleration (and/or velocity) may be determined from probabilistic seismic hazard curves, see Table 2-1. Evaluating maximum ground acceleration from a specified mean annual probability of exceedance is illustrated in Figure 2-2a. Earthquake input excitation to be used for design and evaluation by these provisions is defined by a median amplification smoothed and broadened design/evaluation response spectrum shape such as that shown in Figure 2-2b (from Ref. 2-2) anchored to the maximum ground acceleration and/or velocity. Such spectra are determined in accordance with DOE-STD-1023 (Ref. 2-22).

It should be understood that the spectra shown in Figure 2-2 or in-structure spectra developed from them represent inertial effects. They do not include differential support motions, typically called seismic anchor motion (SAM), of structures, equipment, or distribution systems supported at two or more points. While SAM is not usually applicable to building design, it might have a significant effect on seismic adequacy of equipment or distribution systems.



a) Evaluating Peak Acceleration from Annual Probability of Exceedance with a Seismic Hazard Curve

b) Median Amplification, smoothed and broadened, Design/Evaluation Response Spectra

Figure 2-2. Earthquake Input Excitation is Defined by Maximum Ground Acceleration Anchoring Site-Specific Response Spectra

40540

Table 2-1 Seismic Performance Categories and Seismic Hazard Exceedance Levels

Performance Category	Mean Seismic Hazard Exceedance Levels, P_H	Return Period
0	No Requirements	
1	$2x10^{-3}$	500yr
2	$1x10^{-3}$	1000yr
3	$5x10^{-4}$ $(1x10^{-3})$[1]	2000yr (1000yr)[1]
4	$1x10^{-4}$ $(2x10^{-4})$[1]	10,000yr (5000yr)[1]

[1] For sites such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC, which are near tectonic plate boundaries.

Performance Category 2 and lower SSCs may be seismically designed or evaluated using the approaches specified in building code seismic provisions. However, for Performance Category 3 or higher, the seismic evaluation must be performed by a dynamic analysis approach. A dynamic analysis approach requires that:

1. The input to the SSC model be defined by either a design response spectrum, or a compatible time history input motion.
2. The important natural frequencies of the SSC be estimated, or the peak of the design response spectrum be used as input. Multi-mode effects must be considered.
3. The resulting seismic induced inertial forces be appropriately distributed and a load path evaluation (see Section C.4.2) for structural adequacy be performed.

The words "dynamic analysis approach" are not meant to imply that complex dynamic models must be used in the evaluation. Often equivalent static analysis models are sufficient if the above listed three factors are incorporated. However, use of such simplified models for structures in Performance Category 3 or higher must be justified and approved by DOE. This dynamic analysis approach should comply with the seismic response analysis provisions of ASCE 4 (Ref. 2-3) except where specific exceptions are noted.

The maximum ground acceleration and ground response spectra determined in the manner illustrated in Figure 2-2 are used in the appropriate terms of the UBC equation for base shear. The maximum ground acceleration is also used in the UBC equation for seismic force on equipment and non-structural components. Use of modern site-specific earthquake ground motion data is considered to be preferable to the general seismic zonation maps from the UBC and should be applied according to the guidance provided in DOE-STD-1023 (Ref. 2-22). For structures, UBC provisions require a static or dynamic analysis approach in which loadings are scaled to the base shear equation value. In the base shear equation, inelastic energy absorption capacity of structures is accounted for by the parameter, R_w. Elastically computed seismic response is reduced by R_w values ranging from 4 to 12 as a means of accounting for inelastic energy absorption capability in the UBC provisions and by these criteria for Performance Category 2 and lower SSCs. This reduced seismic response is combined with non-seismic concurrent loads and then compared to code allowable response limits (or code ultimate limits combined with code specified load factors). The design detailing provisions from the UBC, which provide ductility, toughness, and redundancy, are also required such that SSCs can fully achieve potential inelastic energy absorption capability. Normally, relative seismic anchor motion (SAM) is not considered explicitly by model building code seismic provisions. However, SAM should be considered for SSCs in PC-2 or higher categories.

The Uniform Building Code (UBC) has been followed for Performance Categories 1 and 2 because it is believed that more engineers are familiar with this code than other model building codes. The Interagency Committee on Seismic Safety in Construction (ICSSC, Ref. 2-4) has concluded that the following seismic provisions are equivalent for a given DBE:

1. 1994 Uniform Building Code (Ref. 2-5)
2. 1991 NEHRP Recommended Provisions (Ref. 2-6)
3. 1993 BOCA National Building Code (Ref. 2-7)
4. 1994 SBCCI Standard Building Code (Ref. 2-8)

These other model building codes may be followed provided site-specific ground motion data is incorporated into the development of earthquake loading in a manner similar to that described in this document for the UBC.

For Performance Category 3 and 4 SSCs, these seismic design and evaluation criteria specify that seismic evaluation be accomplished by dynamic analysis. The recommended

40542

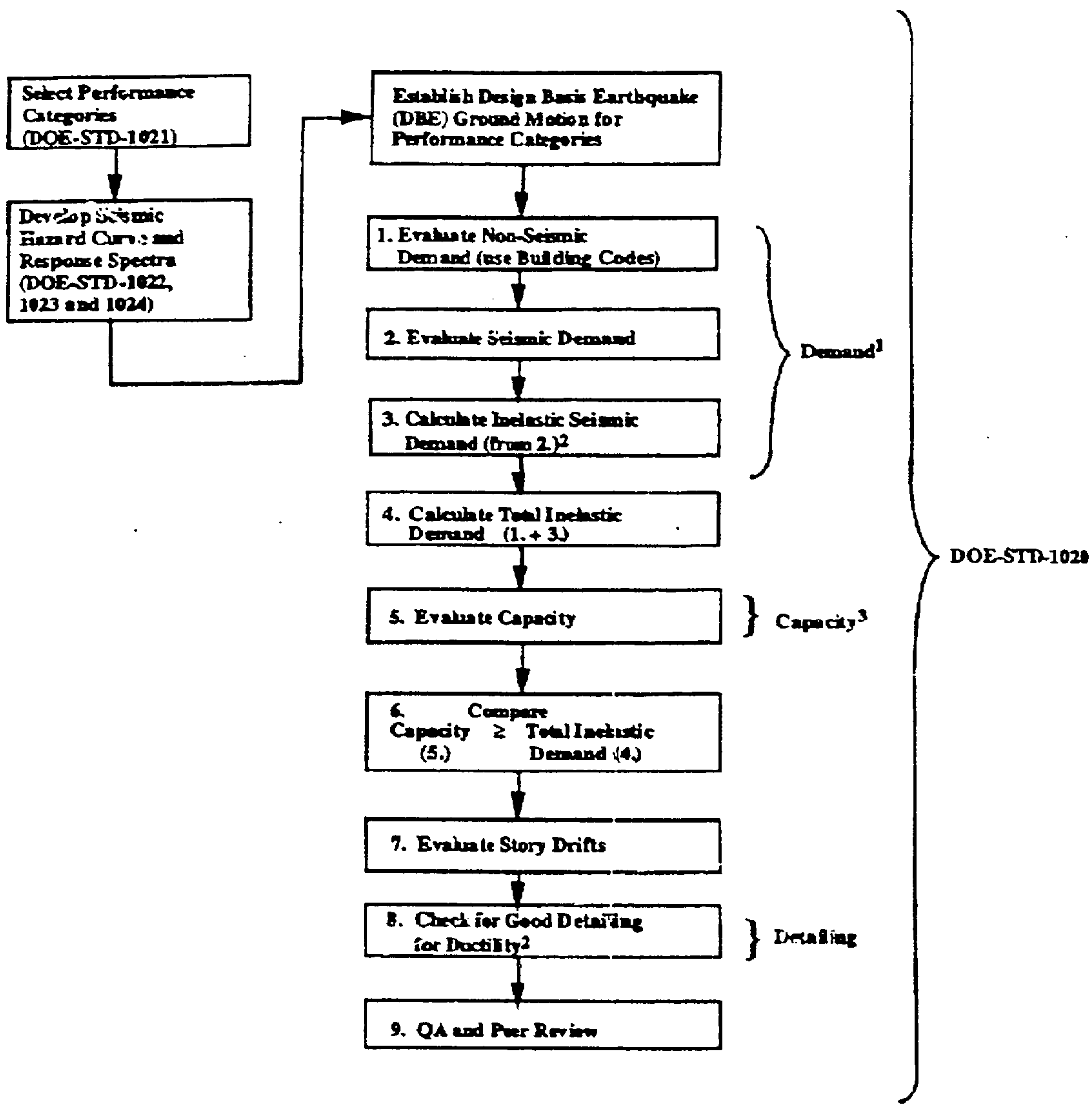
approach is to perform an elastic response spectrum dynamic analysis to evaluate elastic seismic demand on SSCs. Inelastic energy absorption capability is allowed by permitting limited inelastic behavior. By these provisions, inelastic energy absorption capacity of structures is accounted for by the parameter, F_μ. However, strength and ductile detailing for the entire load path should be assured. Elastically computed seismic response is reduced by F_μ values ranging from 1 to 3 as a means of accounting for inelastic energy absorption capability. The same F_μ values are specified for both Performance Categories of 3 and 4. In order to achieve the conservatism appropriate for the different Performance Categories, the reduced seismic forces are multiplied by a scale factor. Scale factors are specified for Performance Category 3 and 4. The resulting factored seismic forces are combined with non-seismic concurrent loads and then compared to code ultimate response limits. The design detailing provisions from the UBC, which provide ductility, toughness, and redundancy, are also required such that SSCs can fully achieve potential inelastic energy absorption capability. Also, explicit consideration of relative seismic anchor motion (SAM) effects is required for Performance Category 3 and higher.

The overall DOE Seismic Design and Evaluation Procedure is shown in Figure 2-3. In addition to the general provisions described in this chapter, the topics discussed in Appendix C should be considered before commencing design or evaluation.

2.3 Seismic Design and Evaluation of Structures, Systems, and Components

- Select Performance Categories of structure, system, or component based on DOE-STD-1021 (Ref. 1-10).
- For sites with Performance Category 3 or 4 structures, systems, and components, obtain or develop a seismic hazard curve and design response spectra in accordance with DOE-STD-1023 (Ref. 2-22) for all performance categories based on site characterization discussed in DOE-STD-1022 (Ref. 1-15). In the interim, Eastern U.S. sites may use DOE-STD-1024. (Ref. 2-23)
- Establish design basis earthquake from P_H, (see Table 2-1) mean seismic hazard curve, and median response spectra.

  For sites with only PC-1 or 2 SSC, and no site-specific seismic hazard curve, obtain seismic coefficients from model building codes.



1. See Section C.4 for further discussion.
2. For evaluation of existing facilities, the strength and detailing of the entire load path must be checked prior to assignment of ductility reduction factors.
3. See Section C.5 for further discussion.

Figure 2-3. DOE Seismic Design and Evaluation Procedure

40544

can be estimated by multiplying calculated drifts by 3 ($R_W/8$). These drifts limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural systems and non-structural elements.

- Elements of the facility shall be checked to assure that all detailing requirements of the UBC provisions are met. The basic UBC seismic detailing provisions must be met if Z is 0.11g or less. UBC Seismic Zone No. 2 provisions shall be met when Z is between 0.12 and 0.24g. UBC Seismic Zone Nos. 3 & 4 provisions shall be followed when Z is 0.25g or more.

- A quality assurance program consistent with model building code requirements shall be implemented for SSCs in Performance Categories 1 and 2. In addition, peer review shall be conducted for Performance Category 2 SSCs.

2.3.2 Performance Category 3 and 4 Structures, Systems, and Components

The steps in the procedure for PC-3 and 4 SSCs are as follows:

- Evaluate element forces, D_{NS}, for the non-seismic loads expected to be acting concurrently with an earthquake.

- Calculate the elastic seismic response to the DBE, D_S, using a dynamic analysis approach and appropriate damping values from Table 2-3. Response Level 3 is to be used only for justifying the adequacy of existing SSCs with adequate ductile detailing. Note that for evaluation of systems and components supported by the structure, in-structure response spectra are used. For PC-3 and PC-4 SSCs, the dynamic analysis must consider 3 orthogonal components of earthquake ground motion (two horizontal and one vertical). Responses from the various direction components shall be combined in accordance with ASCE 4. Include, as appropriate, the contribution from seismic anchor motion. To determine response of SSCs which use $F_\mu > 1$, note that for fundamental periods lower than the period at which the maximum spectral amplification occurs, the maximum spectral acceleration should be used. For higher modes, the actual spectral accelerations should be used.

- Calculate the inelastic seismic demand element forces, D_{SI}, as

$$D_{SI} = SF\frac{D_S}{F_\mu} \qquad (2\text{-}2)$$

where: F_μ = Inelastic energy absorption factor from Table 2-4 for the appropriate structural system and elements having adequate ductile detailing

SF = Scale factor related to Performance Category
= 1.25 for PC-4
= 1.0 for PC-3

Variable scale factors, based on the slope of site-specific hazard curves, may be used as discussed in Appendix C to result in improved achievement of performance goals. SF is applied for evaluation of structures, systems, and components. At this time, F_μ values are not provided for systems and components. It is recognized that many systems and components exhibit ductile behavior for which F_μ values greater than unity would be appropriate (see Section C.4.4.2). Low F_μ values in Table 2-4 are intentionally specified to avoid brittle failure modes.

- Evaluate the total inelastic-factored demand D_{TI} as the sum of D_{SI} and D_{NS} (the best-estimate of all non-seismic demands expected to occur concurrently with the DBE).

$$D_{TI} = D_{NS} + D_{SI} \qquad (2\text{-}3)$$

- Evaluate capacities of elements, C_C, from code ultimate or yield values
  - Reinforced Concrete
    - Use UBC Chapter 19
  - Steel
    - Use UBC Chapter 22 Standards
      - — LRFD provisions, or
      - — Plastic Design provisions, or
      - — Allowable Stress Design provision scaled by 1.4 for shear in members and bolts and 1.7 for all other stresses.

Refer to References 2-9 and 2-10 for related industry standards. Note that strength reduction factors, ϕ, are retained. Minimum specified or 95%

40550

nonexceedence in-situ values for material strengths should be used to estimate capacities.

- The seismic capacity is adequate when C_C exceeds D_{TI}, i.e.:

$$C_C \geq D_{TI} \tag{2-4}$$

- Evaluate story drifts due to lateral forces, including both translation and torsion. It may be assumed that inelastic drifts are adequately approximated by elastic analyses (note that lateral seismic forces are not reduced by F_μ when computing story drifts). Calculated story drifts should not exceed 0.010 times the story height for structures with contribution to distortion from both shear and flexture. For structures in which shear distortion is the primary contributer to drift, such as those with low rise shear walls or concentric braced-frames, the calculated story drift should not exceed 0.004 times the story height. These drift limits may be exceeded when acceptable performance of both the structure and nonstructural elements can be demonstrated at greater drift.

- Check elements to assure that good detailing practice has been followed (e.g., see sect. C.4.4.2). Values of F_μ given in Table 2-4 are upper limit values assuming good design detailing practice and consistency with recent UBC provisions. Existing facilities may not be consistent with recent provisions, and, if not, must be assigned reduced F_μ. Basic UBC seismic detailing provisions shall be followed if the PGA at P_H is 0.11g or less. UBC Seismic Zone No. 2 provisions should be met when the PGA at P_H is between 0.12 and 0.24g. UBC Seismic Zone Nos. 3 & 4 provisions should be followed when the PGA at P_H is 0.25g or more.

- Implement peer review of engineering drawings and calculations (including proper applicaton of F_μ values), increased inspection and testing of new construction or existing facilities.

2.3.3 Damping Values for Performance Category 3 and 4 Structures, Systems, and Components

Damping values to be used in linear elastic analyses are presented in Table 2-3 at three different response levels as a function of D_T/C_C.

D_T is the elastically computed total demand,

$$D_T = D_{NS} + D_S \tag{2-5}$$

and C_C is the code specified capacity.

When determining the input to subcomponents mounted on a supporting structure, the damping value to be used in elastic response analyses of the supporting structure shall be based on the response level reached in the majority of the seismic load resisting elements of the supporting structure. This may require a second analysis.

In lieu of a second analysis to determine the actual response of the structure, Response Level 1 damping values may be used for generation of in-structure spectra. Response Level 1 damping values must be used if stability considerations control the design.

When evaluating the structural adequacy of an existing SSC, Response Level 3 damping may be used in elastic response analyses independent of the state of response actually reached, because such damping is expected to be reached prior to structural failure.

When evaluating a new SSC, damping is limited to Response Level 2. For evaluating the structural adequacy of a new SSC, Response Level 2 damping may be used in elastic response analyses independent of the state of response actually reached.

The appropriate response level can be estimated from the following:

Response Level	D_T/C_C
3**	≥ 1.0
2*	≈ 0.5 to 1.0
1*	≤ 0.5

* Consideration of these damping levels is required only in the generation of floor or amplified response spectra to be used as input to subcomponents mounted on the supporting structure. For analysis of structures including soil-structure interaction effects (sec C.4.3), D_T/C_C ratios for the best estimate case shall be used to determine response level.

** Only to be used for justifying the adequacy of existing SSCs with adequate ductile detailing. However, functionality of SSCs in PC-3 and PC-4 must be given due consideration.

40552

Table 2-3 Specified Damping Values

	Damping (% of critical)		
Type of Component	Response Level 1	Response Level 2	Response Level 3
Welded and friction bolted metal structures	2	4	7
Bearing-bolted metal structures	4	7	10
Prestressed concrete structures (without complete loss of prestress)	2	5	7
Reinforced concrete structures	4	7	10
Masonry shear walls	4	7	12
Wood structures with nailed joints	5	10	15
Distribution systems***	3	5	5
Massive, low-stressed components (pumps, motors, etc.)	2	3	--*
Light welded instrument racks	2	3	--*
Electrical cabinets and other equipment	3	4	5**
Liquid containing metal tanks			
Impulsive mode	2	3	4
Sloshing mode	0.5	0.5	0.5

* Should not be stressed to Response Level 3. Use damping for Response Level 2.

** May be used for anchorage and structural failure modes which are accompanied by at least some inelastic response. Response Level 1 damping values should be used for functional failure modes such as relay chatter or relative displacement issues which may occur at a low cabinet stress level.

*** Cable trays more than one half full of loose cables may use 10% of critical damping.

Table 2-4 Inelastic Energy Absorption Factors, F_μ

Structural System (terminology is identical to Ref. 2-5)	F_μ
MOMENT RESISTING FRAME SYSTEMS - Beams	
Steel Special Moment Resisting Frame (SMRF)	3.0
Concrete SMRF	2.75
Concrete Intermediate Moment Frame (IMRF)	1.5
Steel Ordinary Moment Resting Frame	1.5
Concrete Ordinary Moment Resisting Frame	1.25
SHEAR WALLS	
Concrete or Masonry Walls	
In-plane Flexure	1.75
In-plane Shear	1.5
Out-of-plane Flexure	1.75
Out-of plane Shear	1.0
Plywood Walls	1.75
Dual System, Concrete with SMRF	2.5
Dual System, Concrete with Concrete IMRF	2.0
Dual System, Masonry with SMRF	1.5
Dual System, Masonry with Concrete IMRF	1.4
STEEL ECCENTRIC BRACED FRAMES (EBF)	
Beams and Diagonal Braces	2.75
Beams and Diagonal Braces, Dual System with Steel SMRF	3.0
CONCENTRIC BRACED FRAMES	
Steel Beams	2.0
Steel Diagonal Braces	1.75
Concrete Beams	1.75
Concrete Diagonal Braces	1.5
Wood Trusses	1.75
Beams and Diagonal Braces, Dual Systems	
Steel with Steel SMRF	2.75
Concrete with Concrete SMRF	2.0
Concrete with Concrete IMRF	1.4
METAL LIQUID STORAGE TANKS	
Moment and Shear Capacity	1.25
Hoop Capacity	1.5

Note:

1. Values herein assume good seismic detailing practice per Reference 2-5, along with reasonably uniform inelastic behavior. Otherwise, lower values should be used.
2. $F\mu$ for columns for all structural systems is 1.5 for flexure and 1.0 for axial compression and shear. For columns subjected to combined axial compression and bending, interaction formulas shall be used.
3. Connections for steel concentric braced frames should be designed for at least the lesser of:
   The tensile strength of the bracing.
   The force in the brace corresponding to $F\mu$ of unity.
   The maximum force that can be transferred to the brace by the structural system.
4. Connections for steel moment frames and eccentric braced frames and connections for concrete, masonry, and wood structural systems should follow Reference 2-5 provisions utilizing the prescribed seismic loads from these criteria and the strength of the connecting members. In general, connections should develop the strength of the connecting members or be designed for member forces corresponding to $F\mu$ of unity, whichever is less.
5. $F\mu$ for chevron, V, and K bracing is 1.5. K bracing requires special consideration for any building if Z is 0.25g or more.

40554

1. If an existing SSC is close to meeting the criteria, a slight increase in the annual risk to natural phenomena hazards can be allowed within the tolerance of meeting the target performance goals (See Section 1.3). Note that reduced criteria for seismic evaluation of existing SSCs is supported in Reference 2-16. As a result, some relief in the criteria can be allowed by performing the evaluation using hazard exceedance probability of twice the value recommended in Table 2-1 for the Performance Category of the SSC being considered.
2. The SSC may be strengthened such that its seismic resistance capacity is sufficiently increased to meet these seismic criteria. When upgrading is required it should be designed for the original Performance Goal.
3. The usage of the facility may be changed such that it falls within a less hazardous Performance Category and consequently less stringent seismic requirements.
4. It may be possible to conduct the aspects of the seismic evaluation in a more rigorous manner that removes conservatism such that the SSC may be shown to be adequate. Alternatively, a probabilistic assessment might be undertaken in order to demonstrate that the performance goals can be met.

Requirements of Executive order 12941 (Ref. 1-6), as discussed in the Implementation Guide are to be implemented.

2.4.3 Basic Intention of Dynamic Analysis Based Deterministic Seismic Evaluation and Acceptance Criteria

The basic intention of the deterministic seismic evaluation and acceptance criteria defined in Section 2.3 is to achieve less than a 10% probability of unacceptable performance for a structure, system, or component (SSC) subjected to a Scaled Design/Evaluation Basis Earthquake (SDBE) defined by:

$$SDBE = (1.5SF)(DBE) \qquad (2\text{-}7)$$

where SF is the appropriate seismic scale factor from Equation 2-2.

The seismic evaluation and acceptance criteria presented in this section has intentional and controlled conservatism such that the target performance goals are achieved. The amount of intentional conservatism has been evaluated in Reference 2-1 such that there should be less than 10% probability of unacceptable performance at input ground motion defined by a scale

40560

factor of 1.5SF times the DBE. Equation 2-7 is useful for developing alternative evaluation and acceptance criteria which are also based on the target performance goals such as inelastic seismic response analyses. To evaluate items for which specific acceptance criteria are not yet developed, such as overturning or sliding of foundations, or some systems and components; this basic intention must be met. If a nonlinear inelastic response analysis which explicitly incorporates the hysteretic energy dissipation is performed, damping values that are no higher than Response Level 2 should be used to avoid the double counting of this hysteretic energy dissipation which would result from the use of Response Level 3 damping values.

2.5 Summary of Seismic Provisions

Table 2-5 summarizes recommended earthquake design and evaluation provisions for Performance Categories 1 through 4. Specific provisions are described in detail in Section 2.3. The basis for these provisions is described in Reference 2-1.

Table 2-5 Summary of Earthquake Evaluation Provisions

	Performance Category (PC)			
	1	2	3	4
Hazard Exceedance Probability, P_H	2×10^{-3}	1×10^{-3}	5×10^{-4} (1×10^{-3})[1]	1×10^{-4} (2×10^{-4})[1]
Response Spectra	Median amplification (no conservative bias)			
Damping for Structural Evaluation	5%		Table 2-3	
Acceptable Analysis Approaches for Structures	Static or dynamic force method normalized to code level base shear		Dynamic analysis	
Analysis approaches for systems and components	UBC Force equation for equipment and non-structural elements (or more rigorous approach)		Dynamic analysis using in-structure response spectra (Damping from Table 2-3)	
Importance Factor	I=1.0	I=1.25	Not used	
Load Factors	Code specified load factors appropriate for structural material		Load factors of unity	
Scale Factors	Not Used		SF = 1.0	SF = 1.25
Inelastic Energy Absorption Ratios	Accounted for by R_w from Table 2-2		F_μ from Table 2-4 by which elastic response is reduced to account for permissible inelastic behavior	
Material Strength	Minimum specified or 95% non-exceedance in-situ values			
Structural Capacity	Code ultimate strength or allowable behavior level		Code ultimate strength or limit-state level	
Quality Assurance Program	Required within a graded approach (i.e., with increasing rigor ranging from UBC requirements from PC-1 to nuclear power plant requirements for PC-4)			
Peer Review	Not Required	Required within a graded approach (i.e., with increasing rigor ranging from UBC requirements from PC-2 to nuclear power plant requirements for PC-4)		

[1]For sites such as LLNL, SNL-Livermore, SLAC, LBL, & ETEC which are near tectonic plate boundaries

40561

2.6 References

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40562

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