In the Matter of: PRIVATE FUEL STORAGE, LLC
(Independent Spent Fuel Storage Installation)
Docket No. 72-22-ISFSI; ASLBP No. 97-732-02-ISFSI

PFS Testimony Exhibit List for Unified Contention Utah L/QQ

Tab	Description	Date
	Youngs/Tseng Testimony Exhibits	
LL	Geomatrix Evaluation of Spatial and Temporal Variation of Ground Motion for the Private Fuel Storage Facility ("Geomatrix Evaluation")	03/11/02
MM	SWEC Calculation SC-21 (Evaluation of Cask Storage Pad Flexibility)	03/31/02
	Singh/Soler Testimony Exhibits	
NN	PFS Commitment Resolution Letter #37	08/07/01
OO	Visual NASTRAN Simulations (Filed Separately)	
PP	Excerpts from Deposition Transcript of Moshin R. Khan	03/05/02
QQ	Static and Sliding Friction in Feedback Systems	Jan. 1953
RR	Excerpts from DYNAMO Validation Manual	
SS	Excerpts from ANSYS Training and Validation Manual	
TT	Holtec Coefficient of Restitution and Linear Viscous Damping	
	Trudeau Testimony Exhibits	
UU	SWEC Calculation G(B)-04, Rev. 9 (Stability Analyses of Cask Storage Pads)	07/26/01
VV	SWEC Calculation G(B)-13, Rev. 6 (Stability Analyses of Canister Transfer Building)	07/26/01

Tab	Description	Date
WW	Sliding Stability Chart	
	Ebbeson Testimony Exhibits	
XX	ASCE-4-86 (Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety-Related Nuclear Structures)	Sept. 1986
YY	SWEC Calculation SC-6 (Finite Element Analysis of Canister Transfer Building)	12/04/98
	Lewis Testimony Exhibits	
ZZ	SAR Table 5.1-1	
AAA	HI-STORM FSAR Table 10.3.3a	
BBB	SAR Table 3.4-1	
CCC	SAR, p. 3.4-2	
	Cornell Testimony Exhibits	
DDD	DOE-STD-1020-94 (Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities)	
EEE	Excerpts from Deposition Transcript of Walter J. Arabasz	
FFF	DOE Topical Report TR-003 (Preclosure Seismic Design Methodology for a Geologic Repository at Yucca Mountain)	Aug. 1997
	Trudeau/Wissa Testimony Exhibits	
GGG	Engineering Services Scope of Work for Laboratory Testing of Soil-Cement Mixes	1998

Tab	Description	Date
HHH	American Concrete Institute Report ACI 230.1R-90, "State-of-the-Art-Report on Soil Cement"	01/31/01
III	Excerpts from Deposition Transcript of James K. Mitchell	1997
JJJ	SAR § 2.6.4.11	03/15/02

Document #: 1228591 v.1

Geomatrix Evaluation of Spatial and Temporal Variation of Ground Motions for the Private Fuel Storage Facility, Skull Valley, Utah

A. Estimate of the Angle of Incidence and Its Effect on Storage Pad Response
It is standard calculation in seismology to obtain the ray path for a seismic body wave traveling from a point source at depth to a site on the surface. The primary ray path is one that minimizes the travel time from the point source to the site. The minimum travel time ray path also obeys Snell's law in that the ratio of the sine of the incidence angle, i_i, at a layer boundary to the velocity within the layer, V_i, is constant all along the ray path:

$$\frac{\sin(i_i)}{V_i} = \text{constant}$$

We have applied this approach to computing the angle of incidence at the surface for waves originating on the primary sources of hazard to the PFS site, the Stansbury and East faults. Figure 1 shows the relationship of the PFS site to the two faults (using the central estimate of fault dip of 55°). The strain-compatible site velocity profile is shown on Figure 4 of Geomatrix Calculation 05996.02-G(PO18)-2 (Rev. 1) and is listed in Table 1 with layer thickness and velocities converted to meters and meters/second, respectively. We have calculated the ray paths for three points on the fault plane that span the expected depth range for release of most of the seismic energy (depths of 5 to 15 km). Table 2 lists the computed angles of incidence for this velocity model and the six point sources shown on Figure 1.

Table 1
Skull Valley Mean Strain-Compatible Velocity Profile
From Figure 4 of Calculation 05996.02-G(PO18)-2, Rev 1

Layer Thickness (m)	Total Thickness (m)	Layer Shear Wave Velocity (m/s)
1.524	1.524	456.3
1.524	3.048	126.4
0.610	3.658	189.7
1.829	5.486	237.4
2.438	7.925	231.7
2.743	10.668	249.4
4.572	15.240	291.3
12.192	27.432	523.1
64.008	91.44	883.9
60.960	152.40	1051.6
60.960	213.36	1204.0
1,186.64	1400	1,950
18,600	20000	3,400

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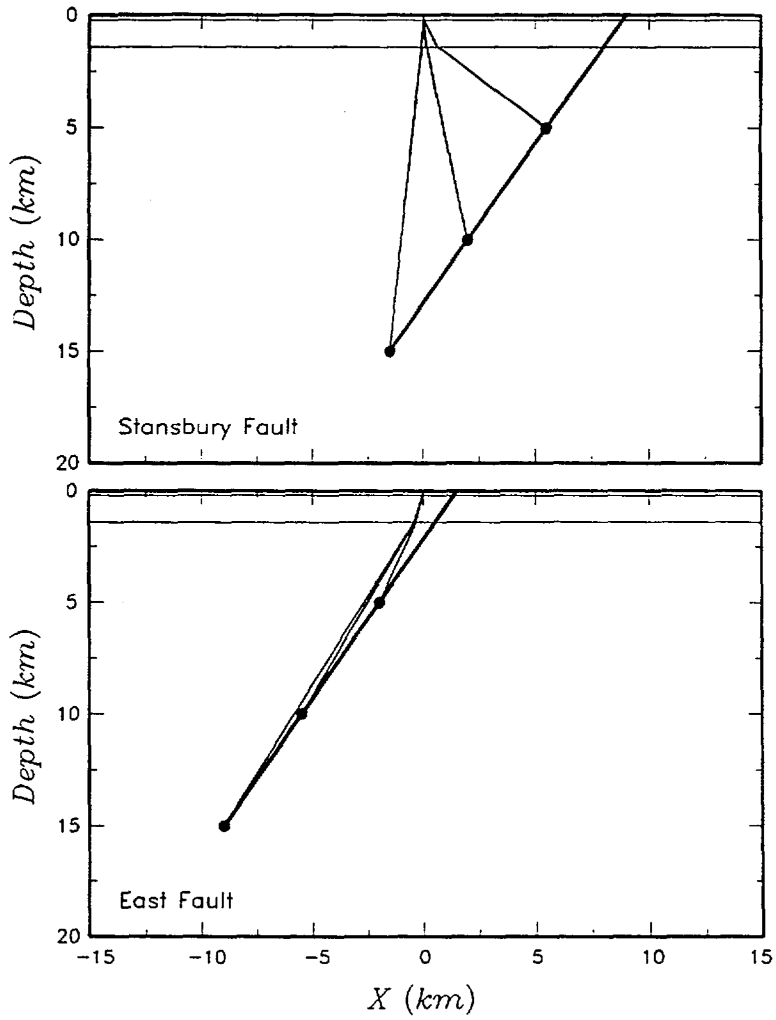

Figure 1 Example ray paths for seismic waves from fault ruptures in Skull Valley

Table 2
Surface Layer Angles of Incidence for Skull Valley Velocity Model (Table 1)

Fault	Source Point Location		Surface Layer Angle of Incidence (°)
	Depth (km)	Surface Distance (km)	
Stansbury fault	5	5.5	6.1
	10	2	1.6
	15	1.5	0.8
East fault	5	-2	3.1
	10	-5.5	3.9
	15	-9	4.1

The ray tracing analysis described above is for an infinite frequency wave, one that will be able to react to all of the layer velocity changes. The velocity model listed in Table 1

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contains thin layers near the surface, which will most influence the response of the site to high frequency waves. The site response to lower frequency waves can be modeled by replacing the detailed velocity model (Table 1) with thicker layers of uniform velocity. For example, the fundamental frequency of the 700-foot thick sedimentary sequence in Skull Valley is approximately 1 Hz. Thus, the response of the site to 1-Hz waves can be approximated using a single 700-foot thick layer with a uniform velocity equal to the average (harmonic mean) of the velocities of all of the layers to a depth of 700 feet (213.4 meters). If one uses this simpler velocity representation to assess the direction of wave propagation, the thicker single layer with uniform velocity will result in larger angles of incidence at the surface than those listed in Table 2.

Analysis of the time-histories of response of the casks (see Section C) indicates that the frequencies of interest are in the range of 1 to 5 Hz. To assess potential ray paths for waves in this frequency range, simplified velocity models were constructed by combining layers of the detailed model listed in Table 1 to produce layers with fundamental frequencies near the target frequency. Table 3 lists these simplified velocity models representative of layers with predominant frequencies of 5, 2.5, and 1 Hz. Table 4 lists the computed angles of incidence for these simplified models.

Table 3
Simplified Skull Valley Velocity Model for 5 Hz Waves

Layer Thickness (m)	Total Thickness (m)	Layer Shear Wave Velocity (m/s)
10.668	10.668	223.0
16.764	27.432	429.8
12.192	27.432	523.1
64.008	91.44	883.9
60.960	152.40	1051.6
60.960	213.36	1204.0
1,186.64	1400	1,950
18,600	20000	3,400

Simplified Skull Valley Velocity Model for 1 Hz Waves

Layer Thickness (m)	Total Thickness (m)	Layer Shear Wave Velocity (m/s)
213.35	213.35	832.1
1,186.64	1400	1,950
18,600	20000	3,400

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Table 4
Surface Layer Angles of Incidence for
Simplified Skull Valley Velocity Models Listed in Table 3

Fault	Source Point Location		Surface Layer Angle of Incidence (°) for Simplified Velocity Model for Frequency:	
	Depth (km)	Surface Distance (km)	1 Hz	5 Hz
Stansbury fault	5	5.5	11.3	3.0
	10	2	2.9	0.8
	15	1.5	1.9	0.4
East fault	5	-2	5.9	1.6
	10	-5.5	7.2	1.9
	15	-9	7.5	2.0

The velocity models listed in Table 3 assume horizontal layer boundaries. Seismic line 2 obtained by Geosphere Midwest [1997, Figure 4.6, reprinted on p. 2 of Attachment A of Geomatrix Calculation 05996.02-G(PO18)-2 (Rev. 1)] shows that the bedrock surface beneath the site dips gently down to the east with a 200-ft drop over the ~3,000-ft length of the profile. Table 5 lists the angles of incidence computing using a Tertiary-bedrock boundary with a 4° dip down to the east. The sloping bedrock changes the incidence angles by a few degrees at most.

Table 5
Surface Layer Angles of Incidence for
Simplified Skull Valley Velocity Models Listed in Table 3 Including 4° Bedrock Slope

Fault	Source Point Location		Surface Layer Angle of Incidence (°) for Simplified Velocity Model for Frequency:	
	Depth (km)	Surface Distance (km)	1 Hz	5 Hz
Stansbury fault	5	5.5	8.9	2.7
	10	2	0.6	0.5
	15	1.5	0.8	0.1
East fault	5	-2	8.2	1.9
	10	-5.5	9.5	2.2
	15	-9	9.9	2.3

The above results indicate that the expected angles of incidence at the surface for seismic waves originating from large-magnitude earthquake ruptures on the adjacent faults is small, typically less than 10° from vertical. Thus, the proximity of the site to the major active faults does not result in high angle of incidence waves measured from vertical (i.e., low angle measured from horizontal) and the assumption of vertically propagating waves is reasonable for the site.

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Inclined waves will result in a difference in arrival time for waves at two adjacent points. The time difference for two points separated by distance w measured in the direction toward the source is approximately equal to $w \sin(i_1)/V_1$, where the subscript 1 refers to the surface layer. The storage pads have a width of 30 feet in the east-west (fault-normal) direction. Using this value for w, the incidence angles listed in Table 4 together with the velocities listed in Table 3, one obtains time differences on the order of 0.001 to 0.002 seconds. These time differences are much smaller than the minimum time step of the time histories developed for the site (0.005 seconds) and would affect only very high frequency motions above the highest ground motion frequency of interest (50 Hz). Thus, the very small time difference for wave arrivals would have negligible effect on the analysis.

The effects of the low incident angles on the input motion (measured from vertical) to the pads as compared to vertically propagating waves also can be examined based on the work by Luco (1976) and Wong and Luco (1978). The controlling parameters are the normalized frequency, $a_0 = \omega a/\beta$, (where ω is the angular frequency in radians/sec, a is the equivalent radius of the pad, and β is the shear-wave velocity of the near-surface layer); and the ratio of shear-wave velocity to apparent wave velocity, β/c (equivalent to the sine of the angle of incidence). The equivalent pad radius = $\sqrt{30 \times 67 / \pi}$ = 25.3 ft = 7.7 m. For a 5-Hz wave, the largest incident angle of 3° results in $\beta/c = \sin(3°) = 0.05$. From Table 3, $\beta = 223$ m/s, resulting in $a_0 = 1.1$. For this case, Luco (1976, Fig. 3) indicates that an SH-wave would induce a torsional motion equivalent to an additional horizontal motion of about 3 percent of the amplitude of the free-field motion at the edge of the pad. For a 1-Hz wave, the largest incident angle of 11.3° results in $\beta/c = \sin(11.3°) = 0.2$. From Table 3, $\beta = 832$ m/s, resulting in $a_0 = 0.06$. For this case Luco (1976, Fig. 3) indicates less than 1 percent additional motion. Accompanying this is a slight reduction in the translation motion.

Wong and Luco (1978, Fig. 4, 5, 6, and 7) show the effect of inclined P/SV waves as a function of a_0 and β/c. The parameter a is now defined as the half-width of a square mat = $\sqrt{30 \times 67} / 2$ = 22.4 ft = 6.8 m. For 5-Hz waves, $a_0 = 1.0$ and β/c is very small (0.05). For this case, Figures 4 through 7 of Wong and Luco (1978) indicate that the response is generally within 5 percent of that for vertical waves. For 1-Hz motions, a_0 is very small (0.06), and the response is again within 5 percent of that for vertical waves.

Thus, it can be concluded that additional rocking and torsional motion of the pad caused by inclined incident waves at the small angles listed in Tables 2 and 4 is insignificant compared to the motion caused by the vertically propagating waves.

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B. Estimate of the Degree of Spatial Incoherence for Storage Pads

The degree of spatial variation in ground motions can be measured by the spatial coherency of ground motion. Abrahamson et al. (1991) developed an empirical model for spatial coherency using data from the Lotung LSST strong motion array in Taiwan. They define a model for the "lagged" coherency, $\gamma(f,\xi)$, which models the effects of scattering on ground motions; and a model for the "unlagged" coherency, $\gamma_U(f,\xi,\xi_r)$, which includes wave passage effects due to inclined waves. Their relationship for unlagged coherency is given by the expression:

$$\gamma_U(f,\xi,\xi_r) = \gamma(f,\xi)\cdot\phi(f,\xi_r)$$

in which

$$\gamma(f,\xi) = \tanh[(2.535 - 0.0118\xi)\left\{\exp(f(-0.155 - 0.000837\xi)) + \tfrac{1}{3}f^{-0.878}\right\} + 0.35]$$

and

$$\phi(f,\xi_r) = \frac{\cos(2\pi\cdot 0.00037 f\xi_r)}{1+(f/19)^4}$$

where f is frequency, ξ is the separation distance, and ξ_r is the separation distance measured along the path toward the source (i.e. fault-normal). The term $\phi(f,\xi_r)$ represents incoherency due to the wave-passage effect. In the east-west direction $\xi = \xi_r = 9.14$ meters. For this direction one obtains the following values for the frequency range of interest:

Table 6
Empirical Coherency for East-West Direction

Frequency (Hz)	$\gamma(f,\xi)$	$\phi(f,\xi_r)$	$\gamma_U(f,\xi,\xi_r)$
1	1.00	1.00	1.00
2.5	0.99	1.00	0.98
5	0.95	0.99	0.94

In the north-south direction, $\xi = 20.4$ meters and $\xi_r = 0$ (direction parallel to fault). For this direction one obtains the following values:

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Table 7
Empirical Coherency for North-South Direction

Frequency (Hz)	$\gamma(f,\xi)$	$\phi(f,\xi_r)$	$\gamma_U(f,\xi,\xi_r)$
1	1.00	1.00	1.00
2.5	0.98	1.00	0.98
5	0.94	1.00	0.93

As indicated by Abrahamson et al. (1991), the above values represent the fraction of the power in the ground motions that can be represented by a vertically propagating plane wave. The values in the table indicate that for the small pad size of interest, nearly all of the power in the ground motions can be represented by a vertically propagating plane wave.

In addition, Abrahamson et al. (1991) show in their Figure 10 the residuals between observed data and their empirical model for two epicentral distance ranges, ≤ 15 km and ≥40 km. The mean residuals for the two distance ranges oscillate about each other and the authors conclude that the data do not indicate a clear dependence of spatial coherency on distance from the source. On this basis we conclude that proximity to the major active faults does not require special evaluation of the effects of spatial variation.

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C. Evaluation of Frequency Range of Importance to Cask Response

The frequency range of importance to cask response was determined by analysis of response time histories obtained at the top of the cask for the worst cases (i.e., a 2-cask system with coefficients of friction of 0.8 and 0.2). This evaluation is based examination of the Fourier amplitude of the output motions computed at the top of the cask and the ratio of the Fourier amplitude between the output motion and the input motion for the two cases. Time histories of the cask response presented in Appendix A show that the tipping and sliding of the cask occurs primarily within the time window of 4 to 7 seconds. Fourier spectra of the velocity time histories of the time window from 4 to 7 seconds for both output and input motions and the spectral ratio of the Fourier amplitude of the output motion divided by those for the input motion are shown on Figures 2 to 5. Both the Fourier spectra of the output motions and the spectral ratios shown on Figures 2 to 5 indicate that the frequency range of peak cask response is between 1 and 5 Hz.

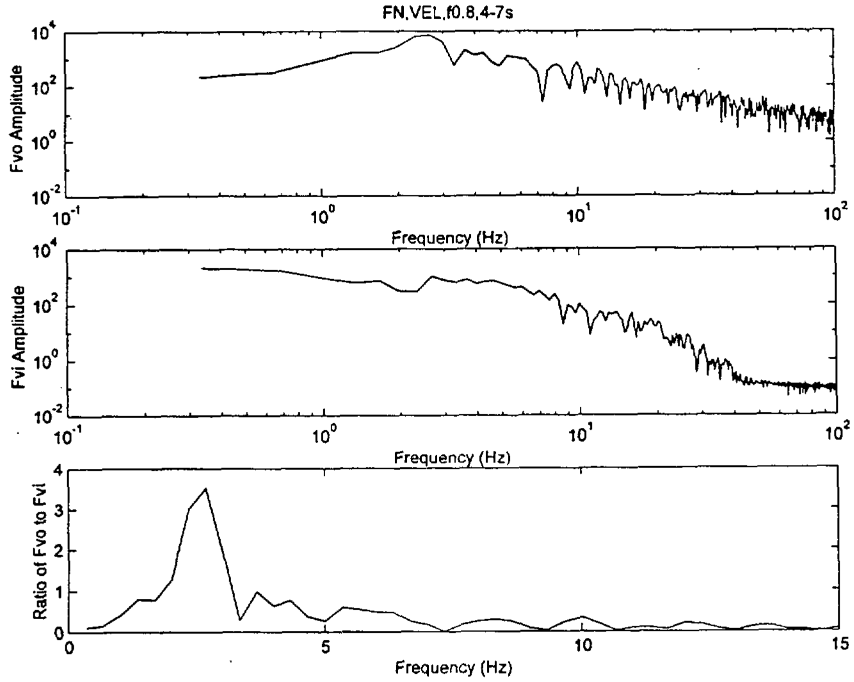

Figure 2: Fourier amplitudes of Cask 2 output motion (top plot), input motion (middle plot), and ratio of Fourier amplitudes (output/input) for motion in X-direction (fault-normal), coefficient of friction = 0.8 and time window 4 to 7 seconds.

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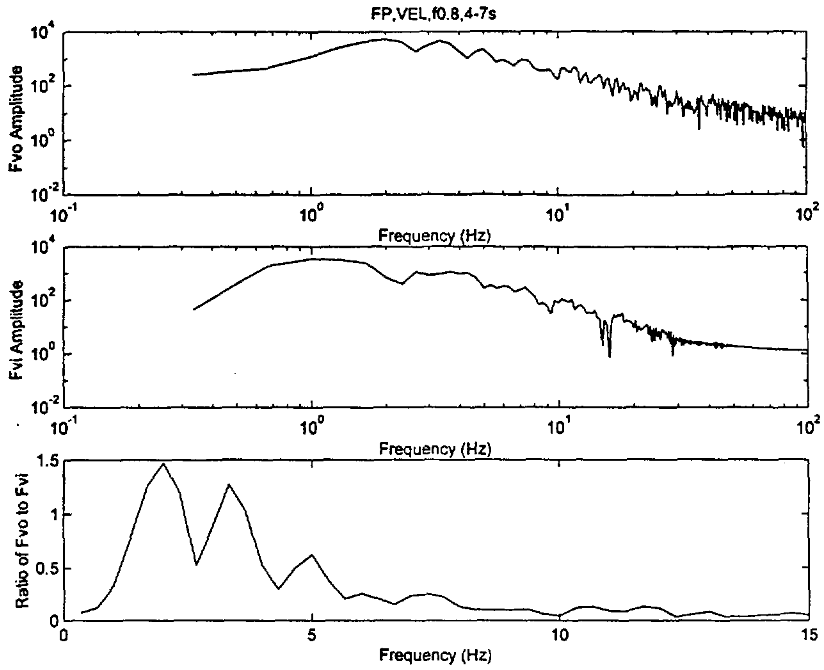

Figure 3: Fourier amplitudes of Cask 2 output motion (top plot), input motion (middle plot), and ratio of Fourier amplitudes (output/input) for motion in Y-direction (fault-parallel), coefficient of friction = 0.8 and time window 4 to 7 seconds.

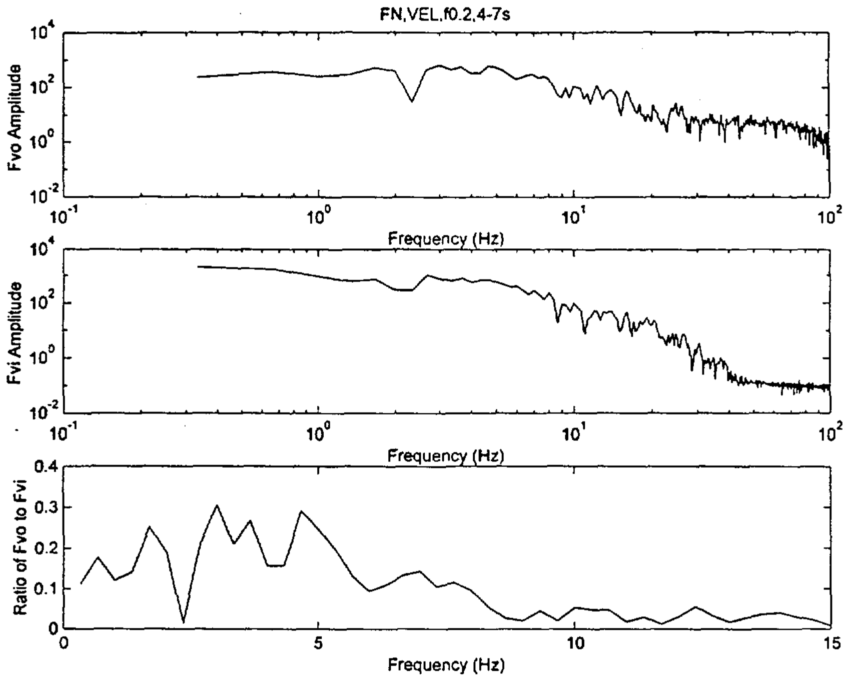

Figure 4: Fourier amplitudes of Cask 2 output motion (top plot), input motion (middle plot), and ratio of Fourier amplitudes (output/input) for motion in X-direction (fault-normal), coefficient of friction = 0.2 and time window 4 to 7 seconds.

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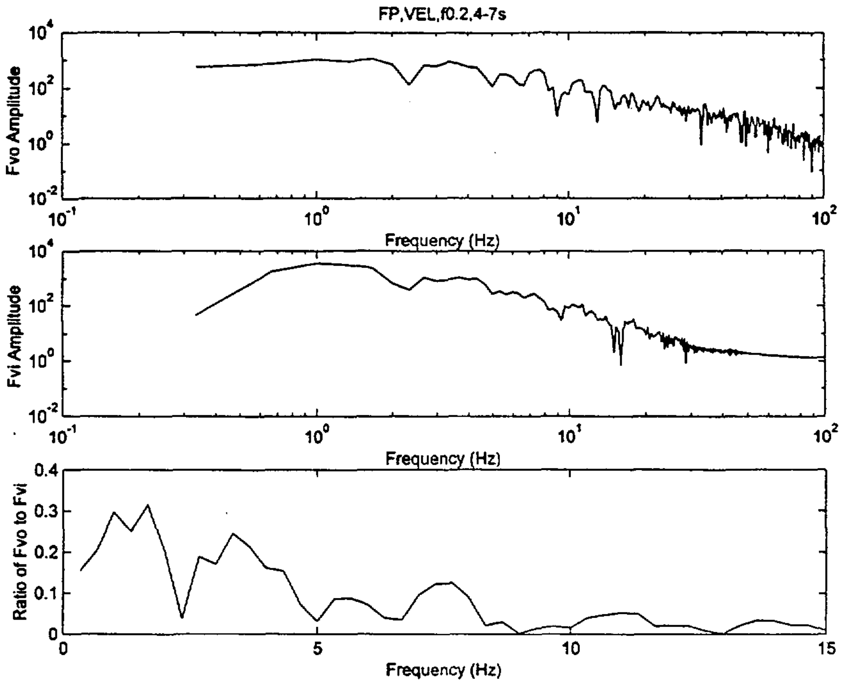

Figure 5: Fourier amplitudes of Cask 2 output motion (top plot), input motion (middle plot), and ratio of Fourier amplitudes (output/input) for motion in Y-direction (fault-parallel), coefficient of friction = 0.2 and time window 4 to 7 seconds.

References

Abrahamson, N.A., J.F. Schneider, and J.C. Stepp, 1991, Empirical spatial coherency functions for application to soil-structure interaction analyses: *Earthquake Spectra*, v. 7, p. 1-27.

Luco, J.E., 1976, Torsional Response of Structures to Obliquely Incident Seismic SH Waves: *Earthquake Engineering and Structural Dynamics*, v. 4, p. 207-219.

Wong, H.L. and Luco, J.E., 1978, Dynamic Response of Rectangular Foundations to Obliquely Incident Seismic Waves: *Earthquake Engineering and Structural Dynamics*, v. 6, p. 3-16.

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Appendix A

Time Histories of Cask Response of a Two Cask System For Coefficients of Friction of 0.8 and 0.2

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Fig. 1-Cask 1 Disp -X vs. Time cof=0.8 at point 2, file:ck1disp8

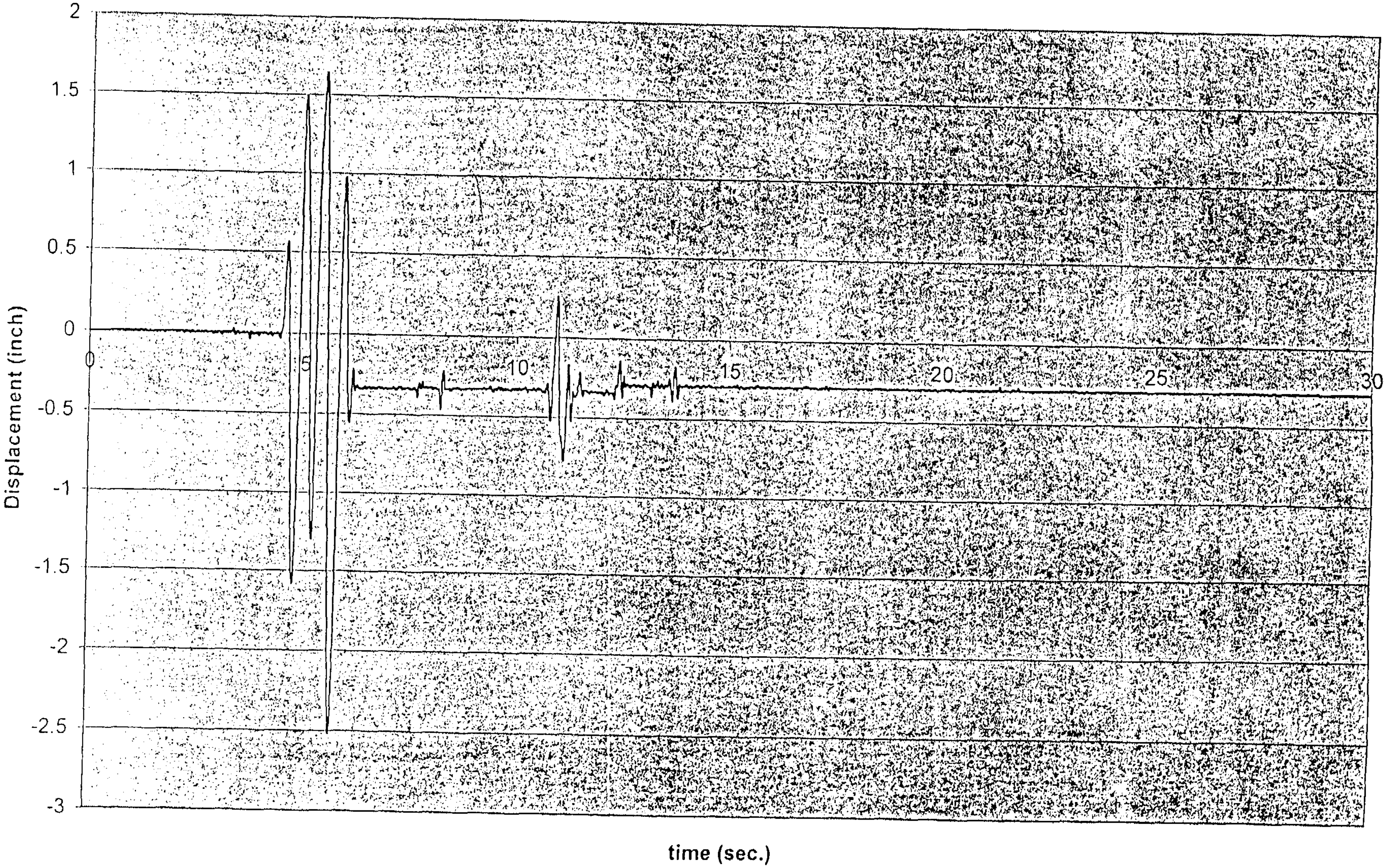

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Fig. 2 Cask 1 Disp Y vs Time, cof=0.8 for point 2, file:ck1disp8

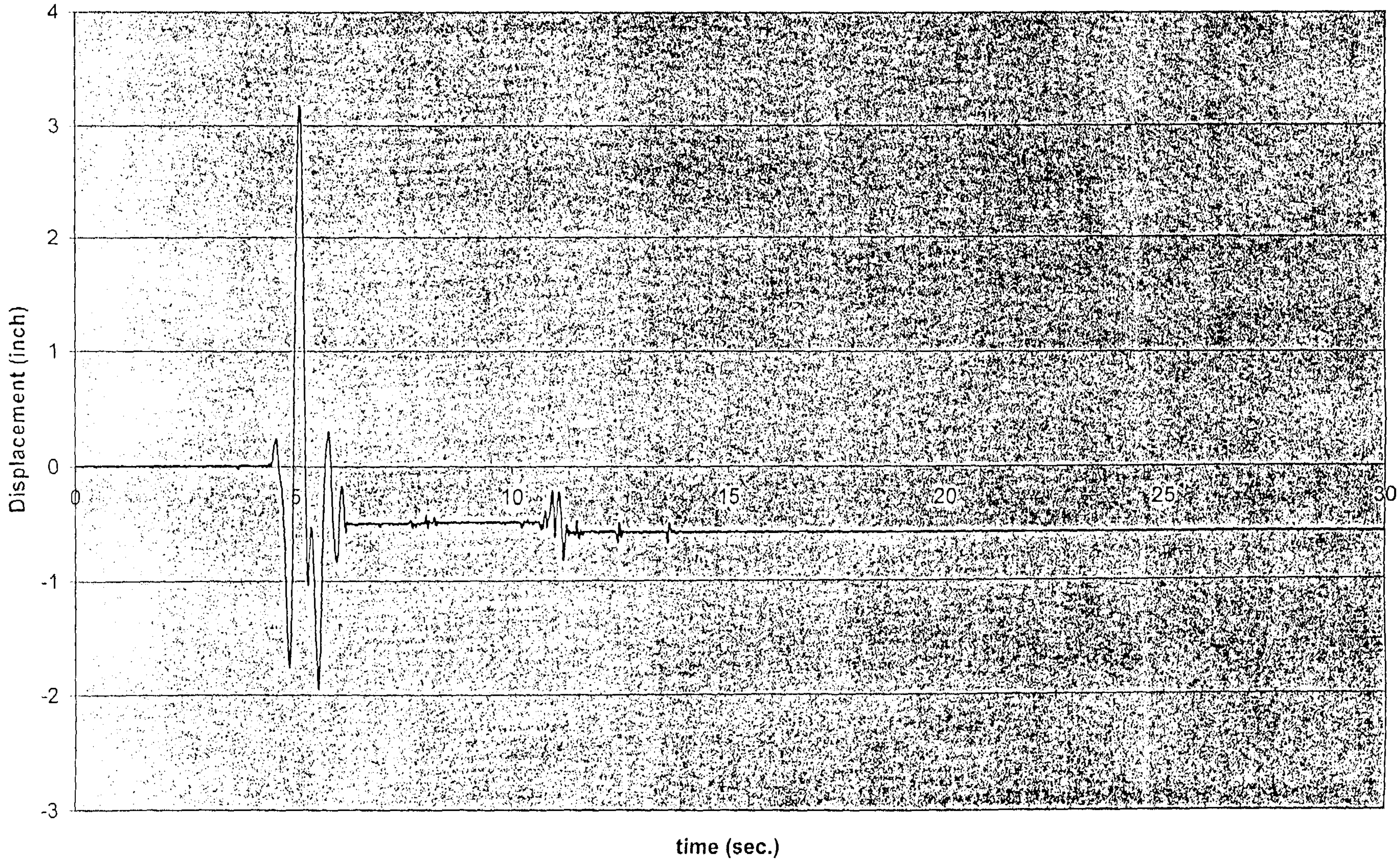

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Fig. 3 Cask 1 Velocity x vs. time cof=0.8, file:pfs108

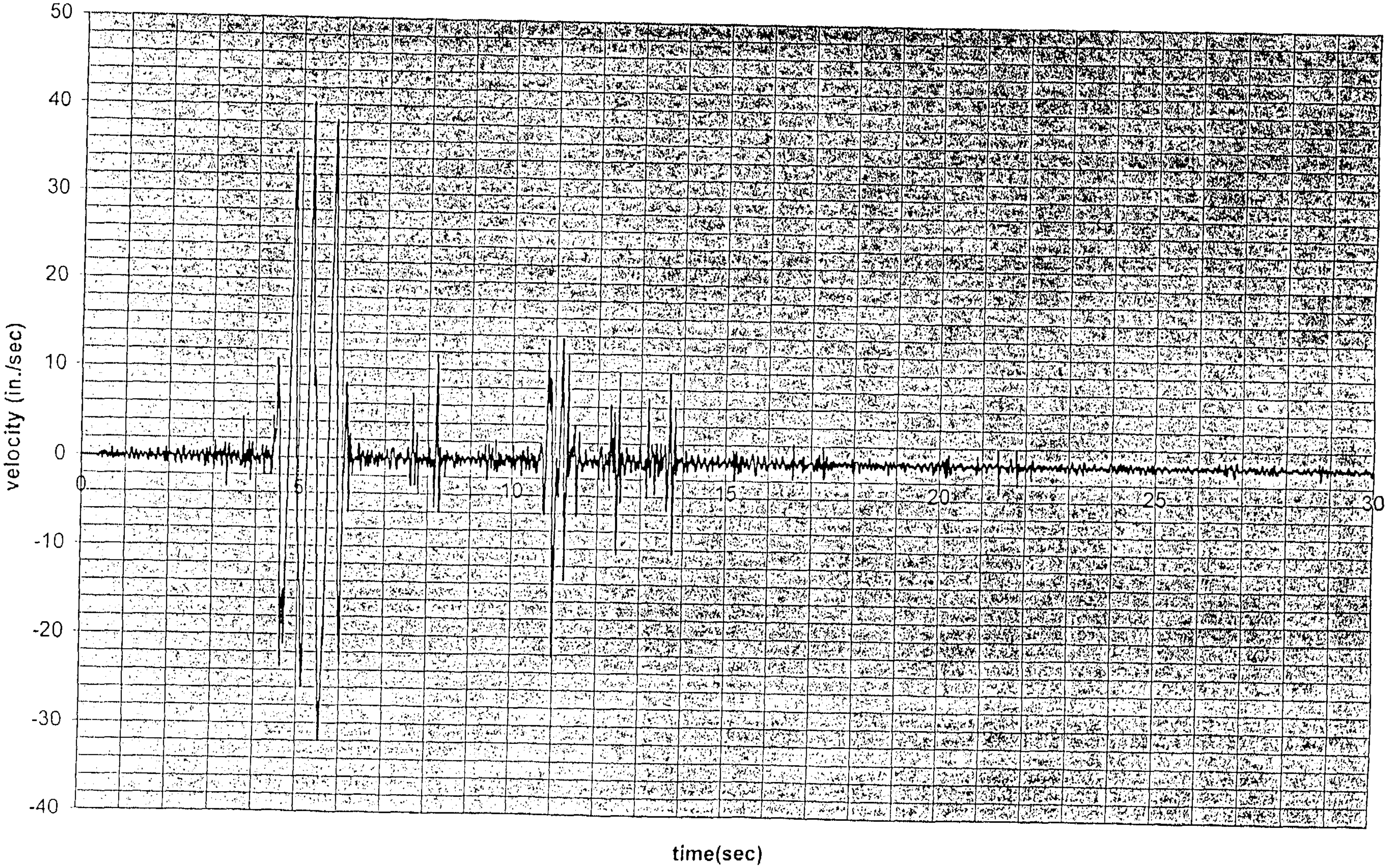

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Fig. 4 Cask 1 Velocity Y vs Time cof=0.8, file:pfs108

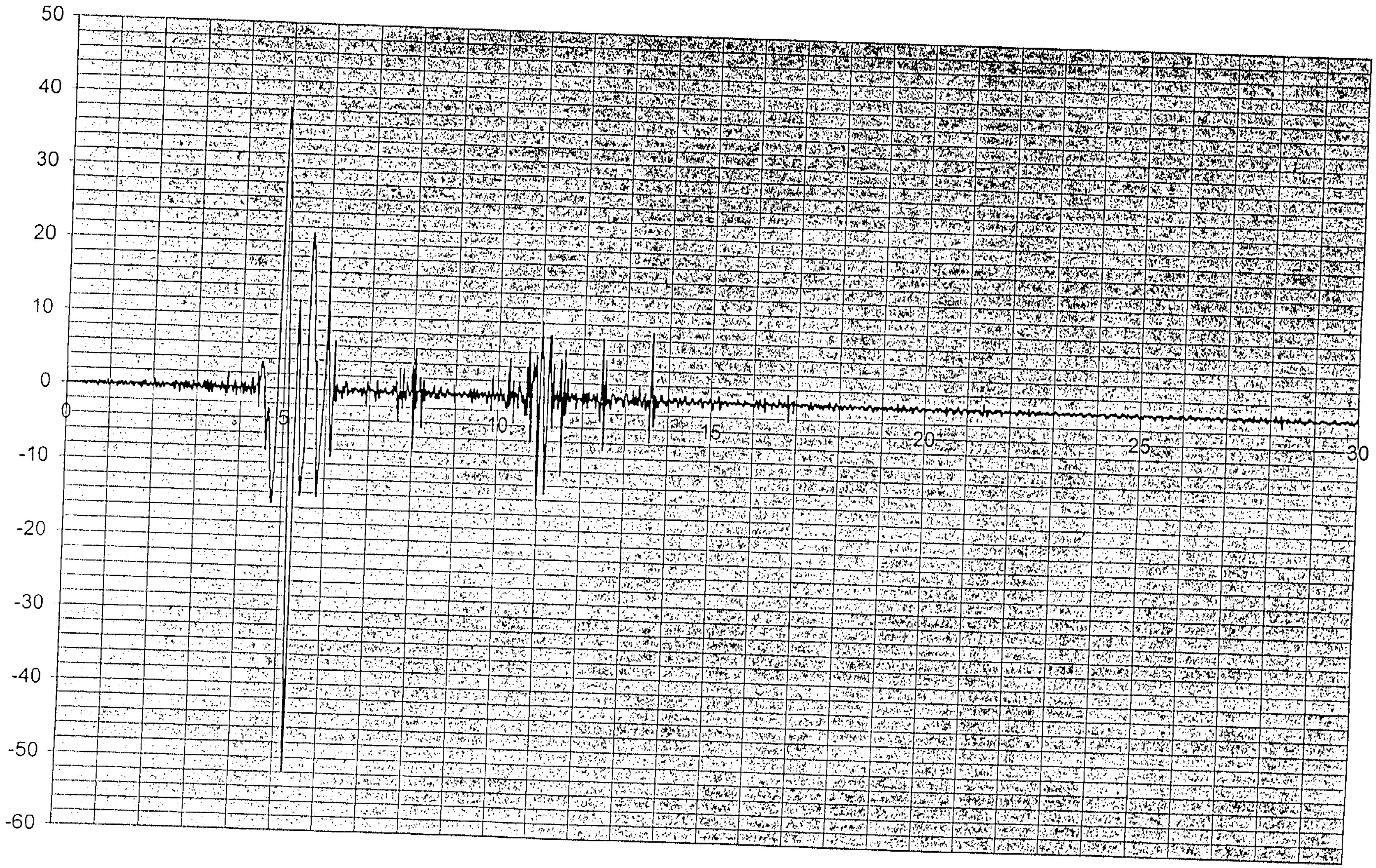

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Fig. 5 Cask 1 X accel vs time cof=0.8, file:pfs108

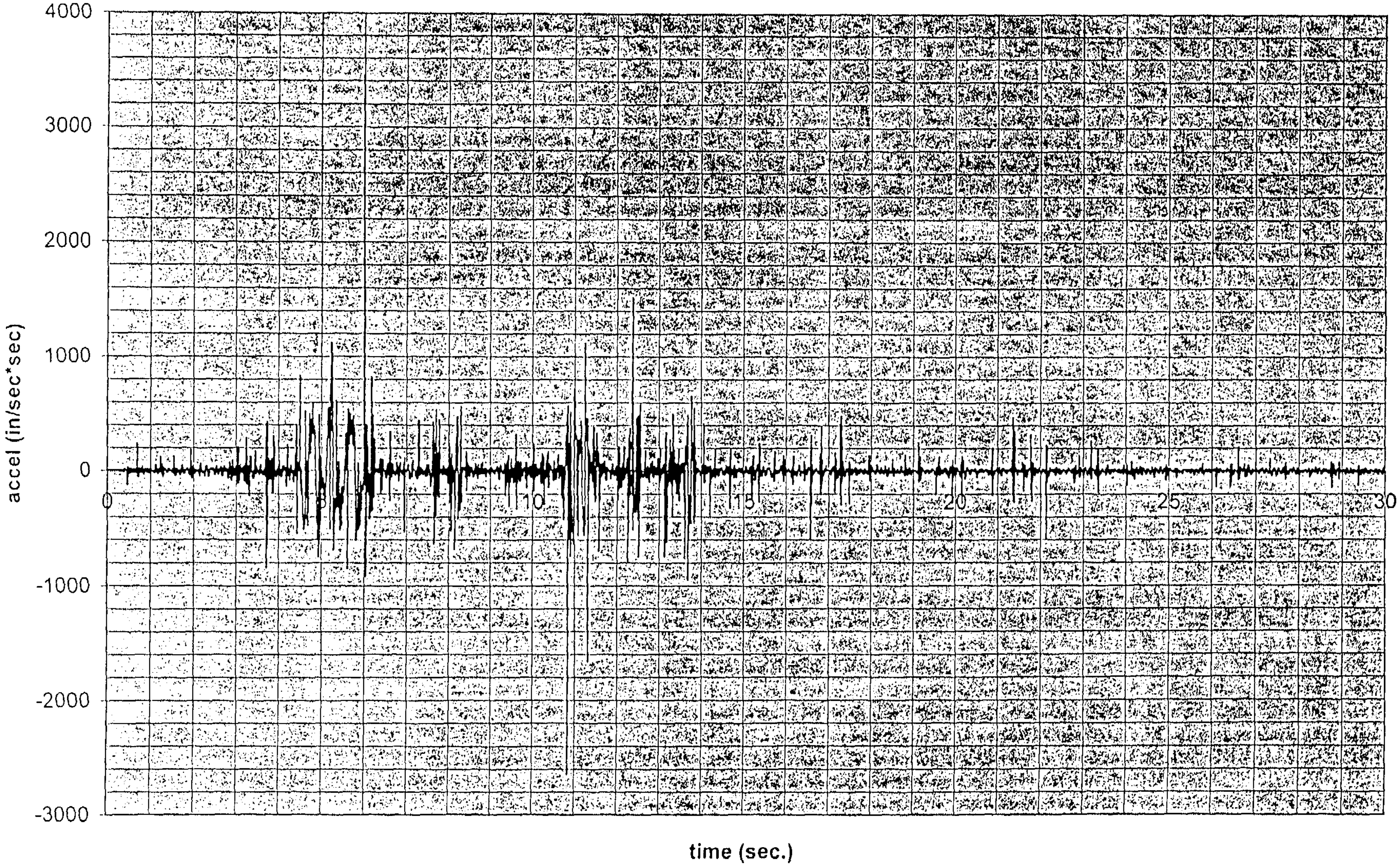

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Fig. 6 Cask 1 Accel-Y vs time, cof=0.8, file:pfs108

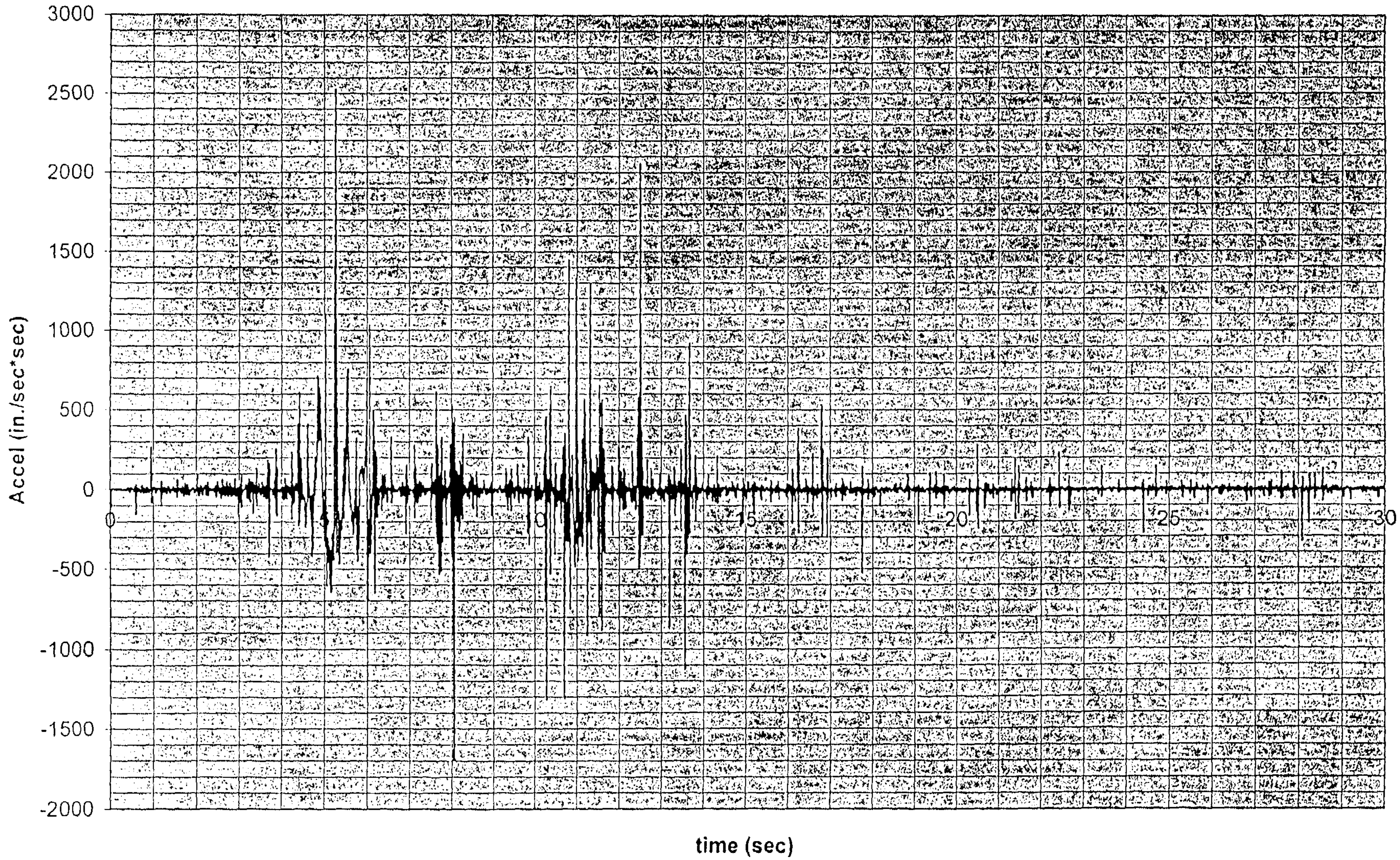

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Fig. 7 Cask 2 Disp-X vs Time at point 2, cof=0.8, file:ck2disp8

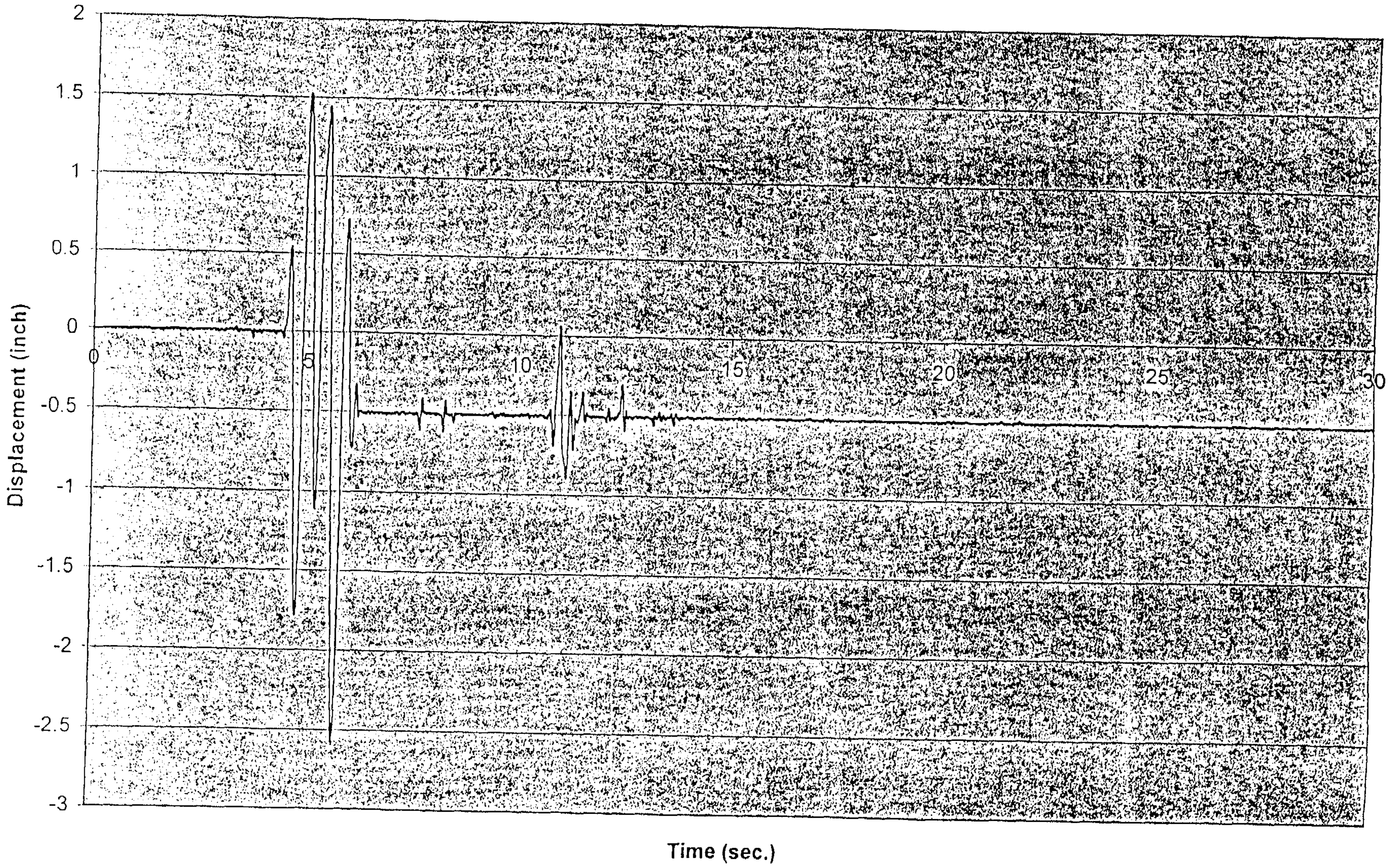

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Fig. 8 Cask 2 Disp-Y vs Time at point 2, cof=0.8, file:ck2disp8

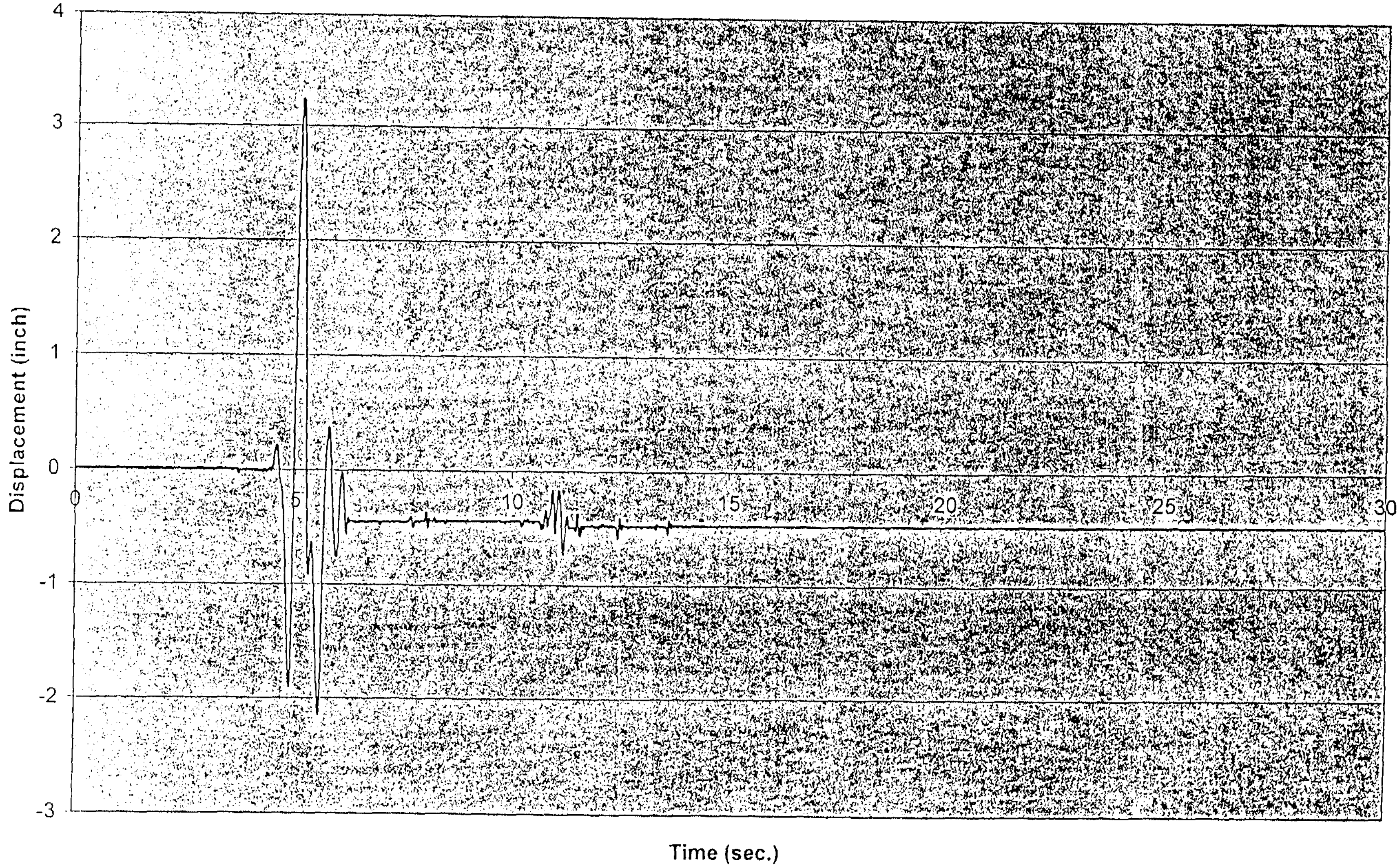

Fig. 9 Cask 2 Velocity X vs Time cof=0.8, file:pfs208

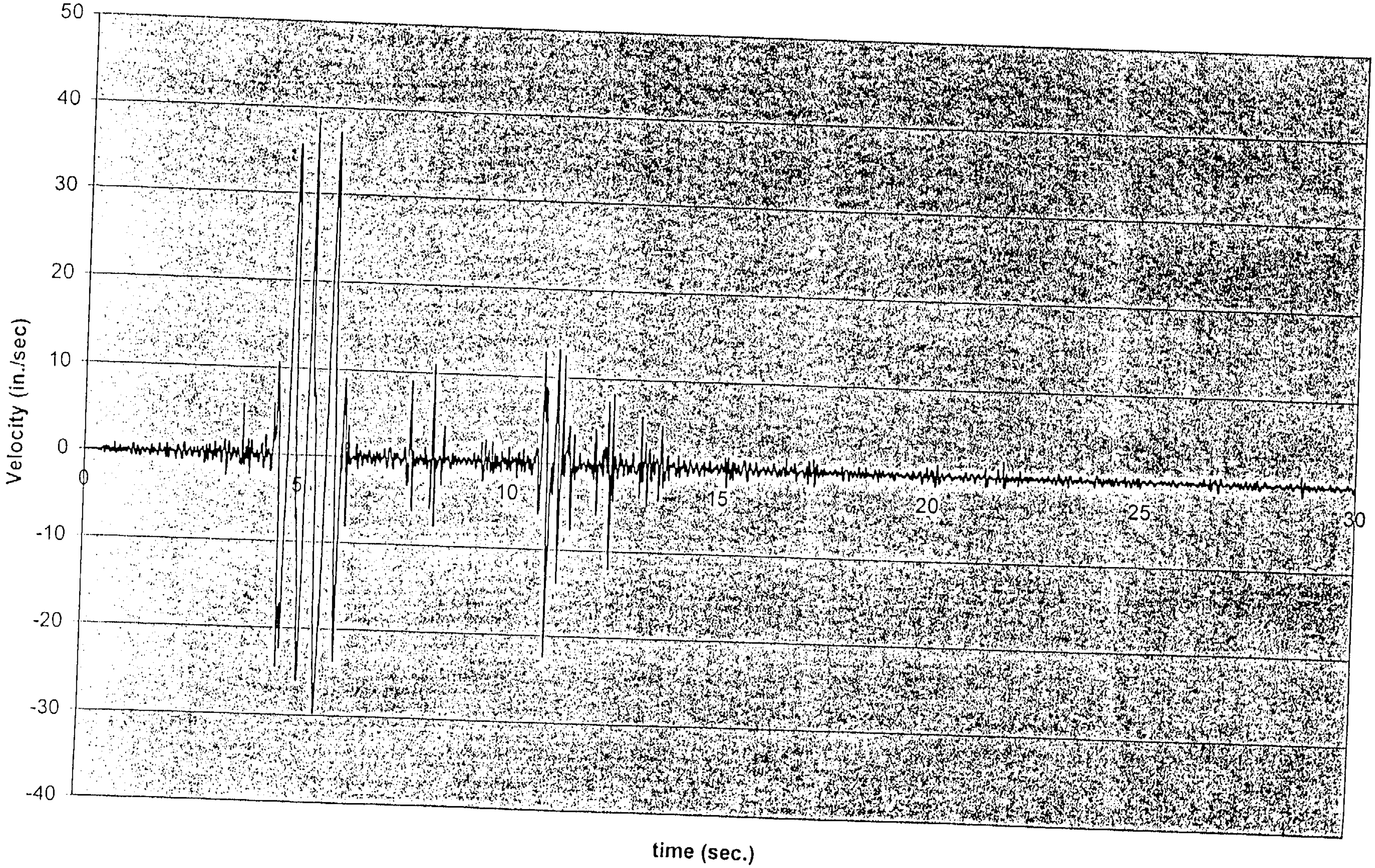

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Fig. 10 Cask 2 Velocity Y vs Time cof=0.8, file:pfs208

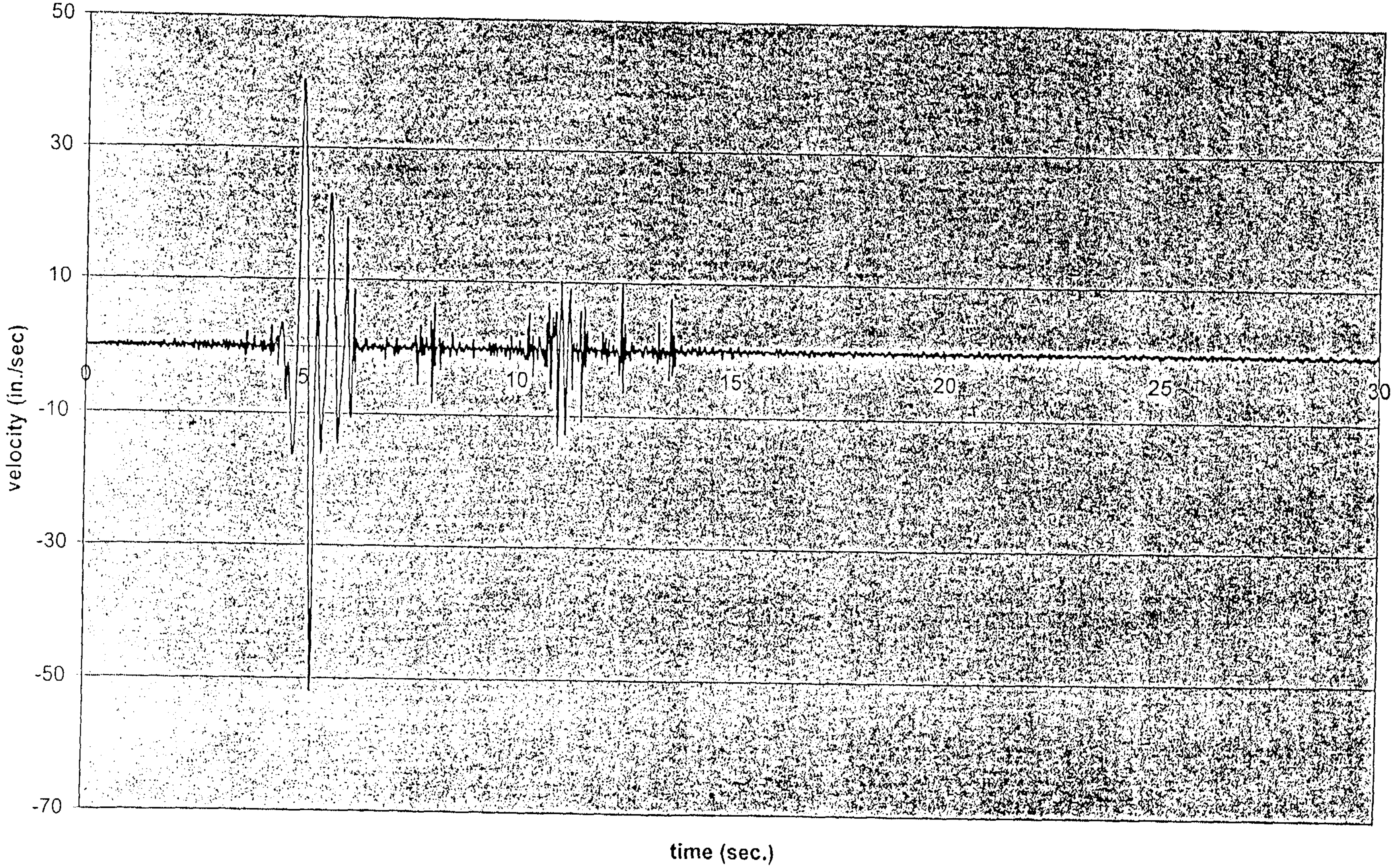

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Fig. 11 Cask 2 X Accel Vs Time coef=0.8, file:pfs208

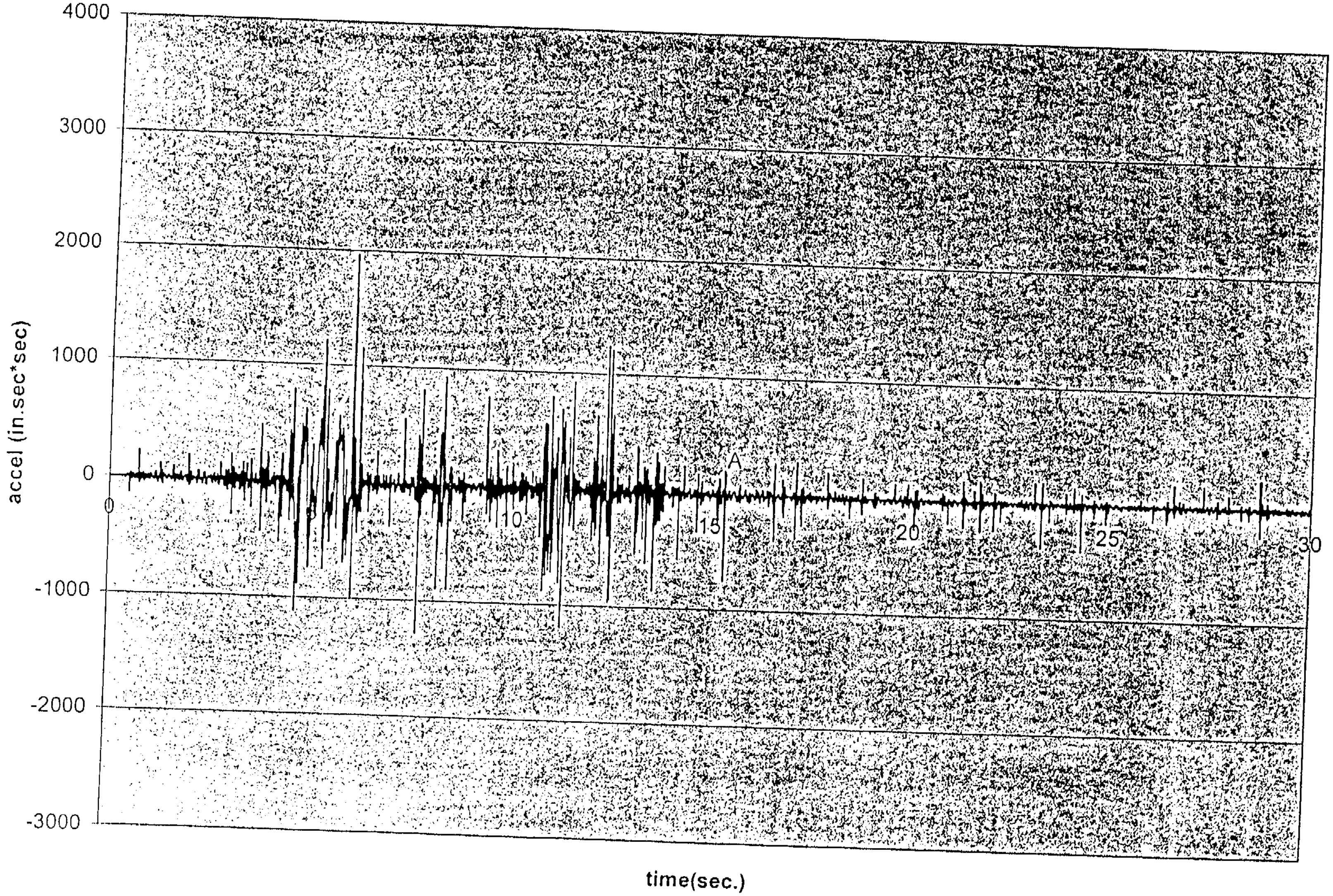

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Fig. 12 Cask 2 Y accel vs time coeff=0.8, file:pfs208

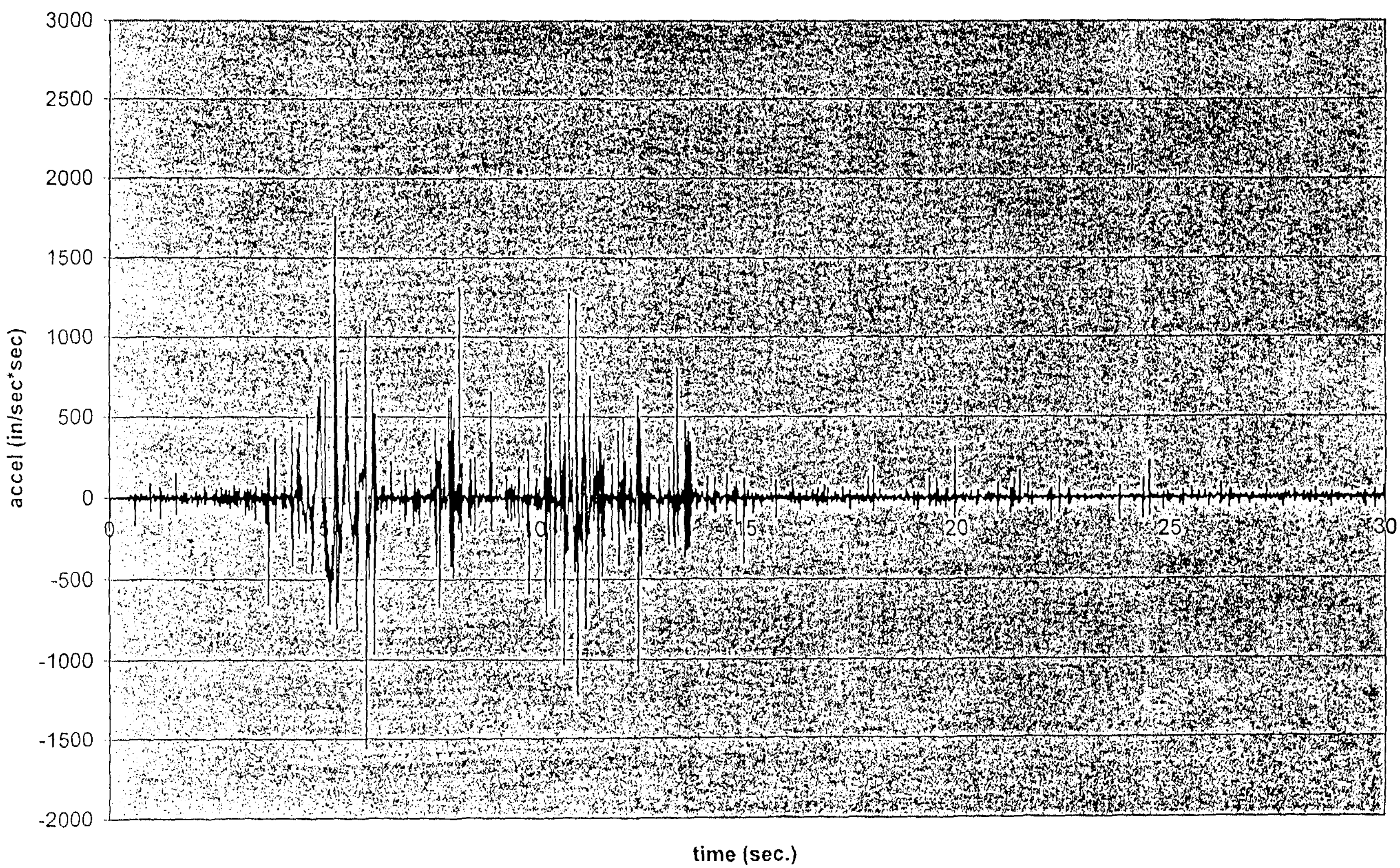

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Fig. 13 Cask 1 Disp-x cof=0.2 at point 2, file:ck1disp2

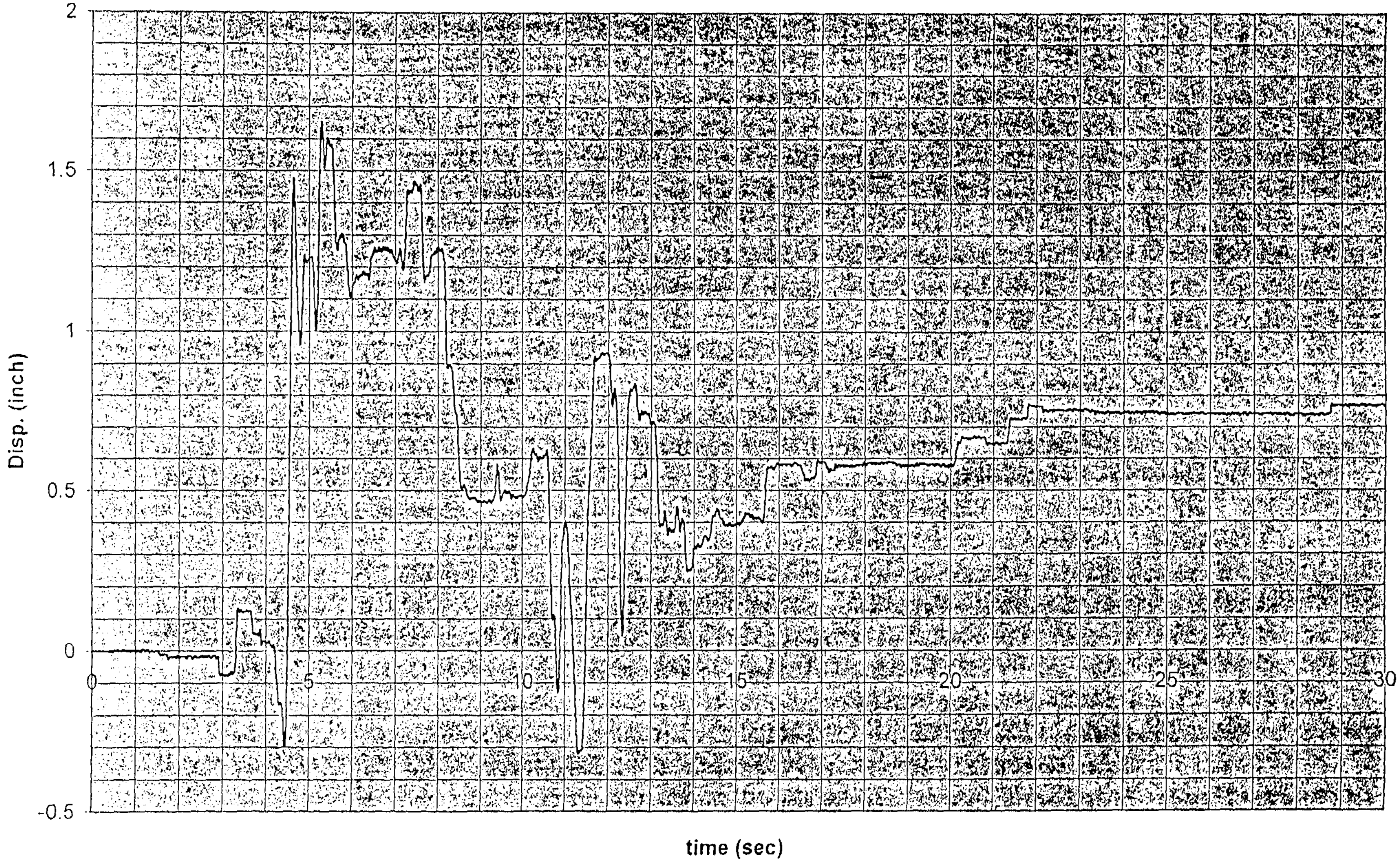

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Fig. 14 Cask 1 Disp-Y, COF=0.2 at point 2, file:ck1disp2

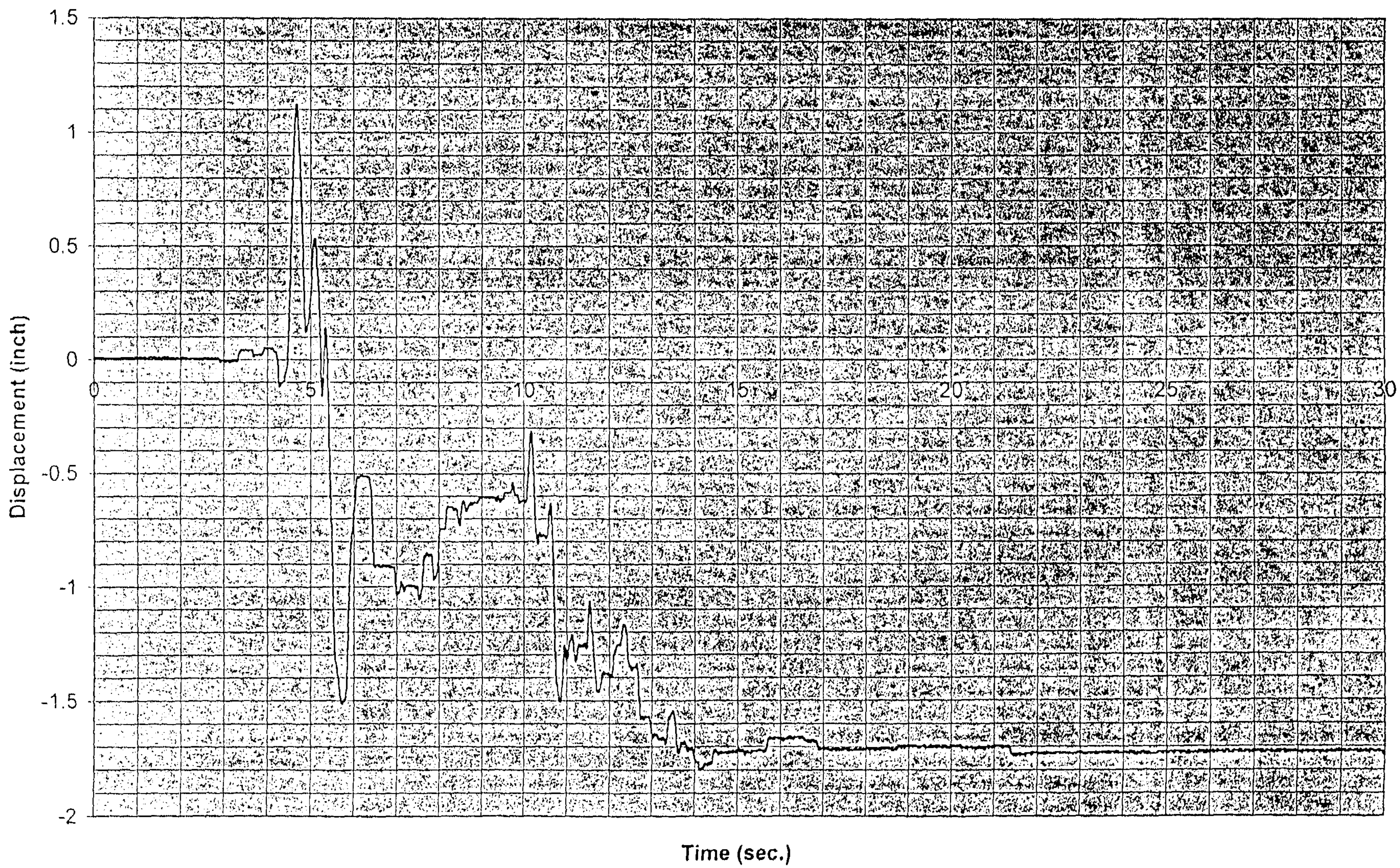

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Fig. 15 Cask 1 Vel-X vs Time,Cof=0.2 at point 2, file:pfs102

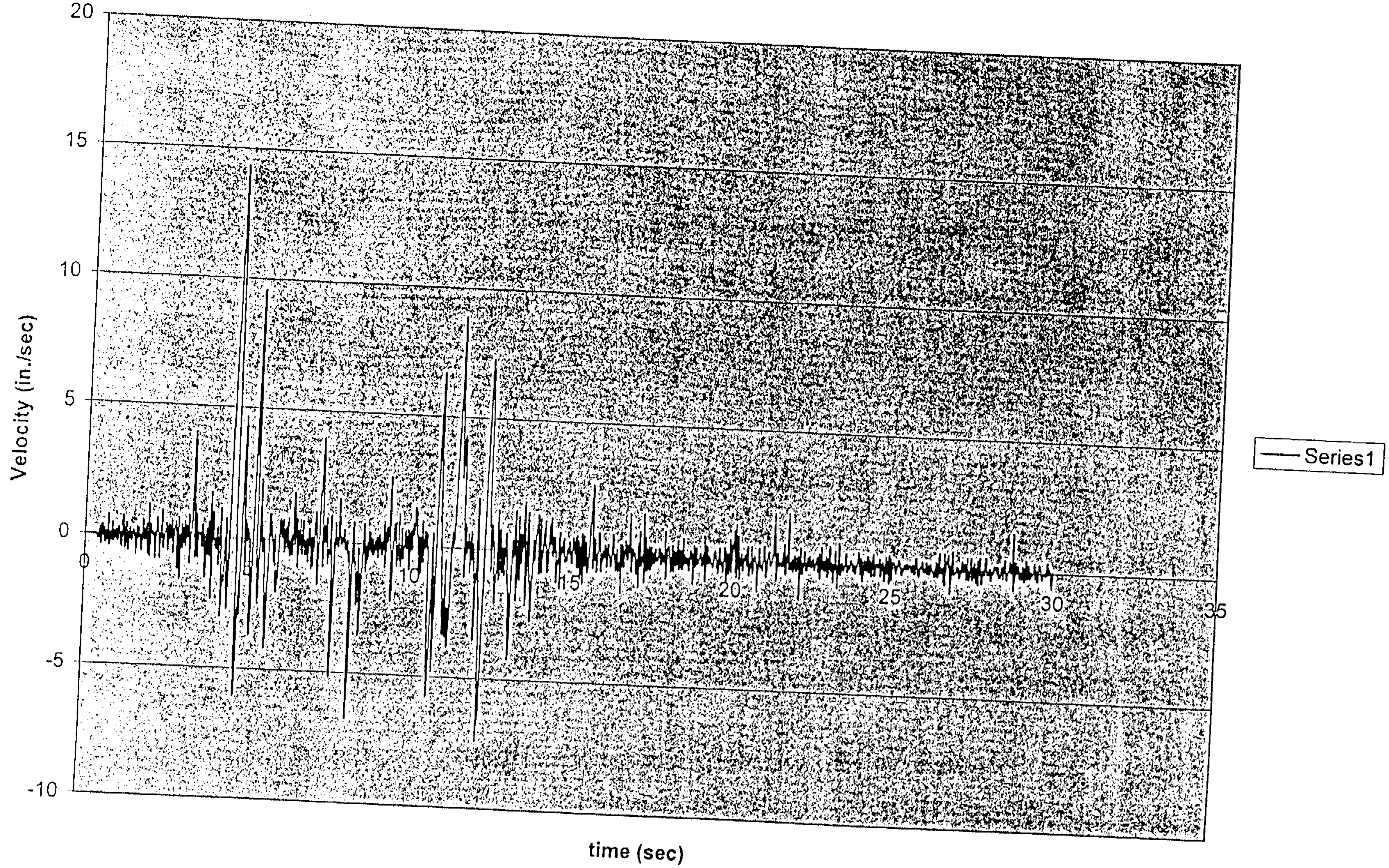

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Fig. 16 Cask 1 Vel-Y vs Time,cof=0.2 at point 2, *file:*pfs102

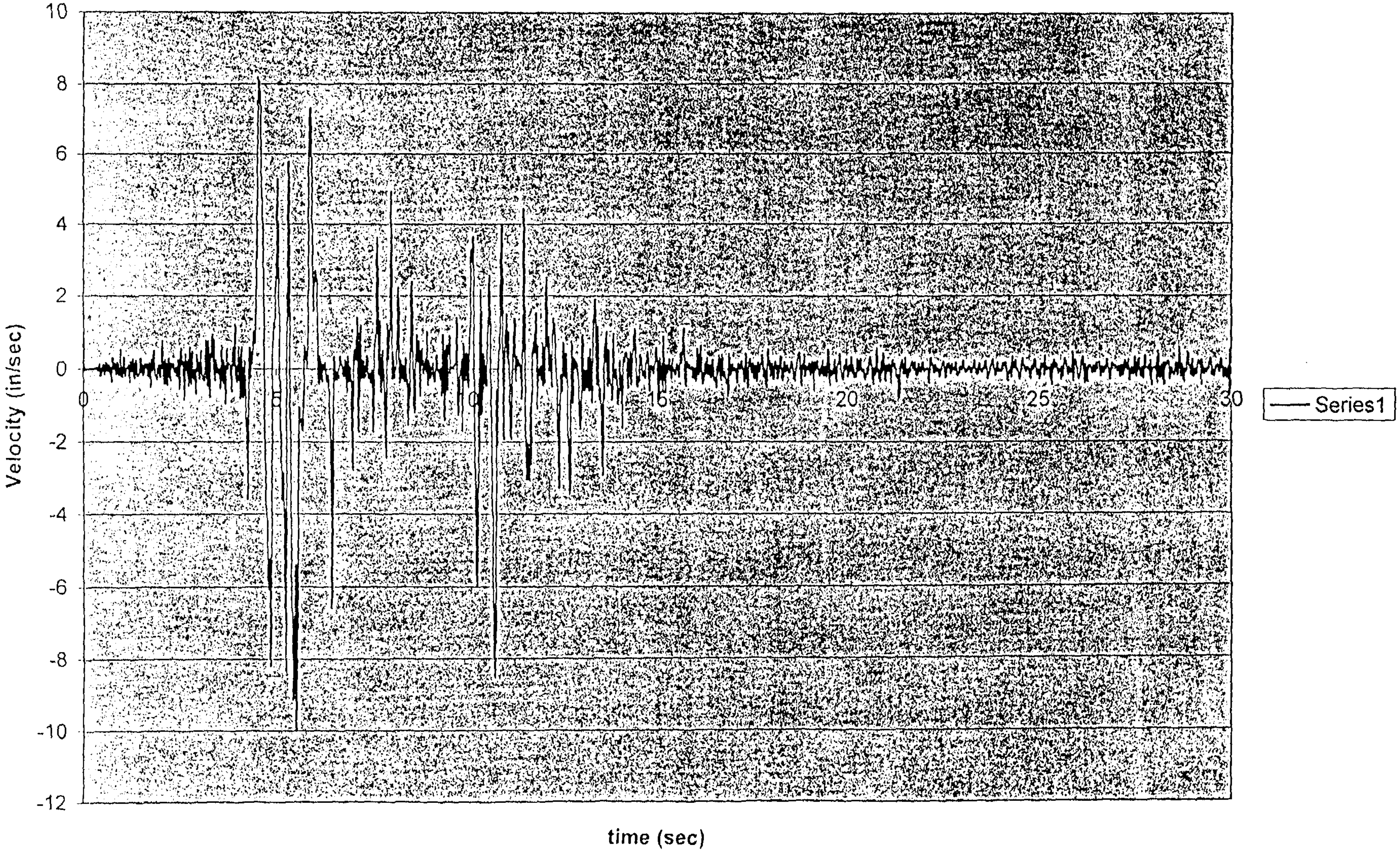

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Fig. 17 Cask 1 Accel-X vs Time,cof=0.2 at point 2, file:pfs102

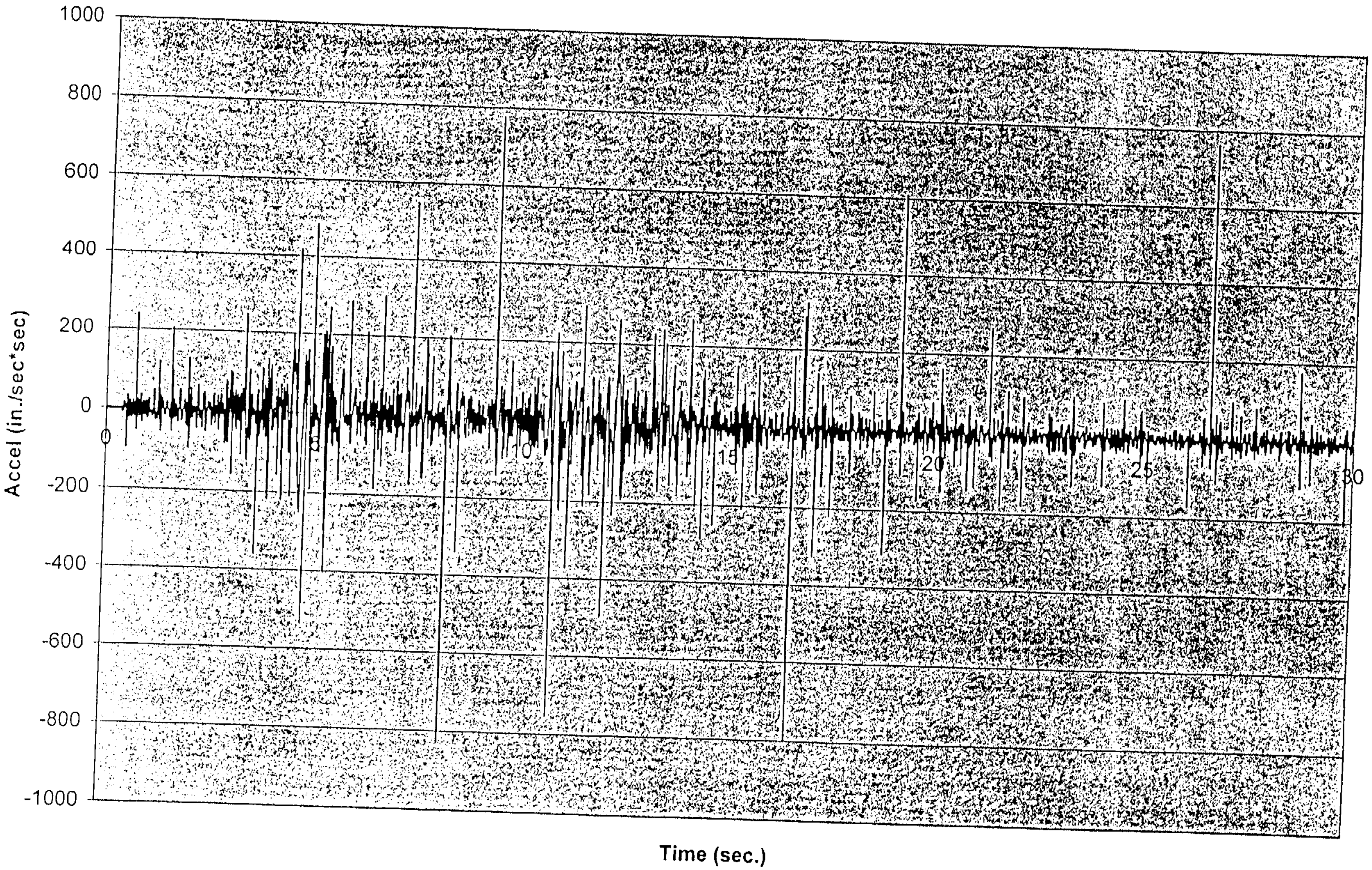

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Fig. 18 Cask 1 Accel-Y vs Time, cof 0.2 at point 2, file:pfs102

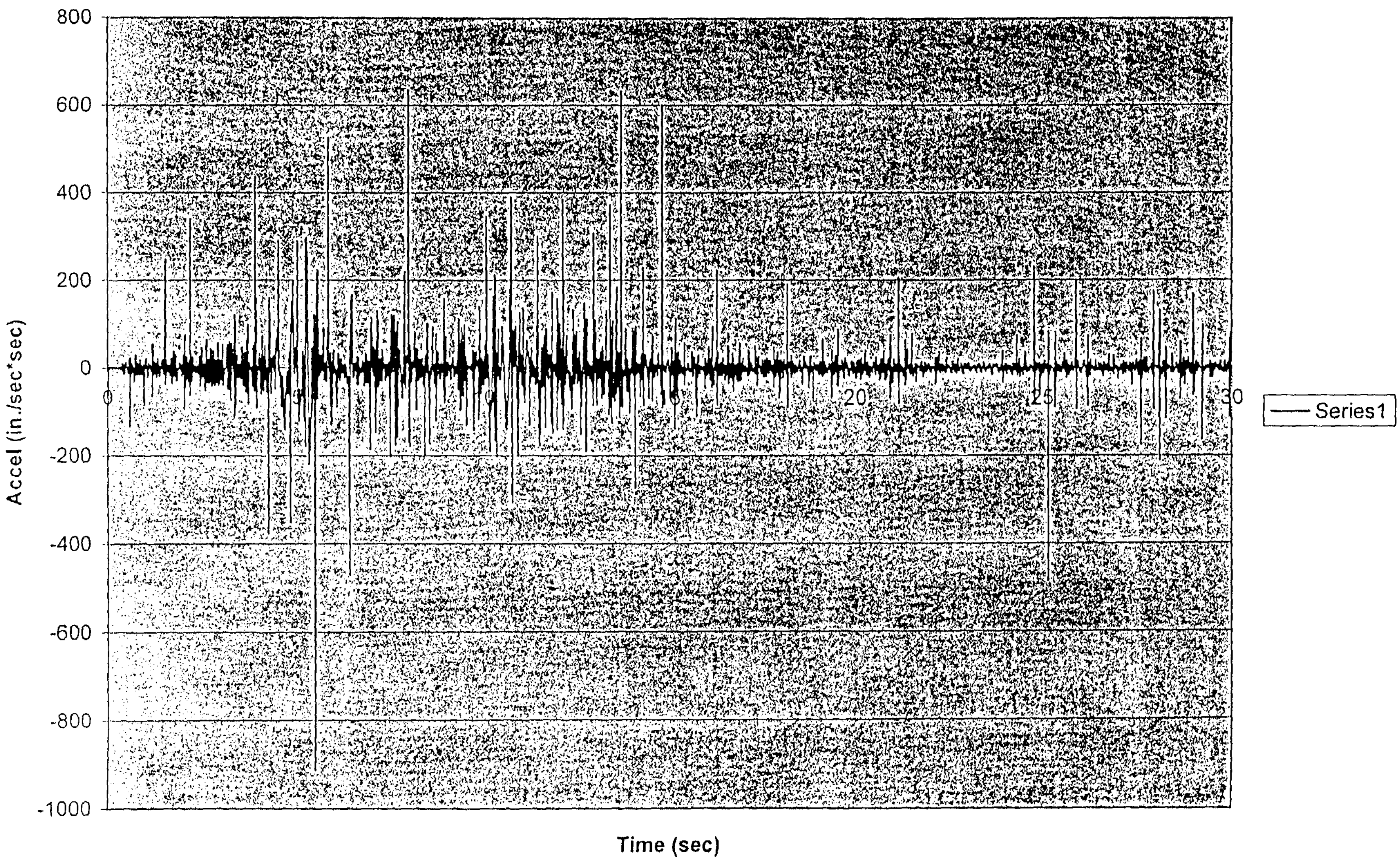

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Fig. 19 Cask 2 Disp-X,COF=0.2 at point 2, file:ck2disp2

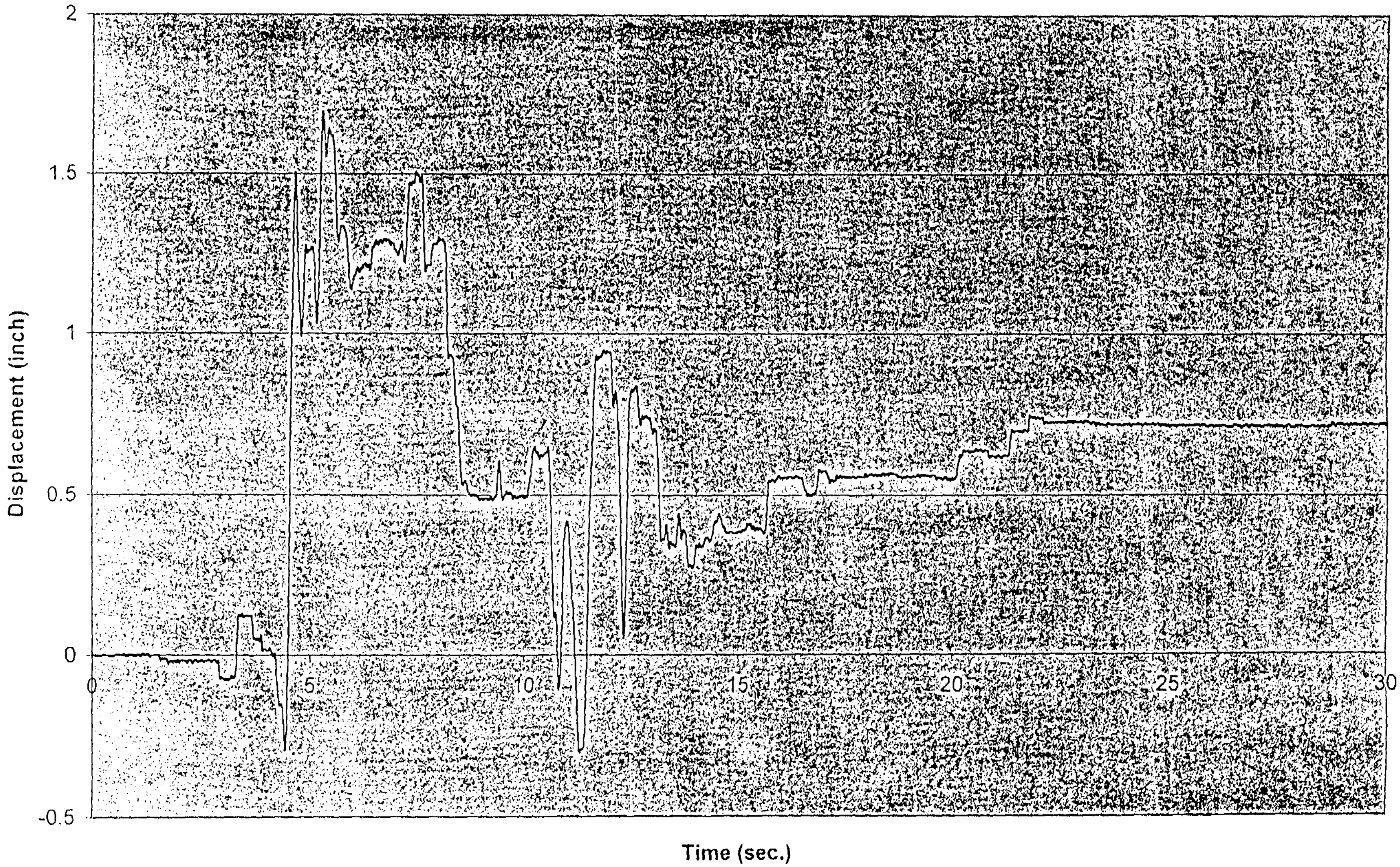

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Fig. 20 Cask 2 Disp-Y, COF=0.2 at point 2, file:ck2disp2

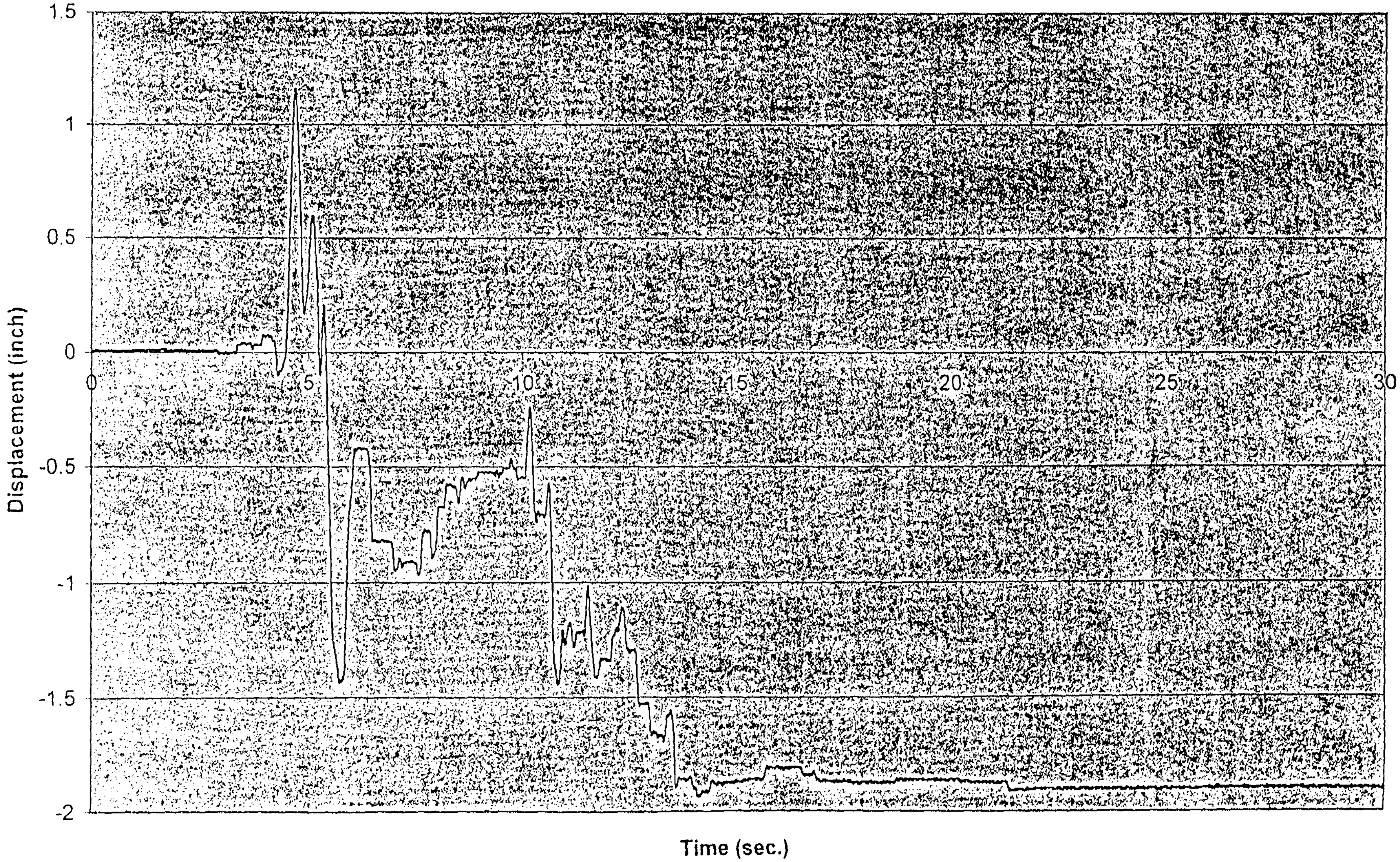

Fig. 21 Cask 2 Vel-X vs time,cof=0.2 at point 2, file:pfs202

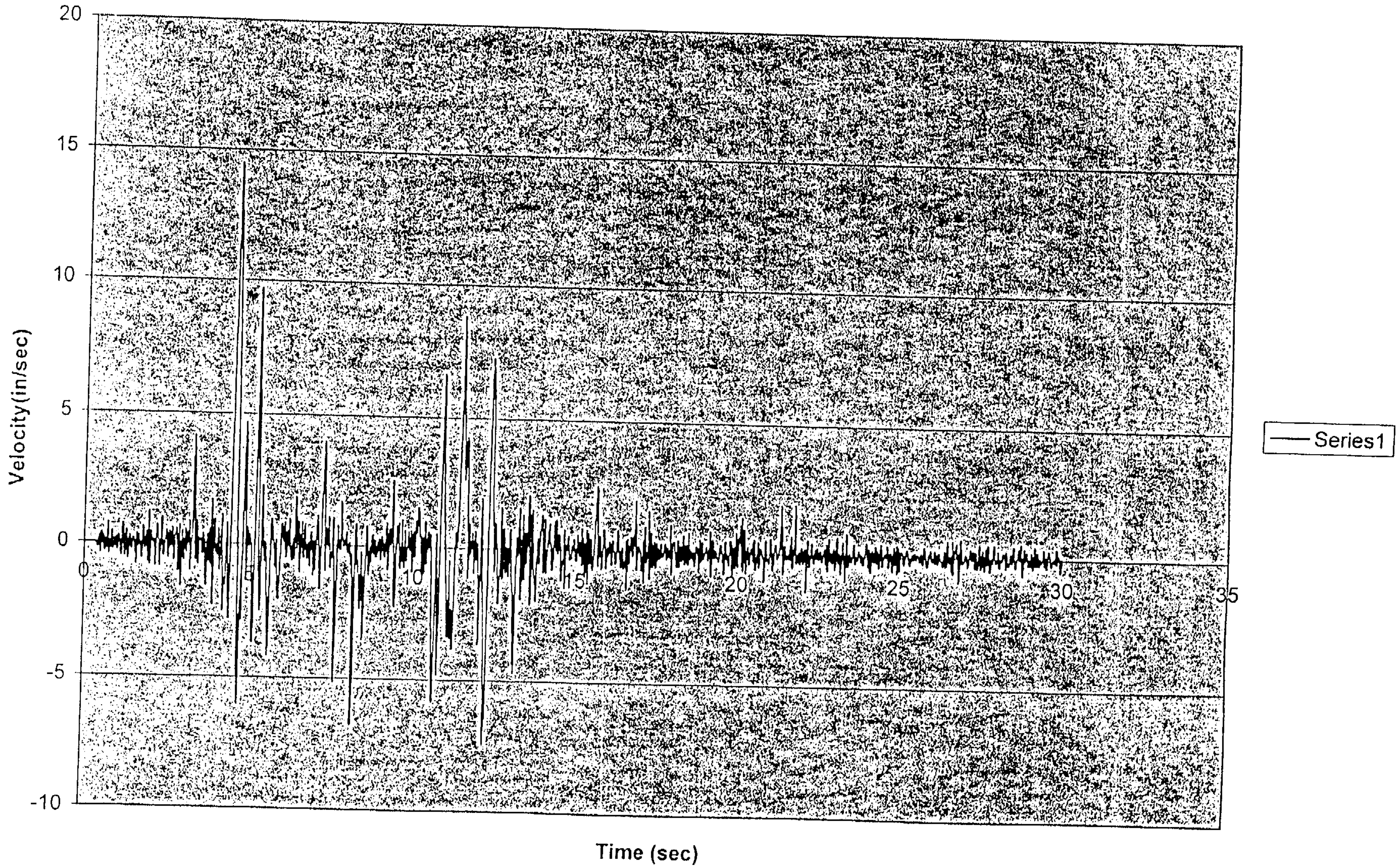

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Fig. 22 Cask 2 Vel-Y vs Time,cof = 0.2, file:pfs202

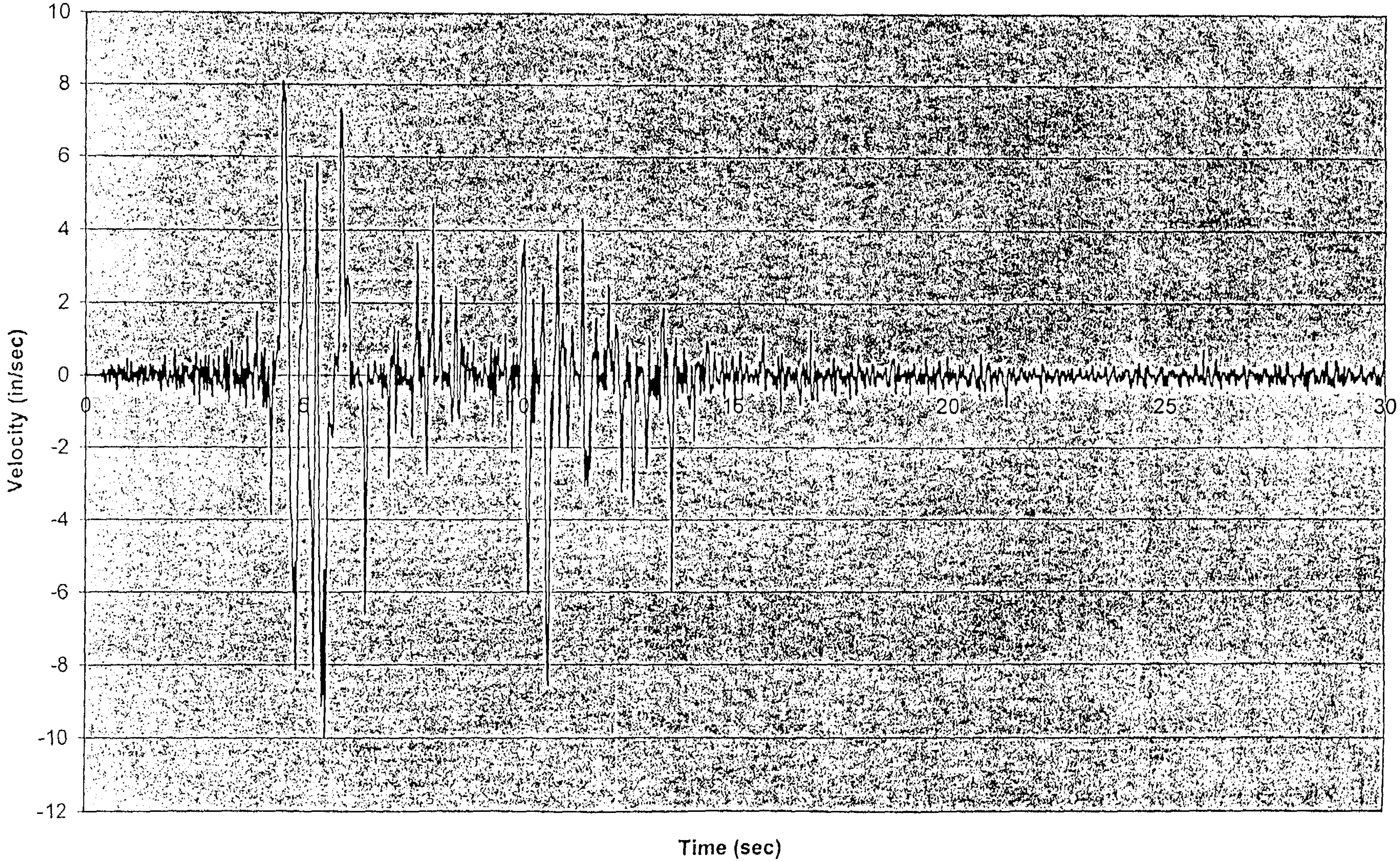

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Fig. 23 Cask 2 Accel-X vs Time,cof = 0.2 at point 2, file:pfs202

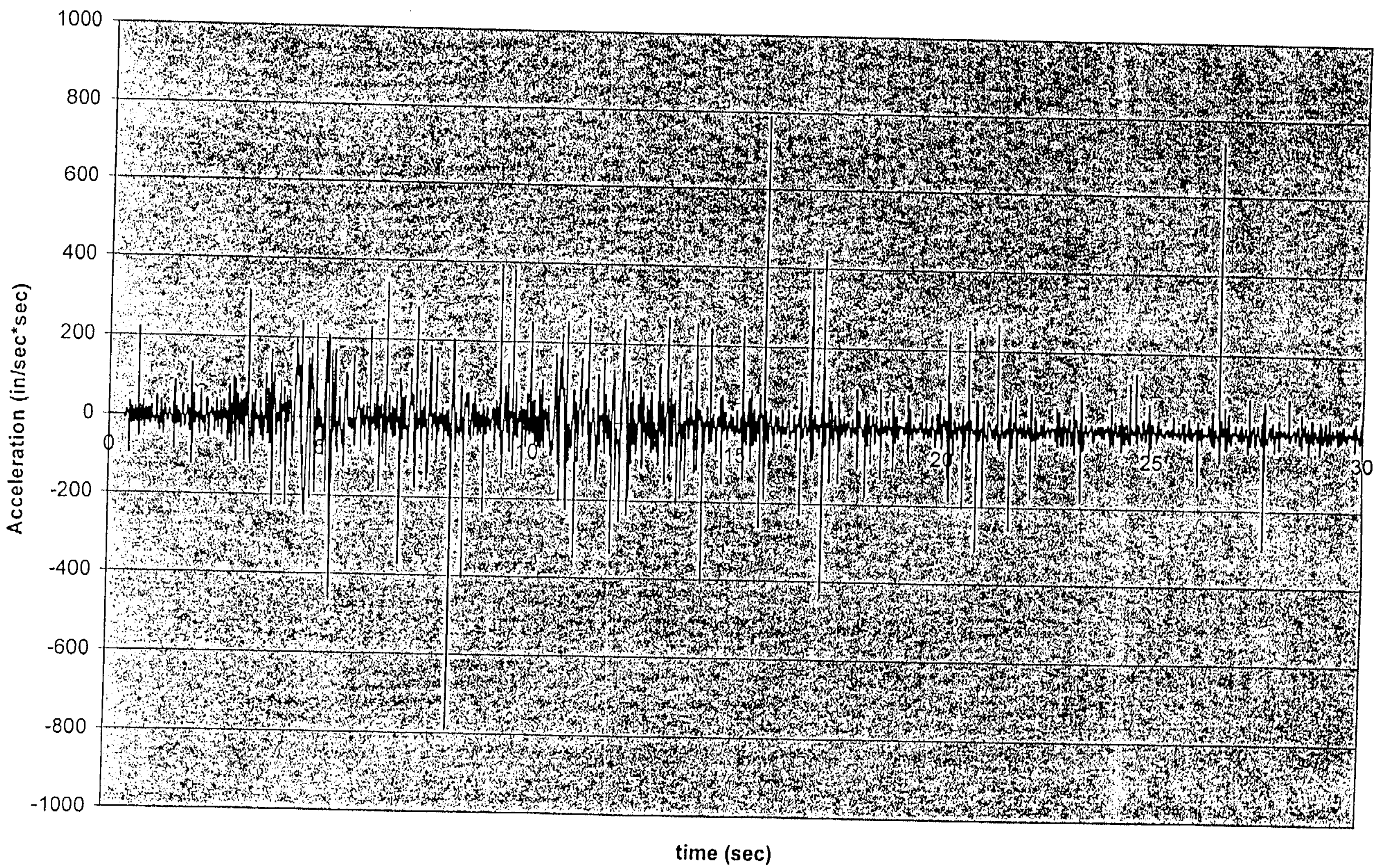

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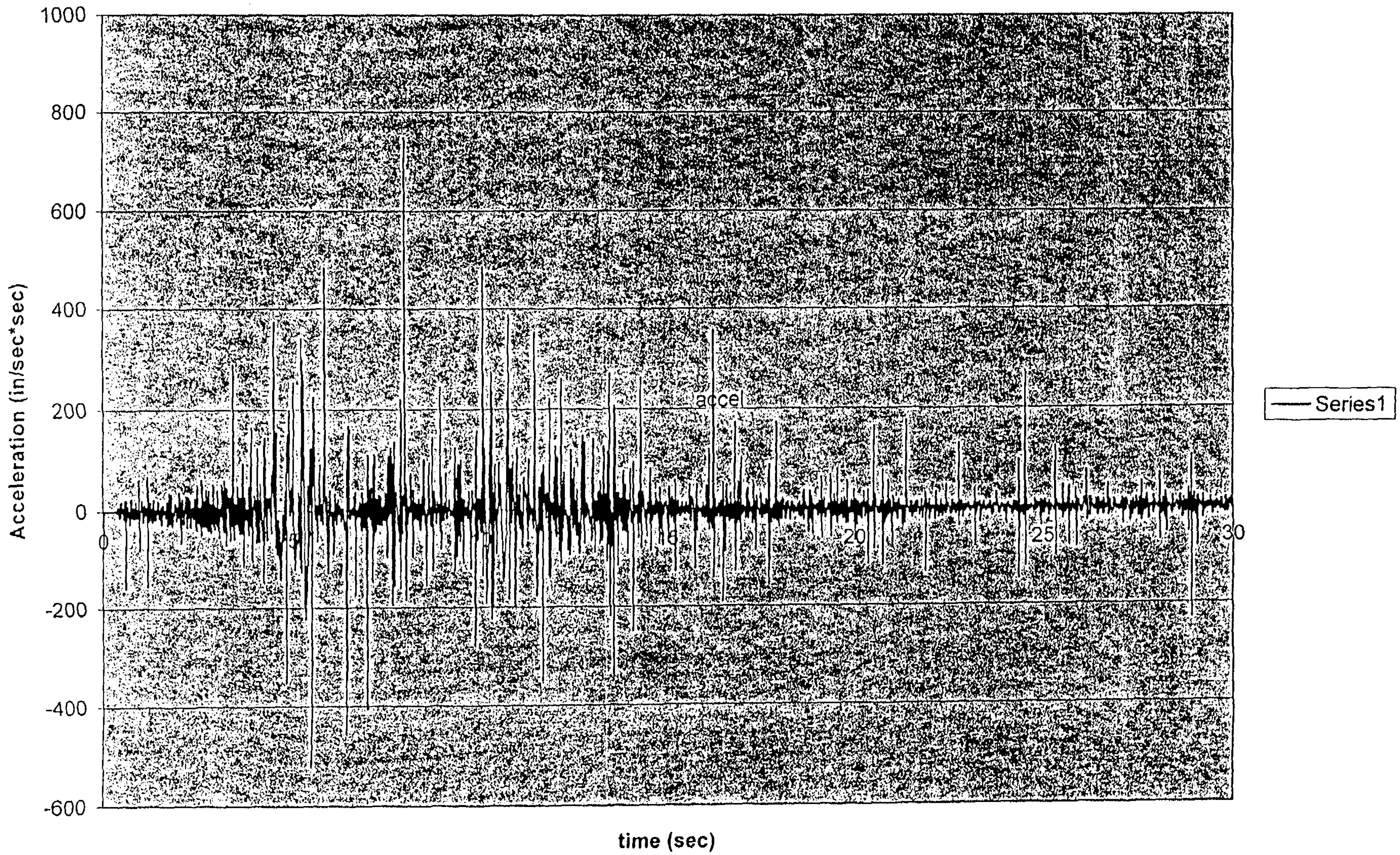
Fig. 24 Cask 2 accel-Y vs Time,cof 0.2, at point 2, *file*:pfs202



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STONE & WEBSTER ENGINEERING CORPORATION

CLIENT & PROJECT	PAGE 1 OF 5
PRIVATE FUEL STORAGE FACILITY – PRIVATE FUEL STORAGE, LLC	PLUS 2 OF ATTACHMENTS

CALCULATION TITLE	QA CATEGORY (X)
Evaluation of Cask Storage Pad Flexibility	__ I - NUCLEAR SAFETY RELATED __ II _X_ 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-21**	**NA**	**345BE**

APPROVALS - SIGNATURE & DATE

PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC. NO	SUPERSEDES CALC. NO. OR REV. NO.	CONFIRMATION REQUIRED (X) YES	CONFIRMATION REQUIRED (X) NO
Bruce E. Ebbeson B.E.Ebbeson 03/28/02	Luben Todorovski L. Todorovski 03/31/02	W. S. Tseng W. Tseng 03/31/02	0	N/A		X

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CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O.NO. 05996.02	DIVISION & GROUP Structural	CALCULATION NO. SC-21	OPTIONAL TASK CODE 345BE	PAGE 3

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PURPOSE

The purpose of this calculation is to evaluate possible effects of pad flexibility on the seismic analysis of the Skull Valley cask storage pads. This assessment is performed to address Paragraphs D.1.b(i) and D.1.b(ii) of the State of Utah's unified contention L/QQ.

METHOD

The influence of the pad flexibility on the impedance functions will be estimated using the information provided in Reference 1.

REFERENCES

1. "Dynamic Response of Flexible Rectangular Foundations on an Elastic Half-Space", M. Iguchi and J.E. Luco, Earthquake Engineering and Structural Dynamics, Vol. 9, 1981.

2. Calculation 05996-G(P017)-2, "Storage Pad Analysis and Design.

ASSUMPTIONS

The paper used as a basis for this evaluation was based on a square foundation on an elastic half-space of uniform properties. The Skull Valley pads are 67' long by 30' wide (Reference 2), and are founded on a layered subgrade. To account for this, following assumptions were made:

1. The size of the equivalent square was determined by using a square with the same area as the rectangular pad.

2. Equivalent uniform soil properties (shear wave velocity, density and Poisson's ratio) were estimated from the best estimate soil properties, presented on sheet 8 of Reference 2, using the top 50' of the soil profile. Values assumed for this evaluation are:

   - Shear wave velocity 750 ft/sec.
   - Density 100 lb/ft^3
   - Poisson's Ratio 0.40 (to be consistent with the value in Ref. 1)

3. The case presented in Reference 1 without the rigid center (model a) was considered more applicable to the real case.

CONCLUSION

The effects of pad flexibility on the impedance functions are not significant.

HISTORICAL DATA REVISION

REVISION NO.	PAGES AFFECTED	DESCRIPTION
Rev. 0	All	Original

CALCULATIONS

Calculate the dimensionless parameter δ:

$\delta = (Et^3)/(\mu\, a^3\, (1-\upsilon^2))$

Where:

E = Young's modulus of the pad (450,000 ksf for 3000 psi concrete)

t = Thickness of pad (3 feet)

μ = Shear modulus of soil = $\rho\, V_s^2$ = (.100 ksf / 32.17)(750 fps)^2 = 1749 ksf

$\upsilon = 0.40$

a = (sqrt (67' x 30'))/4 = 22.4'

Substituting, $\delta = 0.735$

Calculate applicable range of the dimensionless parameter a_o

$a_o = \Omega\, a / V_s$

Where

Ω = frequency of interest, in radians per second

For frequencies between 1 Hz and 5 Hz (6.3 and 31.4 radians per second), a_o ranges from 0.19 to 0.94.

Evaluate effect on impedance functions

From figures 4 (vertical) and 5 (rocking) of Reference 1 for the case of model (a), it can be seen that for values of a_o less than 1.0 and values of δ greater than 0.5 there is little difference in the impedance functions from the completely rigid case. See Attachment A for a details.

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ATTACHMENT A
A-1

The effects of the flexibility of the foundation plate on the vertical and rocking impedance functions are illustrated in Figures 4 and 5, respectively. The results shown in these figures were calculated by subdividing the foundation plate and the contact region into 64 equal square subregions. A value of $\nu = 0{\cdot}4$ was used for Poisson's ratio in the soil. The results presented for values of the relative stiffness $\delta = 0{\cdot}005$, $0{\cdot}05$, $0{\cdot}5$ and ∞ (rigid plate) indicate that, at low frequencies, the dynamic stiffness coefficients $({}_eK_V, {}_eK_M)$ for a flexible foundation plate can be significantly lower than those for a rigid plate. At high frequencies, however, the

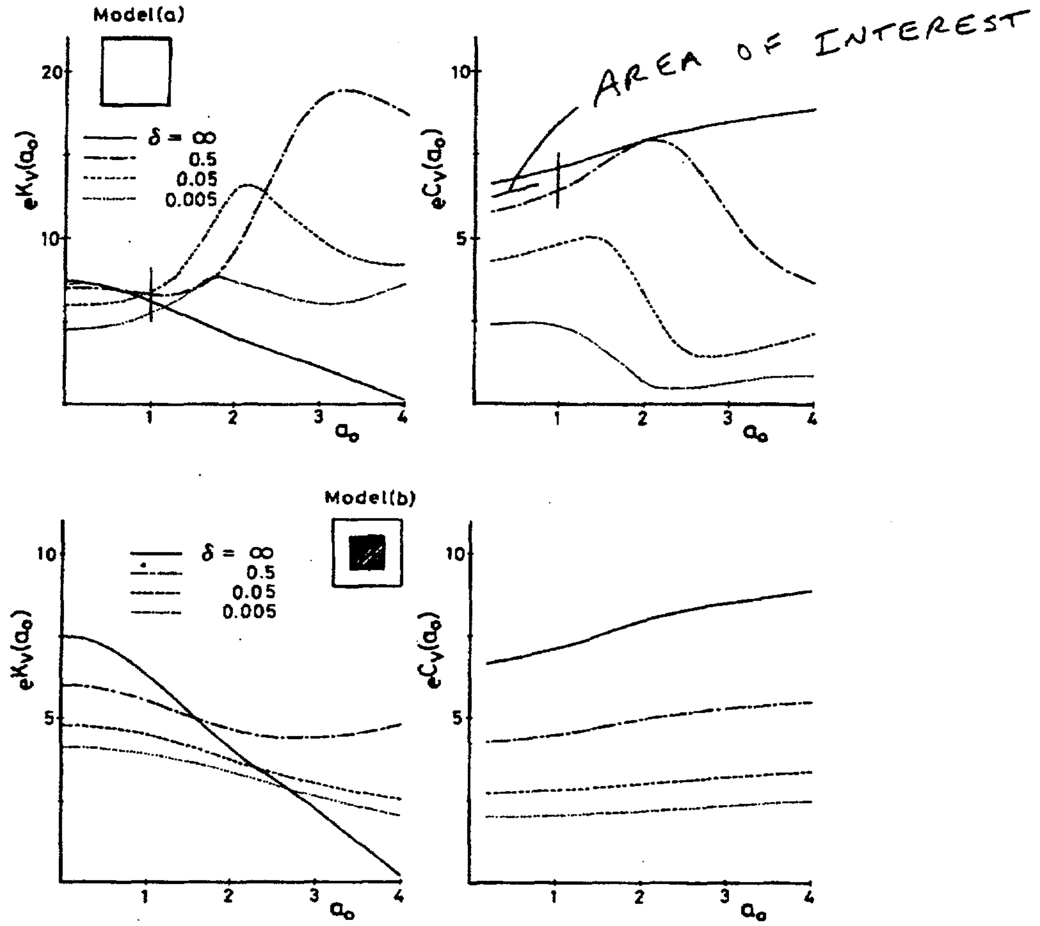

Figure 4. Effects of flexibility of the foundations on the vertical impedance functions ($\nu = 0{\cdot}4$, $\nu_f = 0$)

dynamic stiffness coefficients for flexible foundation plates can be higher than those for a rigid plate. For Model (a), the effects of flexibility of the foundation on the vertical and rocking stiffness coefficients are similar. For Model (b), the effects on the rocking stiffness coefficients are more pronounced.

Perhaps the most significant effect shown in Figures 4 and 5 corresponds to the reduction of the damping coefficients $({}_eC_V, {}_eC_M)$ associated with flexibility of the foundation. It is apparent that a flexible foundation plate is less efficient in radiating energy into the ground than a rigid foundation. For Model (a), the reduction of the vertical damping coefficient is more pronounced than that of the rocking damping coefficient. For Model (b), the reduction of the rocking damping coefficient is more pronounced.

EQUIVALENT RIGID FOUNDATION

Since a considerable amount of information on the dynamic response of rigid foundations is available, it is of interest to explore the possibility of representing a flexible foundation by an equivalent rigid foundation. One possibility is to define the dimensions of the equivalent rigid foundation in such a way that the static stiffness

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ATTACHMENT A

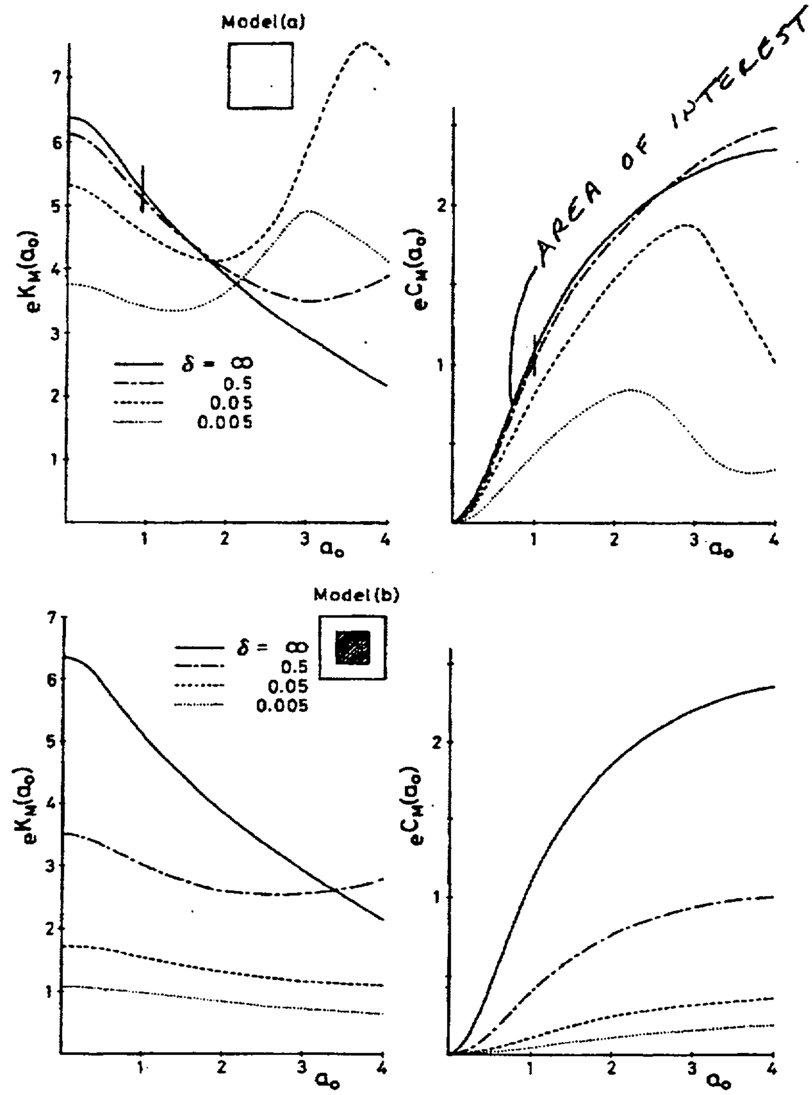

Figure 5. Effects of flexibility of the foundations on the rocking impedance functions ($\nu = 0{\cdot}4$, $\nu_f = 0$)

coefficients (real parts of the impedance function at $a_0 = 0$) coincide with those for the flexible foundation. As an example, the lengths $2\bar{a}_V$ of rigid square foundations having the same static vertical stiffness coefficients as flexible foundations corresponding to Model (b) with $\delta = 0{\cdot}5, 0{\cdot}05$ and $0{\cdot}005$ are defined by $a_V/a = 0{\cdot}798, 0{\cdot}641$ and $0{\cdot}551$, respectively. The corresponding equivalent lengths $2\bar{a}_M$ obtained by equating the static rocking stiffness coefficients are given by $\bar{a}_M/a = 0{\cdot}820, 0{\cdot}646$ and $0{\cdot}552$. For Model (b), values of the ratios $\bar{a}_V/a$ and $\bar{a}_M/a$ range from $1{\cdot}0$ for $\delta = \infty$ to $0{\cdot}5$ for $\delta = 0$. It is interesting to notice that the equivalent length for rocking excitation is slightly higher than that for vertical excitation.

The equivalent rigid foundation described above is based on the static response of the flexible foundation. It is, then, necessary to test the adequacy of the equivalent representation at different frequencies. If the flexible foundation and its equivalent rigid representation give the same force–displacement relationships, the following equations would be satisfied:

$$C_V(a_0) = (a/\bar{a}_V)\,\bar{C}_V(\bar{a}_{0V}) \tag{19}$$

$$C_M(a_0) = (a/\bar{a}_M)^3\,\bar{C}_M(\bar{a}_{0M}) \tag{20}$$

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

American Society of Civil Engineers

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ASCE STANDARD

SEISMIC ANALYSIS OF SAFETY-
RELATED NUCLEAR STRUCTURES &
& COMMENTARY ON
STANDARD FOR SEISMIC ANALYSIS OF
SAFETY RELATED NUCLEAR STRUCTURES

Sept. 1986

Standard for

Seismic Analysis of Safety-Related Nuclear Structures

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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 [redacted] 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+Cv)·G

G/(1+Cv)

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_t.

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.

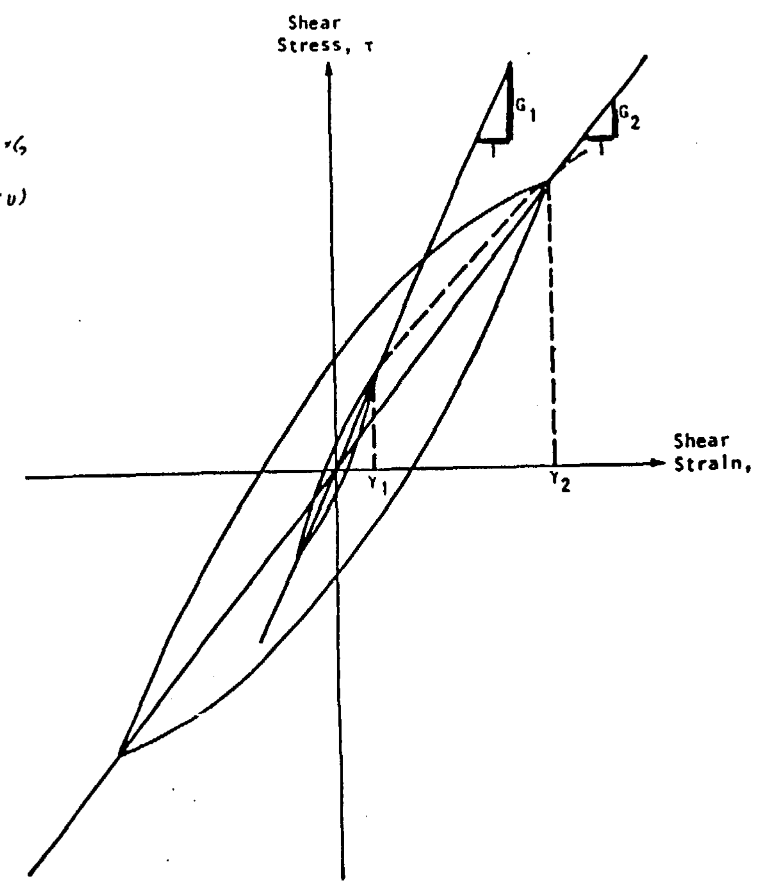

FIGURE 3300-1 DEFINITION DIAGRAM FOR SHEAR MODULUS, G

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 [redacted]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.

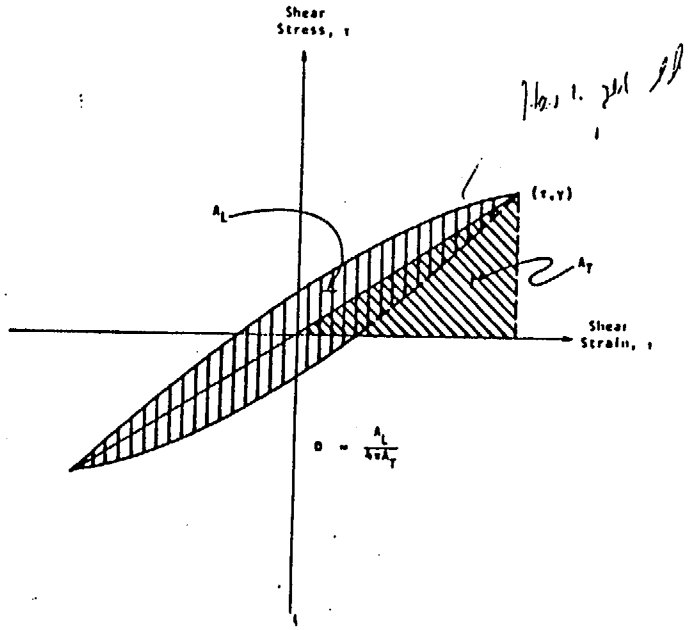

FIGURE 3300-2 DEFINITION DIAGRAM FOR HYSTERETIC DAMPING RATIO, D

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

Private Fuel Storage, L.L.C.

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U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555-0001

August 7, 2001

COMMITMENT RESOLUTION LETTER #37
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In accordance with our July 31, 2001 conference call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA questions and comments regarding the stability analysis for the cask storage pads.

NRC Question/Comment

PFS should provide a basis for the conclusions contained within the SAR that the storage casks do not tip over, collide, nor slide off the storage pad during the seismic event, taking into consideration the potential movement of the cask storage pads of up to 6".

PFS Response

A formal evaluation has been performed for PFS by Holtec International to assess the impact of potential movement of the cask storage pads during a seismic event on the PFS Site Specific HI-STORM Drop/Tipover Analyses, (Holtec Report No. HI-2012653, Revision 1, dated May 7, 2001). The Holtec evaluation is attached for your use.

The results of the evaluation demonstrate that the current conclusions reached in the PFSF Safety Analysis Report remain valid and are bounding for the response of the casks relative to the pad.

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

Sincerely,

JL Cooper for

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

Enclosure

Copy to:

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HOLTEC
INTERNATIONAL

Holtec Center, 555 Lincoln Drive West, Marlton, NJ 08053
Telephone (856) 797-0900
Fax (856) 797-0909

August 6, 2001

Dr. Max DeLong
Executive Engineer
Xcel Energy
414 Nicollet Mall (RS-7)
Minneapolis, MN 55401

Reference: Holtec Project 70651

Dear Dr. DeLong:

In response to your request, we herewith provide the additional information related to the recent site-specific ISFSI pad sliding evaluations performed for Private Fuel Storage (PFS).

SCOPE:

Holtec International has previously performed a series of dynamic simulations of a PFSF ISFSI pad supporting from one to eight spent fuel storage casks and subject to various seismic excitations; these analyses were performed in support of the PFSF site-specific ISFSI licensing submittal. Using design input supplied by PFSF, soil-springs were included in the dynamic model to simulate the effect of the foundation between the base of the ISFSI pad and the top of competent rock driven by the design basis seismic excitation. In the previous Holtec analyses, no separation of the soil from the ISFSI pad lower surface, nor any relative motion (sliding) between the base of the ISFSI pad and the soil surface was assumed. Recent hypothetical bounding analysis (by others) has concluded that postulating loss of surface cohesion could result in as much as six inches of relative displacement of the pad with respect to the soil surface. Therefore, the effect of such relative movement on the response of the casks requires attention. In this letter report, Holtec provides the information needed to conclude that this potential sliding of the ISFSI pad relative to the underlying soil foundation has no significant effect on the conclusions based on the previous dynamic simulations that assumed no sliding.

DISCUSSION:

The loss of cohesion leading to pad movement, relative to the top layer of the soil, is well represented by assuming frictional behavior at the pad/soil interface. Therefore, at some limiting value of horizontal force, the pad begins to move, relative to the soil, and this movement may affect the response of the casks, relative to the pad. Whether the effect on the cask response is detrimental or beneficial is the subject of this letter report.

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We note that the simulation responses to date effectively assume an infinite value for the coefficient of friction between the pad and the soil as the horizontal soil resistance is modeled as a linear spring-damper that is always fully effective. The results from the various simulations predicted minimal movement of the pad and a combination of tipping and sliding of the casks relative to the pad (dependent upon the cask/pad coefficient of friction used). To address the issue at hand, we note that if we postulate the other extreme limit for the pad/soil coefficient of friction, namely zero, then the pad/cask system is fully isolated from the input seismic excitation and the casks experience no motion (either sliding or tipping) relative to the pad. The pad, however, experiences maximum relative movement relative to the soil. Based on this simple physical argument, we are led to the conclusion that any sliding of the pad relative to the soil serves to decrease the energy input to the casks and therefore decreases the motion of the casks relative to the pad. If our argument is valid, then the current FSAR statement (repeated below for completeness) remains valid and supplies bounding values for the response of the casks, relative to the pad.

> "In addition, the vendor performed a site specific analysis for HI-STORM storage casks subjected to the design basis ground motion associated with the probabilistic seismic hazard analysis with the 2,000-yr return period (0.711g horizontal, 0.695g vertical), and determined maximum displacement of the cask of less than 4 inches (Reference 61). The analyses concluded that the casks do not tip over, collide, nor slide off the storage pad for these earthquakes. Soil-structure interaction was considered in the site-specific analyses. The seismic cask stability analyses are fully described in Section 8.2.1."

Although the qualitative argument presented above is convincing in its simplicity, it must be backed by equally convincing confirmatory analyses. A series of dynamic simulations have been performed to confirm the applicability and correctness of the heuristic argument presented previously. Based on these confirmatory results, we conclude that the FSAR statements remain valid as they served to quantify the cask movements relative to the pad.

CONFIRMATORY ANALYSES:

The dynamic simulation model used in all previous submittals on this matter is capable of simulating linear or non-linear behavior across and interface; specifically, the resisting normal force and in-plane forces at the pad/soil interface may be represented by linear springs or by a compression-only normal spring and two orthogonal friction springs. The

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characteristic of each set of two friction springs (FY1, FY2) associated with a compression only normal spring (FW) is as follows:

Let $FH = (FY1^2+FY2^2)^{1/2}$

Then, if the computed value of $FH < \mu\, FW$, the springs FY1 and FY2 behave as simple linear elements at this instant in time with a stiffness and damping associated with the soil.

If the computed value of FH exceeds $\mu\, FW$, then the computed values of FY1 and FY2 are limited to the values that maintain $FH = \mu\, FW$ for the next time step.

Three dynamic analyses were performed using the Holtec QA validated simulation code DYNAMO to evaluate the effect of pad/soil relative motion. These analyses were performed using the following model parameters:

Pad/soil coefficient of friction = 0.306

Seismic input time histories – Latest 2000 Year Return Seismic Event

Cask/pad coefficient of friction = 0.8

Number of casks on ISFSI pad = 8 (2 x 4) array

The three analyses differ in only one aspect; the magnitude of the soil damping associated with the non-linear elements representing normal and in-plane resistance from the soil. For case 1, we assume that the previously computed values for soil resistance due to damping were maintained. For case 2, we assume that the soil damping forces are reduced to 10% of the values used in case 1. Finally, for case 3, we assume that the soil damping forces are reduced to 1% of the values used in case 1. The cases using reduced damping reflect the reality that the damping forces are not active while slip is occurring so that the net effect of the structural damping over the duration of the event must be reduced. The following table summarizes the results obtained for pad center in-plane movement.

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CASE	% OF SOIL DAMPING VALUE PREVIOUSLY USED IN LINEAR ANALYSES	MAX. PAD MOVEMENT (inch) N-S	MAX. PAD MOVEMENT (inch) E-W
1	100	0.537	0.537
2	10	3.989	2.692
3	1	8.808	5.178

As expected, the amount of pad sliding, as a rigid body is a strong function of the level of soil damping assumed to continuously act over the entire duration of the seismic event. Note that cases 2 and 3 bound from either side, the 6" result obtained from a static equivalent analysis using the 100%-40%-40% combination rule.

The results for cask movement relative to the pad from each of the simulations confirmed the initial assertion that as more pad/soil sliding occurred, the cask/pad relative movements decreased and the propensity for cask overturning was nonexistent. For example, for case 2, the maximum cask excursions, relative to the pad, did not exceed 0.02" at the top or bottom of the cask; i.e., even though the cask/pad coefficient of friction was 0.8, the "redirection" of the input energy to moving the pad sufficed to eliminate all overturning cask motion.

Based on the confirming dynamic simulations, we conclude that the initial simulations of the soil/pad interface with linear springs results in the largest values for cask motion relative to the pad; any sliding of the pad relative to the underlying soil due to reduced cohesion has the beneficial effect of reducing or elimination cask movements relative to the pad.

Sincerely,

Brian Gutherman, P.E.
Project Manager

Document ID: 70651014

Emcc: J. Cooper, SWEC

Private Fuel Storage, L.L.C.

7677 East Berry Ave., Englewood, CO 80111-2137
Phone 303-741-7009 Fax: 303-741-7806
John L. Donnell, P.E., Project Director

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555-0001

August 7, 2001

COMMITMENT RESOLUTION LETTER #37
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In accordance with our July 31, 2001 conference call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA questions and comments regarding the stability analysis for the cask storage pads.

NRC Question/Comment

PFS should provide a basis for the conclusions contained within the SAR that the storage casks do not tip over, collide, nor slide off the storage pad during the seismic event, taking into consideration the potential movement of the cask storage pads of up to 6".

PFS Response

A formal evaluation has been performed for PFS by Holtec International to assess the impact of potential movement of the cask storage pads during a seismic event on the PFS Site Specific HI-STORM Drop/Tipover Analyses, (Holtec Report No. HI-2012653, Revision 1, dated May 7, 2001). The Holtec evaluation is attached for your use.

The results of the evaluation demonstrate that the current conclusions reached in the PFSF Safety Analysis Report remain valid and are bounding for the response of the casks relative to the pad.

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

Sincerely,

JL Cooper for

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

Enclosure

Copy to:
Mark Delligatti-1/1
John Parkyn-1/1
Jay Silberg-1/1
Sherwin Turk-1/1
Asadul Chowdhury-1/1
Scott Northard-1/1
Denise Chancellor-1/1
Richard E. Condit-1/1
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HOLTEC INTERNATIONAL

Holtec Center, 555 Lincoln Drive West, Marlton, NJ 08053

Telephone (856) 797-0900
Fax (856) 797-0909

August 6, 2001

Dr. Max DeLong
Executive Engineer
Xcel Energy
414 Nicollet Mall (RS-7)
Minneapolis, MN 55401

Reference: Holtec Project 70651

Dear Dr. DeLong:

In response to your request, we herewith provide the additional information related to the recent site-specific ISFSI pad sliding evaluations performed for Private Fuel Storage (PFS).

SCOPE:

Holtec International has previously performed a series of dynamic simulations of a PFSF ISFSI pad supporting from one to eight spent fuel storage casks and subject to various seismic excitations; these analyses were performed in support of the PFSF site-specific ISFSI licensing submittal. Using design input supplied by PFSF, soil-springs were included in the dynamic model to simulate the effect of the foundation between the base of the ISFSI pad and the top of competent rock driven by the design basis seismic excitation. In the previous Holtec analyses, no separation of the soil from the ISFSI pad lower surface, nor any relative motion (sliding) between the base of the ISFSI pad and the soil surface was assumed. Recent hypothetical bounding analysis (by others) has concluded that postulating loss of surface cohesion could result in as much as six inches of relative displacement of the pad with respect to the soil surface. Therefore, the effect of such relative movement on the response of the casks requires attention. In this letter report, Holtec provides the information needed to conclude that this potential sliding of the ISFSI pad relative to the underlying soil foundation has no significant effect on the conclusions based on the previous dynamic simulations that assumed no sliding.

DISCUSSION:

The loss of cohesion leading to pad movement, relative to the top layer of the soil, is well represented by assuming frictional behavior at the pad/soil interface. Therefore, at some limiting value of horizontal force, the pad begins to move, relative to the soil, and this movement may affect the response of the casks, relative to the pad. Whether the effect on the cask response is detrimental or beneficial is the subject of this letter report.

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We note that the simulation responses to date effectively assume an infinite value for the coefficient of friction between the pad and the soil as the horizontal soil resistance is modeled as a linear spring-damper that is always fully effective. The results from the various simulations predicted minimal movement of the pad and a combination of tipping and sliding of the casks relative to the pad (dependent upon the cask/pad coefficient of friction used). To address the issue at hand, we note that if we postulate the other extreme limit for the pad/soil coefficient of friction, namely zero, then the pad/cask system is fully isolated from the input seismic excitation and the casks experience no motion (either sliding or tipping) relative to the pad. The pad, however, experiences maximum relative movement relative to the soil. Based on this simple physical argument, we are led to the conclusion that any sliding of the pad relative to the soil serves to decrease the energy input to the casks and therefore decreases the motion of the casks relative to the pad. If our argument is valid, then the current FSAR statement (repeated below for completeness) remains valid and supplies bounding values for the response of the casks, relative to the pad.

> "In addition, the vendor performed a site specific analysis for HI-STORM storage casks subjected to the design basis ground motion associated with the probabilistic seismic hazard analysis with the 2,000-yr return period (0.711g horizontal, 0.695g vertical), and determined maximum displacement of the cask of less than 4 inches (Reference 61). The analyses concluded that the casks do not tip over, collide, nor slide off the storage pad for these earthquakes. Soil-structure interaction was considered in the site-specific analyses. The seismic cask stability analyses are fully described in Section 8.2.1."

Although the qualitative argument presented above is convincing in its simplicity, it must be backed by equally convincing confirmatory analyses. A series of dynamic simulations have been performed to confirm the applicability and correctness of the heuristic argument presented previously. Based on these confirmatory results, we conclude that the FSAR statements remain valid as they served to quantify the cask movements relative to the pad.

CONFIRMATORY ANALYSES:

The dynamic simulation model used in all previous submittals on this matter is capable of simulating linear or non-linear behavior across and interface; specifically, the resisting normal force and in-plane forces at the pad/soil interface may be represented by linear springs or by a compression-only normal spring and two orthogonal friction springs. The

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characteristic of each set of two friction springs (FY1, FY2) associated with a compression only normal spring (FW) is as follows:

Let $FH = (FY1^2+FY2^2)^{1/2}$

Then, if the computed value of $FH < \mu FW$, the springs FY1 and FY2 behave as simple linear elements at this instant in time with a stiffness and damping associated with the soil.

If the computed value of FH exceeds μFW, then the computed values of FY1 and FY2 are limited to the values that maintain $FH = \mu FW$ for the next time step.

Three dynamic analyses were performed using the Holtec QA validated simulation code DYNAMO to evaluate the effect of pad/soil relative motion. These analyses were performed using the following model parameters:

Pad/soil coefficient of friction = 0.306

Seismic input time histories – Latest 2000 Year Return Seismic Event

Cask/pad coefficient of friction = 0.8

Number of casks on ISFSI pad = 8 (2 x 4) array

The three analyses differ in only one aspect; the magnitude of the soil damping associated with the non-linear elements representing normal and in-plane resistance from the soil. For case 1, we assume that the previously computed values for soil resistance due to damping were maintained. For case 2, we assume that the soil damping forces are reduced to 10% of the values used in case 1. Finally, for case 3, we assume that the soil damping forces are reduced to 1% of the values used in case 1. The cases using reduced damping reflect the reality that the damping forces are not active while slip is occurring so that the net effect of the structural damping over the duration of the event must be reduced. The following table summarizes the results obtained for pad center in-plane movement.

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The results for cask movement relative to the pad from each of the simulations confirmed the initial assertion that as more pad/soil sliding occurred, the cask/pad relative movements decreased and the propensity for cask overturning was nonexistent. For example, for case 2, the maximum cask excursions, relative to the pad, did not exceed 0.02" at the top or bottom of the cask; i.e., even though the cask/pad coefficient of friction was 0.8, the "redirection" of the input energy to moving the pad sufficed to eliminate all overturning cask motion.

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Sincerely,

Brian Gutherman, P.E.
Project Manager

Document ID: 70651014

Emcc: J. Cooper, SWEC

COPY OF TRANSCRIPT

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	) Docket No. 72-22) ASLPB No. 97-732-02-ISFSI
PRIVATE FUEL STORAGE L.L.C.	)) DEPOSITION OF:
(Private Fuel Storage Facility)	)) **DR. MOHSIN R. KHAN**
	))) (Utah Contention L/QQ)

Tuesday, March 5, 2002 - 12:10 p.m.

Location: Office of Parsons, Behle & Latimer
201 S. Main, Suite 1800
Salt Lake City, Utah

Reporter: Vicky McDaniel
Notary Public in and for the State of Utah

Q. When you say a correct solution --
A. For the input parameters that you use.
Q. I guess it means they gave you the same solution?
A. That's right, for the parameters that you use.
Q. And suppose you input the wrong parameter in the SAP -- in both codes, for example.
A. Then you will have both wrong results, sure. But for same, identical input, both give identical solution. For those parameters, it's benchmark.
Q. So if the input's wrong, the output's wrong on both of them?
A. Exactly.
Q. Garbage in, garbage out?
A. Exactly, sure.
Q. Now, in paragraph 62 you describe ANSYS as a, I believe another general purpose program?
A. Yes.
Q. I take it by use of the word "other" that SAP2000 is also a general purpose structural analysis program?
A. Yes.
Q. And what do you mean by a general purpose code?

drop the cask and it's 5 percent damping. How far should that cask be going back up?

A. I mean, I don't know how much it will bounce back, but --

Q. Can you calculate, if you know the damping is 5 percent, you drop a cask from 10 inches and the damping is 5 percent, can you tell me -- can you calculate how much the cask will bounce back up?

A. I'm sure, given all the programs, one could find. But to me is, again, when you are dropping this cask, there is no -- there is no damping associated to the cask when it touches the surface. At that time energy start dissipating, and the stiffness of the restraining object is the one that's absorbing most of the energy, okay? It is not the damping.

So if you are assuming that the damping is a significant contributor, you have to quantify it. You're taking two components which are resisting the motion. We don't know what the damping is. All we know is, for an object that deforms significantly, depending on the energy level, you could assume in that direction. But in the upward direction you have no -- you have no stiffness, so what kind of damping you're going to have in the upward direction.

Q. I thought it was only damping that

dissipated the energy. Am I wrong there?

A. I believe that there are two mechanisms which absorbs energy, absorption of energy. Your spring is absorbing energy to reduce the motion. Okay? When you have pure sliding, your sliding is actually dissipating energy and reducing the motion. When you have zero friction you can slide a long distance. Okay? When you have -- so in a vertical direction your spring is providing similar kind of behavior as your friction is providing in an unanchored system when it's moving on a horizontal surface.

Q. So you're saying that when I drop something, it's not the damping that's dissipating the energy but it's the spring on the ground absorbing the energy?

A. A significant amount of energy is absorbed by the crushing of the springs that you have.

Q. Well, I take a spring, okay. Take a spring. I push it down. Doesn't the spring push back up at me?

A. That's your spring action, but that's not the damping action. You could take this thing and crush it, okay? It's not the damping that has stopped it. It is the stiffness at the start of the motion.

Q. But doesn't the spring force back up again, or forces the cask back up again?

A. The spring will force it back again.

Q. And there's basically some damping associated with the movement of the spring in that sense, but the spring doesn't go -- doesn't force the cask back up as high as it was before?

A. It's only based on a small amount of damping. We don't know what that will be. It depends on the level of energy that you have. It depends on level of stress you have in the system. If you have a zero stress, you will have no -- very small damping.

Q. In your model, okay -- what did you say about zero stress again, now, just a second ago? Can you read that back?

(The record was read as follows: "It's only based on a small amount of damping. We don't know what that will be. It depends on the level of energy that you have. It depends on level of stress you have in the system. If you have a zero stress, you will have no -- very small damping.")

A. See, the Reg Guide I believe 161 defines for various level of earthquake for the kind of structures that you have which are going through certain deformation what kind of damping you should have. So if your structure is going through a state of deformation that you expect it's going to have, you associate damping with those values. And that's why, you know, if you have an anchored cask where you are going to see high level of stresses in the anchored board or a structural member, you use 4 to 5 percent damping. For low-level earthquake where your stress level in the component are smaller, you use low damping values. And that's the basis of damping.

Q. Are you familiar with the term "impact damping"?

A. Yes.

Q. What does that mean?

A. The damping that could be associated during an impact. And that's experimentally determined, by the way.

Q. And so you have the cask impacting the pad; there would be impact damping between the cask and the pad?

A. Yeah. And that would depend on the level of impact, type of impact. It's a function of the amplitude.

Q. Do you account for impact damping in your modeling?

A. There is some small amount of damping equal to what I reported in this calculation that's for every structural element that's associated.

Q. In your model does the spring element dissipate any energy?

A. Every element is used to deform, and therefore it absorbs energy or interacts against the motion.

Q. It's appropriate to have the spring to absorb energy, then?

A. Sure. We use spring all the time.

Q. Going back to my example of the ball. If you drop the ball from a foot or ten inches, whatever the case may be, and if you have no damping, doesn't the ball bounce back as high as it was before?

A. It's a coefficient of restitution depending on how the ball is -- it also depends on the gravity is pulling, you know. Let's say we can jump 50 feet, but we will always come back to earth because the gravity's always pulling you down. So it has nothing to do with damping. You will always be on the ground.

Q. But if you have no damping and you drop the ball at a certain force, it will go back up the same distance that you drop it from?

A. Well, if you don't have the absorption of energy as it bounces, if you have a perfectly rigid surface, infinitely rigid surfaces, you could say the coefficient of restitution between the two is such that you have no dissipation of energy. But you still have the gravitational forces acting against an object, and it will always try to bring it back to the earth.

Q. You mentioned something, restitution?

A. Coefficient of restitution.

Q. What's the coefficient of restitution?

A. I think it's the -- as the -- when you drop the ball, the ratio between the force impacted and the reactive force, it gives you, whether it's a perfectly rigid bounce or it's an elastic bounce.

Q. And if you have a perfectly elastic surface, what happens then?

A. Well, perfect elastic surface, it should bounce back. But there is no perfectly elastic surface. There is no such thing as perfectly elastic and there is no such thing as perfectly plastic. Everything on this earth deforms. Okay? So a ball will always go back to the ground after a certain bounce.

Q. Now, the coefficient of restitution, is that a function of damping or is that a function of something else in addition to damping?

A. It could be function of whole bunch of phenomena surrounding when the ball falls. Could be, if there's a high wind, that could stop it. If it's in a vacuum, you may have a different thing. Surrounding does affect what happens to the ball.

Q. Can you measure the loss of energy by the percent of coefficient of restitution? In other words, if something goes down and it comes up only so high, it means it's lost a certain amount of energy, correct?

A. But see, the question is, which is the absorbing phenomenon? Is your energy being absorbed by the surface that deforms it, or is it something else? And I think it's anybody's guess. It could be the elastic surface is absorbing some of the -- as it crushes, it absorbs energy. So it could be a combination. I can't say for sure.

Q. Suppose I throw a ball. Give you another example. Suppose I throw a ball, horizontal motion. Suppose I throw a ball against a wall and it comes back a certain amount, okay? And it doesn't come back all the way. What absorbs some of the energy? Is it the spring or damping or what?

A. Like I said, it could be a combination. Could be anybody's guess. It could be air resistance, actually.

Q. What studies have you done with respect to impact damping percentage, coefficient of restitution?

A. We never -- we never relied upon impact damping values.

Q. So you didn't do any work on impact damping values?

A. In our judgment we felt that the duration of impacts were too short to include those damping, so we used simply pure stiffness values. For local element where it impacts, for that element it was purely elastic collision and no damping were allowed.

Q. So no damping?

A. For that element.

Q. So you didn't study impact damping or analyze it?

A. No.

Q. We've used a couple terms, okay, energy absorption and energy dissipation. Used it in connection with different elements -- damping, spring, etc. Could you define what you mean by energy dissipation, first of all?

A. That's a loss of energy.

Q. A loss of energy?

A. Yes, during friction phenomenon.

Q. During friction?

A. Yeah, loss.

Q. Excuse me?

A. During friction to surface.

Q. Can it be loss of energy other than

friction?

A. Sure.

Q. So it's a loss of energy from an object?

A. Yeah. You could have damping in the system, you could have a crushing in the system, you could have a permanent deformation in the system.

Q. And how do you define energy absorption?

A. Energy absorption?

Q. Yeah.

A. You apply force and something deforms, and it does not respond. It captures that energy and retains it. You have loss of energy. That's how you absorb the energy.

Q. Suppose I crush a Coke can, okay? Is that absorption or dissipation?

A. It's an absorption. This is absorption.

Q. Does a linear elastic spring dissipate energy, or does it only absorb energy?

A. Absorbs energy.

Q. It doesn't dissipate?

A. No, it does not dissipate. The damping associated with that elastic motion would dissipate energy. So a spring always when it is associated with a damper, that's why they call it spring damper element. A damper basically dissipates the energy using the damping effect.

When you have an equation of motion you have three components to it, and they all are in equal degree. You have inertia, you have forces which are a function to velocity, and then you have forces which are a function to stiffness. When you add them together, that forms the equilibrium of the equation. And then you may also have in it frictional phenomena as an item, friction effect.

Q. One last question in this area. It's your position that 5 percent is too much, whether you view it as energy dissipation by damping or energy absorption by the spring. Is that correct?

A. That is my judgment.

Q. So it's not a matter of how you define it?

A. Yeah.

Q. You're claiming that there's no loss of energy that's sufficient -- equal to 5 percent?

A. Yeah, for this -- when you're doing this sliding and taking friction into consideration, using 5 percent -- equivalent to 5 percent damping for those gap elements is high.

Q. And for that you would include any energy absorption by the spring itself?

A. Yeah, because you are absorbing all the energy through friction anyways, and I would use a very small damping.

Q. If you have vertical motion up and down, you're basically saying it's all through friction?

A. No. There's no friction when it's vertical motion up and down.

Q. Isn't that a part of the modeling here?

A. If you model as it as a 3-D model, then friction is dominating your phenomena. Okay? What is the most dominant phenomenon, okay? If your most dominant phenomenon is your sliding, then the friction is the one that's taking care of all the energy in the system.

(Recess from 4:33 to 4:45 p.m.)

Q. I if could turn to some of the results you have in your table in the report. First of all, I'd just like to look at Table 3. This is, Table 3 is the result of the third mathematical model where you've assumed motion in all three directions, X, Y, and Z?

A. Plus cask height, effect of cask height, whatever the structural properties are.

Q. Now, you have in here -- in this Table 3 you have a column called Stiffness for Non-Linear Elements, and we have the vertical stiffness column.

A. Yes.

A. We use stiffness values all the time, every time we analyze the structure. For an anchored cask it could be zero in the upward direction.

Q. So how many times have you picked a contact stiffness value for sliding analysis?

A. A program --

Q. How many times have you picked a contact stiffness value for sliding, for lift-off analysis?

A. For this case?

Q. No, just in general. How many times have you picked a contact stiffness analysis for purposes of analyzing sliding or tipping?

A. This is the case.

Q. This is the first case?

A. Yes.

Q. First time you've done it, correct?

A. That's right.

Q. Okay. Dr. Khan, you say in paragraph 70, I believe it is, "The Altran analysis did not take into account for the amplification due to soil structural interaction in the 2,000-year earthquake input time histories." Then you go on to say, therefore, the vertical input motions at the base of the cask should be higher. I'm confused what you're saying in that paragraph 70. I think you also have something in your provide you this value you should use for the stiffness.

Q. I said, did you follow the guidance that ANSYS provides in determining what stiffness to use?

A. ANSYS never provided any guidance on sliding, how to calculate the stiffness for a sliding problem.

Q. Did you follow the guidance of ANSYS in terms of how to arrive at appropriate contact stiffness guide for the problem you were looking at?

A. They would never give you an answer.

Q. So there is no guidance from ANSYS?

A. I used their program.

Q. You didn't use any guidance from them in terms of how to develop the appropriate contact stiffness for the problem you were working?

A. They would never say for a sliding problem what contact --

Q. I'm not asking that. I'm just saying, you did not follow the guidance of ANSYS with respect to arriving at the proper contact stiffness?

MS. NAKAHARA: Asked and answered.

A. There is no guidance. All I can say is, there is no guidance from ANSYS how to solve a nonlinear sliding problem with large horizontal motions.

Q. Okay. In paragraph 72 of your declaration, I believe you say that, in your opinion, "the only way to validate Holtec's analyses is for Holtec to benchmark its sliding displacements calculated by Holtec's non-linear mathematical model with actual shake table test data. This is common practice in the seismic performance field. I frequently perform shake table tests to benchmark mathematical models."

How would you go about doing a shake table test for the Holtec cask?

A. Well, you know, find a shake table, maybe in Japan or someplace. I'll have a prototype model, shake it and apply the ground motion that you see. And you'll benchmark your nonlinear solution with a sliding displacement, impact loads inside the casks, and then substantiate your model, that this is what you're getting from your analysis and from your testing for sliding, displacement, tipping, and whatnot. And then you could go and use a bigger cask, a different size, because at that time you have a basis, parametric basis for that model.

Q. You've mentioned Japan. Why do you mention Japan?

A. They have a bigger table.

G-1-2

JOURNAL OF APPLIED PHYSICS VOLUME 24, NUMBER 9 SEPTEMBER, 1953

Static and Sliding Friction in Feedback Systems

J. Tou* and P. M. Schultheiss
Department of Electrical Engineering, Yale University, New Haven, Connecticut
(Received January 16, 1953)

One of the most common nonlinearities encountered in servomechanisms design is the friction phenomenon in electromechanical systems. Conventional linear theory fails to predict its effect upon system performance. This paper extends familiar techniques to the treatment of friction nonlinearity in servosystems. Frequency-response methods are employed throughout and the theoretical results are verified by means of an analog computer. Sliding friction and static friction are represented by describing functions which form the critical factors in determining system stability. The analysis indicates that certain series equalizers designed from linear theory may fail to achieve effective compensation in the presence of sliding and static friction. On the other hand, a subsidiary loop may avoid the stability problem while still realizing an essentially equivalent loop gain function.

I. INTRODUCTION

WHILE basic analysis and synthesis procedures for linear feedback systems have become well established during the last decade, there is no correspondingly broad approach to nonlinear problems. Except in very simple cases, no general solutions are possible, and the designer must rely either on machine computation or on various linear or quasi-linear approximations. A variety of such approximations has been developed to fit numerous types of systems and successful design procedures have been discovered for a great many practical problems. It is the purpose of this paper to extend one of these techniques so as to make it applicable to the analysis of feedback systems involving sliding and static friction. Particular attention will be paid to certain loop gain functions which appear to be quite satisfactory on the basis of linear analysis but are found to be unstable in practice as a result of friction phenomena. Methods of predicting, and hence presumably preventing, such behavior will be outlined.

II. REVIEW OF BASIC PROCEDURE

The technique to be employed was first devised by Kochenburger[1] for the analysis of contractor servomechanisms and subsequently adapted for use with other nonlinear devices.[2,3] The basic procedure has been described extensively in the literature[1-3] and will therefore be outlined only briefly. It is unique in that it permits use of the frequency domain in an approach to problems involving certain types of nonlinear elements. If a sinusoidal voltage is applied to a nonlinear device, the output is generally not sinusoidal. However, under rather general conditions the fundamental component of the output will be greater than any harmonic, a difference which will be further emphasized by effective low-pass filters such as servomotors. Adequate accuracy can therefore be obtained in many cases by considering only the fundamental component of the output.[1,2] Since the amplitude and phase of the fundamental component varies with the amplitude of the applied sinusoid, the approximate characteristics of the nonlinear device are represented by an "amplitude-describing function" $H_d(x)=f(x)e^{j\theta(x)}$ (see Fig. 1). If x represents the amplitude of a sinusoidal input signal, $f(x)$ is the ratio of the fundamental output to the input amplitude and $\theta(x)$ is the phase shift of the output fundamental relative to the input signal. Note that $H_d(x)$ is frequency invariant, it depends only on the input amplitude.

Once $H_d(x)$ is known, a stability analysis can proceed essentially as in the linear case. Consider the simple loop shown in Fig. 2. System stability is governed by the roots of the equation

$$1+H_d(x)H(s)=0, \tag{1}$$

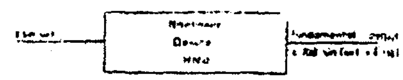

FIG. 1. Describing function of a nonlinear device.

or

$$H(s)=-1/H_d(x). \tag{2}$$

In the linear case $H_d(x)=1$ and the stability problem reduces to the conventional one, solved easily by means of a Nyquist plot. The only modification required for the nonlinear case under the assumptions stated is a change in the critical point which now becomes $-1/H_d(x)$ instead of -1. Thus the critical point changes with the signal amplitude, and it becomes necessary to plot an amplitude locus $-1/H_d(x)$ in addition to the frequency locus $H(s)$. If the amplitude locus lies completely outside the frequency locus, the system is stable under all conditions of operations.[4] Figure 3 shows intersecting loci. Here the system is unstable for small disturbances, but stable for large disturbances so

* Now with Philco Corporation, Philadelphia, Pennsylvania.
[1] R. J. Kochenburger, Elec. Eng. 69, 687 (1950). See also Trans. Am. Inst. Elec. Engrs. 69, 270 (1950).
[2] E. C. Johnson, dissertation, Massachusetts Institute of Technology, 1951; Trans. Am. Inst. Elec. Engrs. 71II, 169 (1952).
[3] E. S. Sherrard, Trans. Am. Inst. Elec. Engrs. 71II, 312 (1952).

[4] Frequently the inverse loci, $1/H(s)$ and $-H_d(x)$, are plotted. The choice is governed simply by computational convenience in particular instances.

82.

that oscillations will tend to stabilize near the intersection point P which thus specifies the steady-state conditions, at least to a first approximation.[1]

In summary, analysis of the stability problem will require the following steps.

(a) Determination of the wave form at the output of the nonlinear device resulting from a sinusoidal input.

(b) Calculation of the describing function $H_d(x)$ from the wave shape obtained in (a).

(c) Plot and interpretation of the amplitude and frequency loci for the system under consideration. For the cases of particular interest here this step requires rearrangement of the conventional block diagram in order to secure effective separation of all transfer functions into two classes: The class of all linear but frequency sensitive components and that of all nonlinear but frequency insensitive elements.

The following definitions will be used throughout this paper. Static friction is the torque required to initiate rotation. Sliding friction is the velocity-independent component of the torque necessary to maintain such motion once started. Viscous friction is that component of the torque which is linearly proportional to the angular velocity of the rotating member.

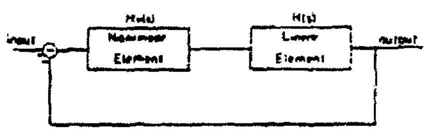

FIG. 2. Feedback loop with nonlinear element.

III. SLIDING FRICTION IN SERVOSYSTEMS

A. Wave Form Resulting from a Sinusoidal Input Torque to a System with Sliding Friction

If only sliding friction is considered the entire friction phenomenon can be represented by the characteristic curve of Fig. 4.

Consider a rotating member with moment of inertia J and angular acceleration $\ddot{\theta}$. Because of sliding friction the effective accelerating or decelerating torque τ_e is related to the applied torque τ_a through the equation

$$\tau_e = \tau_a \pm T_s, \tag{3}$$

where T_s is defined by Fig. 4. From Newton's law of motion,

$$\tau_a = T_s + J\ddot{\theta} \text{ for angular velocity } \dot{\theta} > 0 \tag{4}$$

$$\tau_a = -T_s + J\ddot{\theta} \text{ for } \dot{\theta} < 0. \tag{5}$$

From Eqs. (3), (4), and (5), the angular acceleration of the rotating member is given by

$$\ddot{\theta}(t) = \tau_e(t)/J. \tag{6}$$

Hence $\ddot{\theta}(t)$ has the same wave form as $\tau_e(t)$.

If the applied torque τ_a is sinusoidal,

$$\tau_a(t) = \Gamma_a \sin\omega t. \tag{7}$$

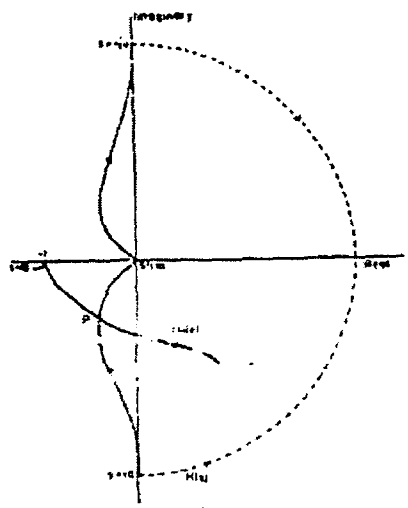

FIG. 3. Amplitude and frequency loci.

The corresponding steady-state wave forms are sketched in Figs. 5 and 6. The effective torque wave derived from Eq. (3) is shown in dotted lines. The discontinuities of the τ_e wave correspond to zeros of the $\dot{\theta}$ wave because the frictional torque T_s changes sign at those instants. On the θ curve, point P is the point of inflection, corresponding to maximum acceleration. Since the steady state is of primary interest, the reference time is chosen after the oscillation has reached its steady-state value. $\dot{\theta}(t)$ passes through zero at

$$\omega t = n\pi - \alpha, \quad n = 0, 1, 2, 3 \cdots,$$

while

$$\tau_a = T_s \quad \text{at} \quad \omega t = \alpha.$$

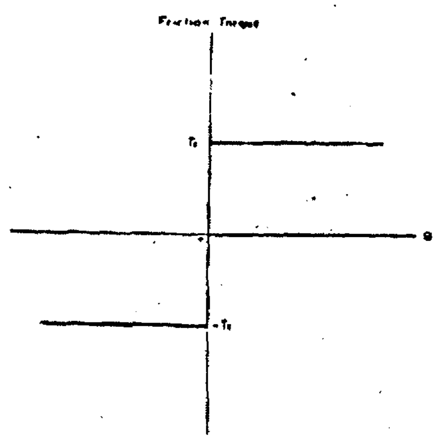

FIG. 4. Sliding friction characteristic.

G-1-4

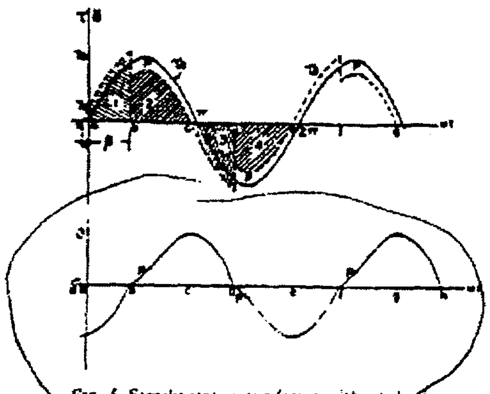

FIG. 5. Steady-state wave forms without dead zones.

It follows that

$$\alpha = \sin^{-1}\lambda, \tag{8}$$

where

$$\lambda = T_c/T_m. \tag{9}$$

Only the angle β corresponding to the first discontinuity point remains unknown. Once it has been evaluated in terms of λ, the wave form is completely determined. There are two possibilities: If $\alpha \leq \beta$, there is no dead zone in the τ_e wave (Fig. 5). If $\alpha > \beta$, there are dead zones as indicated in Fig. 6. These two cases will be considered separately.[3]

Mathematical Representation of the Steady-State Wave Forms

Case (1).—No dead zone. $\alpha \leq \beta$.

Refer to Fig. 5. In the absence of viscous friction, the following steady-state conditions exist.

Shaded area No. 1 = shaded area No. 2
= shaded area No. 3
= shaded area No. 4, etc.[4]

But between a and b,

$$\tau_e = T_m \sin\omega t + T_c$$
$$= T_m(\sin\omega t + \sin\alpha) = T_m(\sin\omega t + \lambda); \tag{10}$$

and between b and c,

$$\tau_e = T_m \sin\omega t - T_c$$
$$= T_m(\sin\omega t - \sin\alpha) = T_m(\sin\omega t - \lambda). \tag{11}$$

Then

$$\text{area No. } 1 = \int_{-\alpha}^{\beta} T_m(\sin\omega t + \sin\alpha)d(\omega t)$$
$$= T_m(-\cos\beta + \beta \sin\alpha + \cos\alpha + \alpha \sin\alpha), \tag{12}$$

and

$$\text{area No. } 2 = \int_{\beta}^{\pi-\alpha} T_m(\sin\omega t - \sin\alpha)d(\omega t)$$
$$= T_m[\cos\alpha - (\pi - \alpha)\sin\alpha + \cos\beta + \beta \sin\alpha]. \tag{13}$$

If Eqs. (12) and (13) are set equal and simplified, the result is

$$\cos\beta = \pi \sin\alpha/2$$

or

$$\beta = \cos^{-1}(\pi\lambda/2). \tag{14}$$

For the extreme case, $\beta = \alpha$, Eq. (14) becomes

$$\sin^{-1}\lambda = \cos^{-1}(\pi\lambda/2)$$

or

$$\lambda^2 + (\pi\lambda/2)^2 = 1.$$

A solution for λ yields

$$\lambda = \lambda_c = 0.536. \tag{15}$$

λ_c is the critical value of the quantity T_c/T_m. There is no dead zone for $\lambda \leq \lambda_c$, and there are dead zones for $\lambda > \lambda_c$.

Case (2).—With dead zones. $\alpha > \beta$.

In like manner, one obtains from Fig. 6:

shaded area No. 1 = shaded area No. 2
= shaded area No. 3
= shaded area No. 4, etc.

But, between a and b,

$$\tau_e = T_m(\sin\omega t + \lambda), \tag{16}$$

between b and c,

$$\tau_e = 0; \tag{17}$$

and between c and d,

$$\tau_e = T_m(\sin\omega t - \lambda). \tag{18}$$

Hence,

$$\text{area No. } 1 = \int_{-\alpha}^{\beta} T_m(\sin\omega t + \lambda)d(\omega t)$$
$$= T_m(-\cos\beta + \beta\lambda + \cos\alpha + \alpha\lambda), \tag{19}$$

$$\text{area No. } 2 = \int_{\alpha}^{\pi-\alpha} T_m(\sin\omega t - \lambda)d(\omega t)$$
$$= T_m[\cos\alpha - (\pi - \alpha)\lambda + \cos\alpha + \alpha\lambda]. \tag{20}$$

Equating (19) and (20) and simplifying

$$\lambda\beta - \cos\beta = (1 - \lambda^2)^{\frac{1}{2}} - (\pi - \sin^{-1}\lambda)\lambda. \tag{21}$$

For the extreme case $\alpha = \beta = \sin^{-1}\lambda$,

$$\lambda \sin^{-1}\lambda - (1 - \lambda^2)^{\frac{1}{2}} = (1 - \lambda^2)^{\frac{1}{2}} - (\pi - \sin^{-1}\lambda)\lambda,$$

or

$$\lambda = \lambda_c = 0.536 \text{ as before.} \tag{15}$$

[3] Note that a dead zone or region of zero effective torque and velocity such as bc on Fig. 6 occurs whenever the applied torque is smaller in magnitude than T_c at the instant when the velocity reaches zero.

[4] Velocity is proportional to the integral of torque in the absence of viscous friction.

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U-1-5

B. Calculation of the Describing Function

Since $\tau_e(t)$ is a periodic function of time, it can be expressed in terms of a Fourier series:

$$\tau_e(t)=\frac{b_0}{2}+\sum_{n=1}^{\infty}(a_n \sin n\omega t+b_n \cos n\omega t). \qquad (22)$$

It has been pointed out that, to a first approximation, $\tau_e(t)$ can be represented by the fundamental component of its Fourier series. From symmetry considerations, $b_0=0$. This implies oscillation about the rest position which is a condition of primary interest in stability analyses.[7] Then

$$\tau_e(t)=a_1 \sin\omega t+b_1 \cos\omega t, \qquad (23)$$

where

$$a_1=\frac{1}{\pi}\int_{\theta}^{2\pi+\theta}\tau_e(t)\sin(\omega t)d(\omega t) \qquad (24)$$

and

$$b_1=\frac{1}{\pi}\int_{\theta}^{2\pi+\theta}\tau_e(t)\cos(\omega t)d(\omega t). \qquad (25)$$

Evaluation of the Fourier Coefficients

Case (1).—No dead zone, $\lambda\leq\lambda_c$ or $\alpha\leq\beta$.

$$a_1=\frac{2}{\pi}\int_{-\alpha}^{\beta}T_s(\sin\omega t+\lambda)\sin(\omega t)d(\omega t)$$
$$+\frac{2}{\pi}\int_{\beta}^{\pi-\alpha}T_s(\sin\omega t-\lambda)\sin(\omega t)d(\omega t)$$
$$=T_s(1-2\lambda^2). \qquad (26)$$

Similarly

$$b_1=2T_s\lambda[(2/\pi)^2-\lambda^2]^{\frac{1}{2}}. \qquad (27)$$

Equation (23) may also be written in the form

$$\tau_e(t)=C_1\sin(\omega t+\delta), \qquad (28)$$

where

$$C_1=(a_1^2+b_1^2)^{\frac{1}{2}}=T_s\left[1-4\left(1-\frac{4}{\pi^2}\right)\lambda^2\right]^{\frac{1}{2}} \qquad (29)$$

and

$$\delta=\tan^{-1}\frac{b_1}{a_1}=\tan^{-1}\frac{2\lambda[(2/\pi)^2-\lambda^2]^{\frac{1}{2}}}{1-2\lambda^2}. \qquad (30)$$

Hence, with an applied torque $\tau_a(t)=T_a\sin\omega t$, the

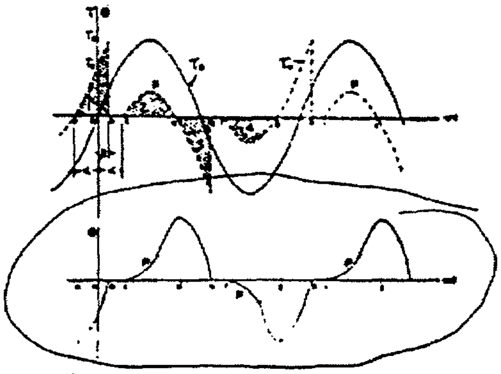


FIG. 6. Steady-state wave forms with dead zones.

effective torque is

$$\tau_e(t)=T_s\left[1-4\left(1-\frac{4}{\pi^2}\right)\right]^{\frac{1}{2}}$$
$$\times\sin\left[\omega t+\tan^{-1}\frac{2\lambda[(2/\pi)^2-\lambda^2]^{\frac{1}{2}}}{1-2\lambda^2}\right]. \qquad (31)$$

In accordance with the definition of Sec. II, the describing function for the sliding-friction element is

$$H_s(\lambda)=f(\lambda)\angle\delta(\lambda)$$
$$=\left[1-4\left(1-\frac{4}{\pi^2}\right)\lambda^2\right]^{\frac{1}{2}}\angle\tan^{-1}\frac{2\lambda[(2/\pi)^2-\lambda^2]^{\frac{1}{2}}}{1-2\lambda^2}. \qquad (32)$$

Case (2).—With dead zones, $\lambda>\lambda_c$ or $\alpha>\beta$.

$$a_1=\frac{1}{\pi}\int_{\alpha}^{\pi+\beta}T_s(\sin\omega t-\lambda)\sin(\omega t)d(\omega t)$$
$$+\frac{1}{\pi}\int_{\pi+\alpha}^{2\pi+\beta}T_s(\sin\omega t+\lambda)\sin(\omega t)d(\omega t)$$
$$=(T_s/\pi)[\pi-(\alpha-\beta)-\sin\alpha(\cos\alpha+\cos\beta)$$
$$-\cos\beta(\sin\alpha+\sin\beta)]. \qquad (33)$$

Similarly

$$b_1=(T_s/\pi)(\sin\alpha+\sin\beta)^2. \qquad (34)$$

Hence in complete analogy with case (1) the friction-describing function is given by the expression

$$H_s(\lambda)=f(\lambda)\angle\delta(\lambda), \qquad (35)$$

where

$$f(\lambda)=\frac{1}{\pi}\{[\pi-(\alpha-\beta)-\sin\alpha(\cos\alpha+\cos\beta)-\cos\beta(\sin\alpha+\sin\beta)]^2+[(\sin\alpha+\sin\beta)^2]^2\}^{\frac{1}{2}}$$

and

$$\delta(\lambda)=\tan^{-1}\frac{(\sin\alpha+\sin\beta)^2}{\pi-(\alpha-\beta)-\sin\alpha(\cos\alpha+\cos\beta)-\cos\beta(\sin\alpha+\sin\beta)}.$$

[7] Extensions to nonzero means are possible but complicate the analysis appreciably. See reference 1.

B5

10.0 PROBLEM 7 - SIMULATION OF FRICTION BEHAVIOR

10.1 Problem

A mass rests on a frictional surface which permits a frictional resistance force ± R. The mass is driven by an external sinusoidal force. Figure 9.1 applies to this case if we set $x_0 \rightarrow \infty$, $F_s = 0$, $C_1 = 0$, and $F(t) = \beta \sin wt$.

10.2 Purpose of Problem

This problem illustrates the phenomena of "dead bands" in the response. Dead bands are regions of time when a moving mass, subject to an oscillating force, stops for a finite period of time. It provides a very severe test of the numerical simulation of frictional behavior. The phenomena could be expected to occur in the fuel rack analysis since the seismic load provides a reversing motion, and the pedestals rest on a frictional surface. A successful validation demonstrates that the DYNARACK algorithm, based on a high, but finite, frictional stiffness, is capable of reproducing the theoretical response.

10.3 Comparison Solution

The exact solution to this problem is provided in [10.1]. Tou and Schutheiss have given solutions for this situation. The interesting features of the motion are that if $R/\beta < .536$, the motion is roughly sinusoidal, but has discontinuities in acceleration. If $R/\beta > .536$, then the motion is sporadic, there being so-called dead bands within which no motion occurs. When $R/\beta > 1$, no motion is possible except for an initial transient. Appendix G-1 is a complete copy of the reference.

10.4 DYNARACK Solution

The equation of motion is

$$m \frac{d^2 x}{dt^2} = \beta \sin wt \pm R$$

We simulate the event for $m = \beta = 1$, and run three cases for $R/\beta = \lambda =$ .3, .7, and 1.01. Input data files are given in Appendix G-2. The friction spring constant (Figure 9.1) is set at $K_f = 1 \times 10^7$ lb./in. to simulate an "infinite" slope.

To construct the velocity versus time results, DYNARACK internally archives velocity at user specified time steps. These velocity time files can then be graphed. Input files and data files for the graphs are given in Appendix G-2 for the results plotted in 10.3.

10.5 Results

Figures 10.1 - 10.3 show the results for the three values of λ. It is clearly evident that DYNARACK is capable of reproducing the expected phenomena. In Figure 10.3, the small non-zero velocity components subsequent to the initial transient are due to the presence of the finite spring constant K_f. Comparison with the work of Ref. [10.1] shows excellent agreement and we conclude that the frictional representation in DYNARACK is validated.

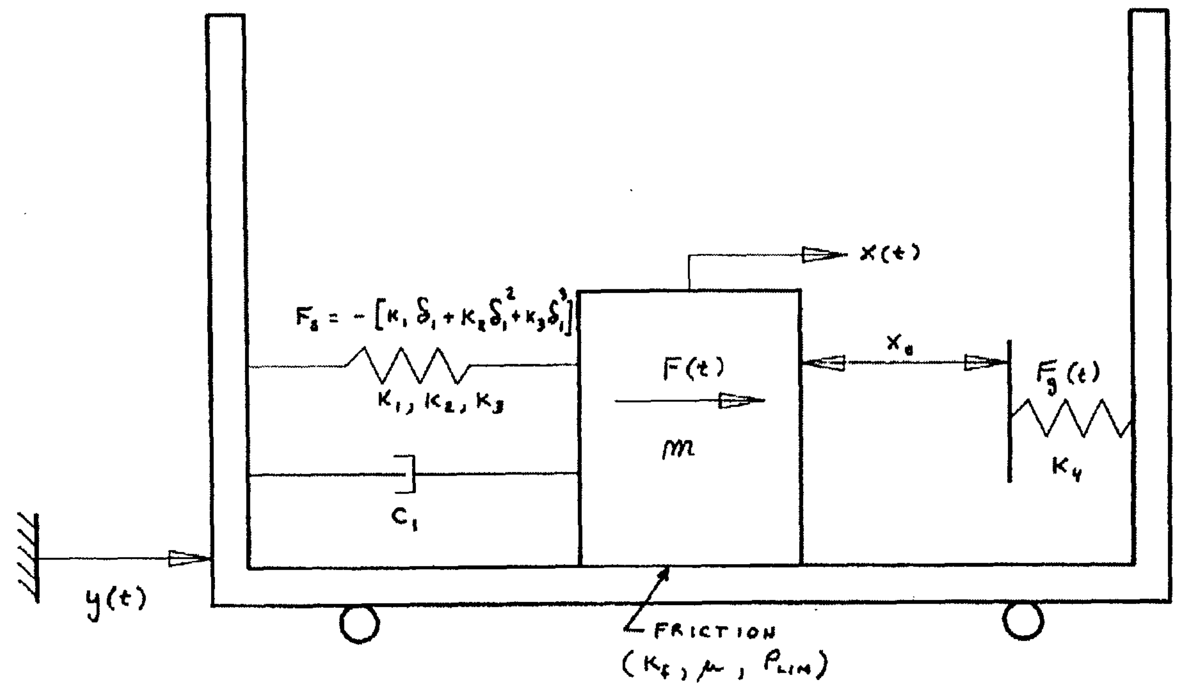
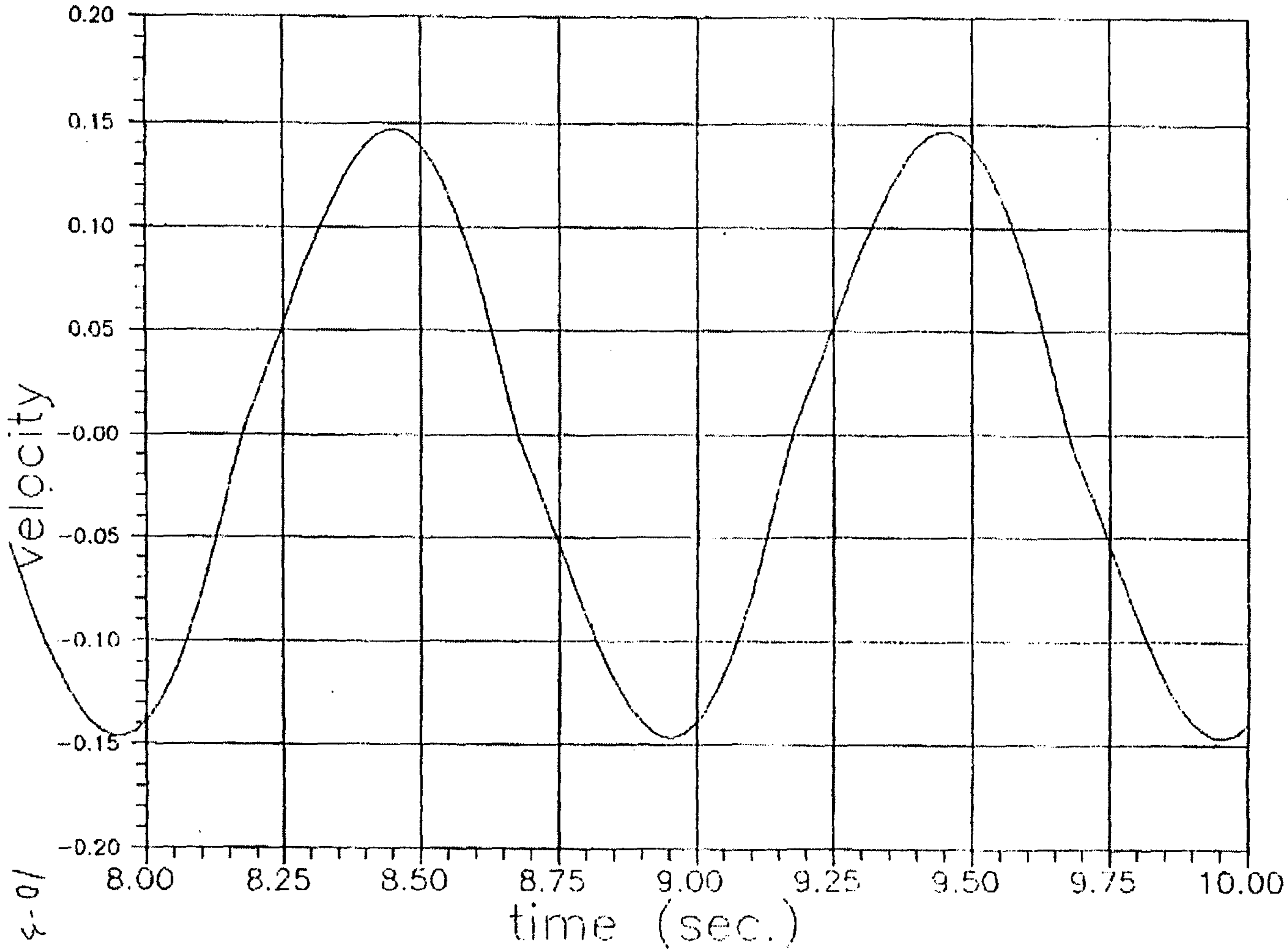


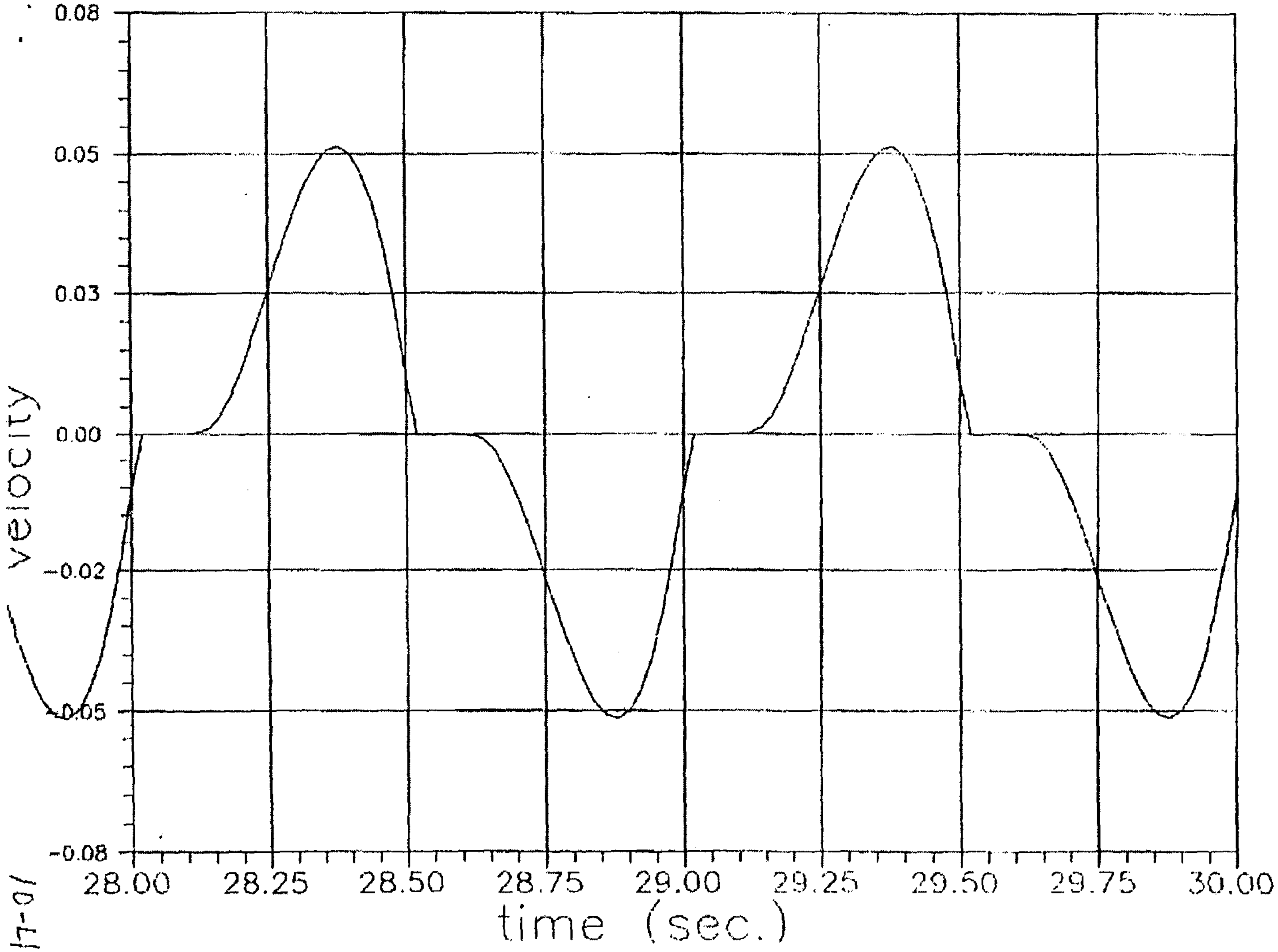
Figure 9.1 One Degree-of-Freedom Model

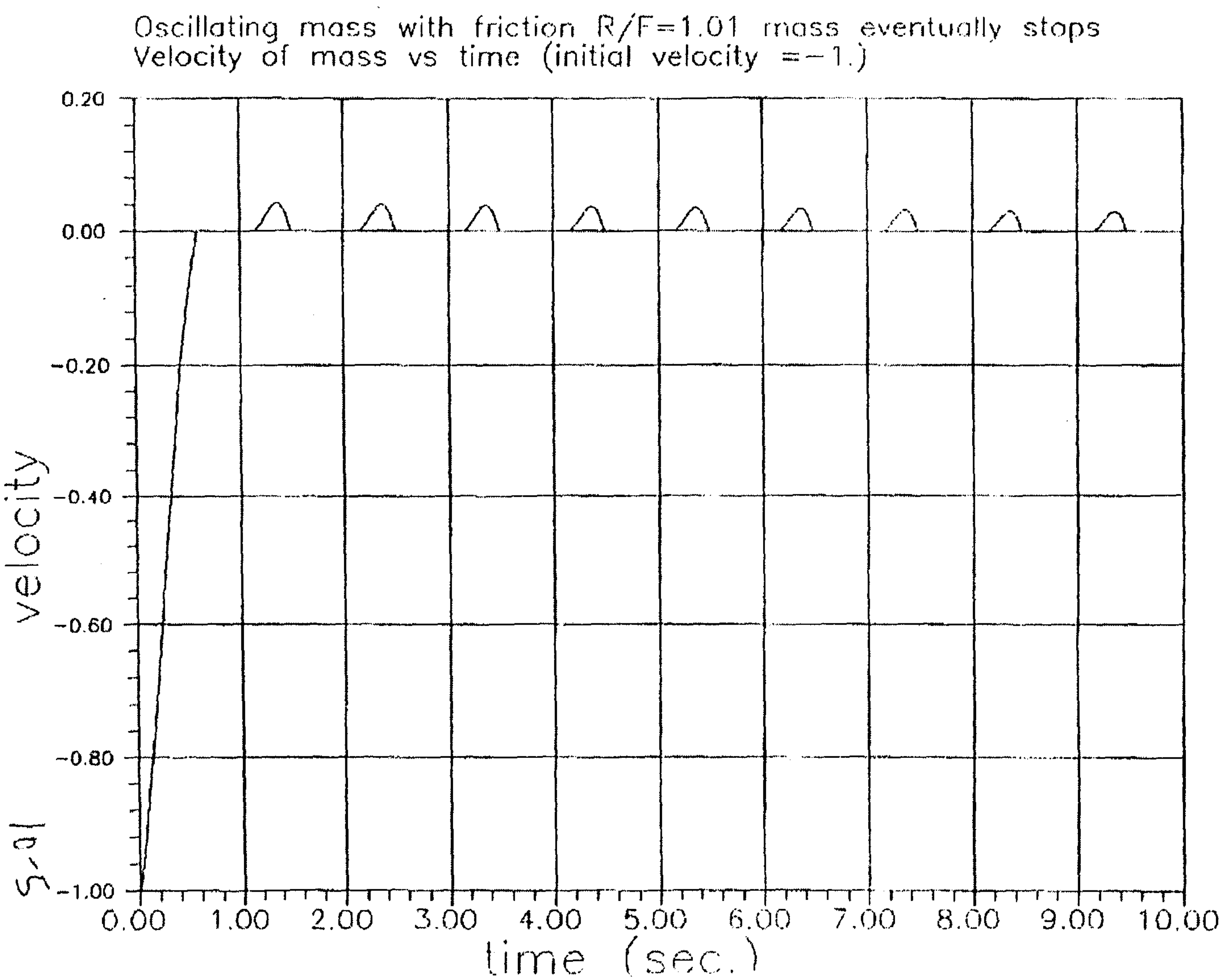
Oscillating mass with friction R/F=.3 No Dead Bands
Velocity of mass vs time (initial velocity =0.)



10-3

Oscillating mass with friction R/F=.7 Dead Bands Present
Velocity of mass vs time (initial velocity =0.)





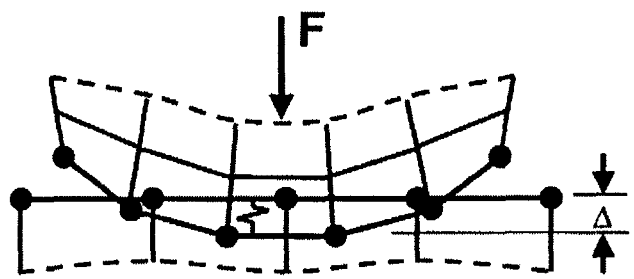
Oscillating mass with friction R/F=1.01 mass eventually stops
Velocity of mass vs time (initial velocity =-1.)
0.20
0.00
-0.20
-0.40
-0.60
-0.80
-1.00
velocity
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
time (sec.)

IC-5

A. Basic Concepts

Review:

- Recall that all ANSYS contact elements use a penalty stiffness (contact stiffness) to help enforce compatibility at the contact interface.



The contact spring will deflect an amount Δ, such that equilibrium is satisfied:

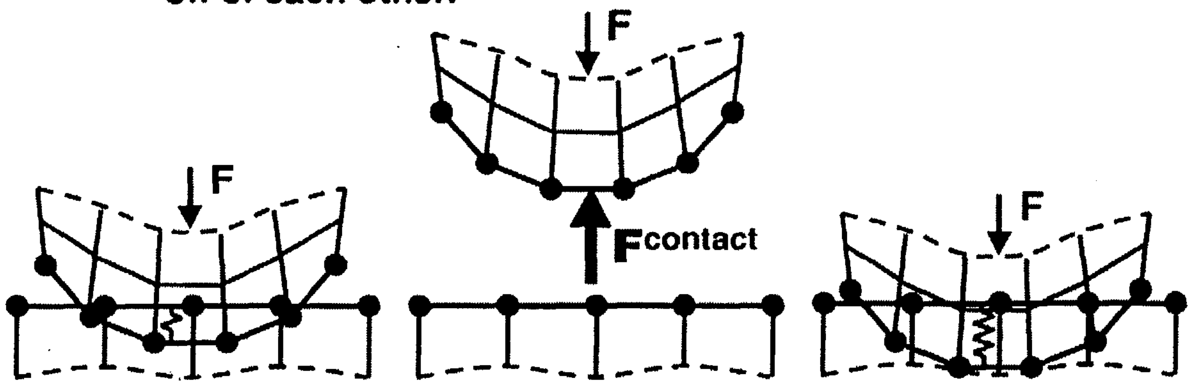
$$F = k\,\Delta$$

where k is the contact stiffness.

- Some finite amount of penetration, $\Delta > 0$, is required mathematically to maintain equilibrium.
- However, physical contacting bodies do not interpenetrate ($\Delta = 0$).

... Basic Concepts

- As an analyst, you face a dilemma:
  - Minimum penetration gives best accuracy.
    - Therefore, the contact stiffness should be very great.
  - However, too stiff a value causes convergence difficulties.
    - The model can oscillate, with contacting surfaces bouncing off of each other.



Iteration *n* Iteration *n+1* Iteration *n+2*

... Determining a value

- As a practical matter, a good first trial value for bulky contact stiffness would be $k_{contact} = f_{bulk} \times k_{Hertz}$, where f_{bulk} is a factor usually between 0.1 and 10 for bulky solids.
  - Because the starting estimated value of f_{bulk} ranges over at least two orders of magnitude, and because $k_{contact}$ will be adjusted by trial-and-error anyway, it is usually not justifiable to worry about the element's size when estimating the penalty stiffness.
- *For bulky solids*, simply estimate the penalty stiffness by

$$k = f_{bulk} \times E$$

  - where the factor f_{bulk} is usually between 0.1 and 10, and a good starting value for f_{bulk} is often $f_{bulk} = 1.0$.
  - This estimate assumes an approximate "unit" element size; for very large or very small elements, you might need to adjust the starting value of f_{bulk} accordingly.

... Determining a value

- Determining a good stiffness value usually requires some experimentation. The following procedure may be used as a guideline:
  - 1. Use a low value of stiffness to start.
  - 2. Run the analysis to a fraction of the final load.
  - 3. Check the penetration and number of equilibrium iterations used in each substep.
    - As a rough, quick check, if you can visually detect penetration in a true-scale displaced plot of the entire model, the penetration is probably excessive. Increase the stiffness and restart.
    - If many iterations are needed for convergence (or if convergence is never achieved), reduce the stiffness and restart.
  - *Note: Penalty stiffness can be modified from one load step to another, and can be adjusted in a restart.*

VM73: Free Vibration with Coulomb Damping

ANSYS

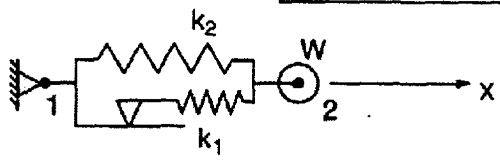
Reference: Tse (Ref. 12), Page 175, Case 1
Analysis Type(s): Full transient dynamic analysis (ANTYPE=4)
Element Type(s): Combination elements (COMBIN40)

Test Case

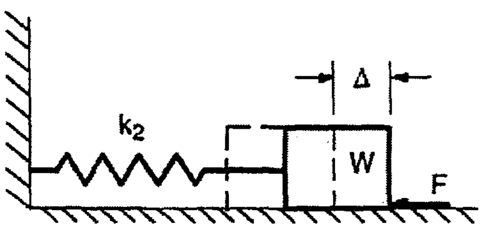
A spring–mass system with coulomb damping is displaced a distance Δ and released. Dry friction is assumed to act as a limiting sliding force F between the sliding mass and the surface. Determine the displacement u at various times t.

Material Properties
$W = 10$ lb
$k_2 = 30$ lb/in
$m = W/g$

Loading
$\Delta = -1$ in
$F = 1.875$ lb



Representative Finite Element Model



Problem Sketch

	X	$\dot{X}$
t = 0	–1.	0.

Initial Conditions

Analysis Assumptions and Modeling Notes

One combination element is used with the slider in parallel with the spring. The slider spring constant (k_1 = 10,000 lb/in) is arbitrarily selected high enough to minimize the elastic contact effect but low enough to also allow a practical integration time step size. The integration time step (0.2025/405 = 0.0005 sec) is based on $\simeq$1/Nf where N=20 and f is the system natural frequency. At release, the mass acceleration is not necessarily zero. Therefore, a load step with a small time period is used to ramp up to the appropriate acceleration while maintaining an essentially zero velocity. The final time of 0.2025 sec allows one full cycle of motion. POST26 is used to postprocess results from the solution phase.

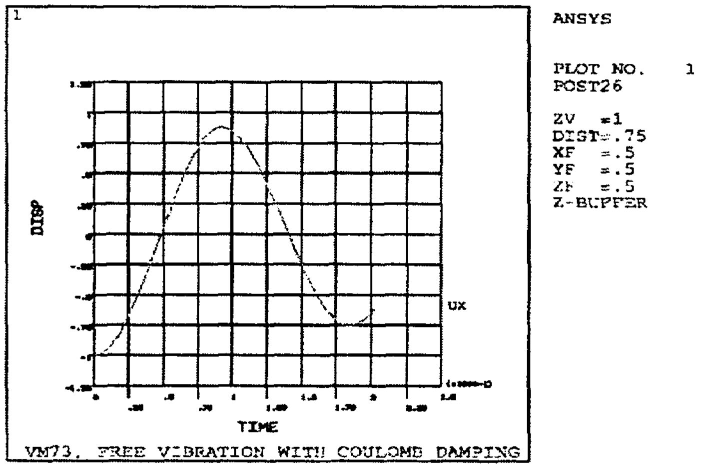
VM73: Free Vibration with Coulomb Damping (continued)

ANSYS

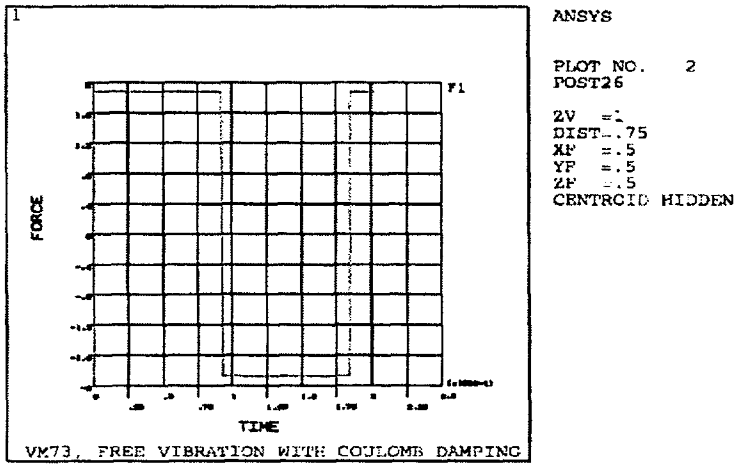
Results Comparison

	Target	ANSYS	Ratio
u, in (t = 0.09 sec)	0.87208	0.87205	1.000
u, in (t = 0.102 sec)	0.83132	0.83018	0.999
u, in (t = 0.183 sec)	–0.74874	–0.74875	1.000

VM73: Free Vibration with Coulomb Damping (continued) ANSYS



VM73 – Displacement vs. Time Display



VM73 – Sliding Force vs. Time Display

HOLTEC INTERNATIONAL
CALCULATION SHEET

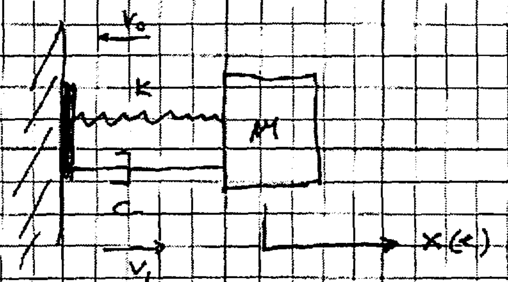
Page F-1 Rev.

Project No. Report No. Other

Prepared By Date Reviewed By Date

COEFFICIENT OF RESTITUTION AND LINEAR VISCOUS DAMPING

Consider a 1-DOF mass-spring-damper after impact with a fixed target



The equation of motion is:

$$M\ddot{X} + C\dot{X} + KX = 0$$

$$X(0) = 0 \qquad \dot{X}(0) = -V_0$$

If we define ω_0, ζ by the relation

$$\omega_0 = \sqrt{K/M} \qquad 2\zeta\omega_0 = \frac{C}{M}$$

then the equation of motion is

$$\ddot{X} + 2\zeta\omega_0\dot{X} + \omega_0^2 X = 0 \qquad (1)$$

Per C. Smith, Applied Mechanics - More Dynamics, John Wiley, 1976, pp 187-189, the solution to eq (1), subject to the prescribed initial conditions is:

$$X(t) = -\frac{V_0}{\omega} e^{-\xi\omega_0 t} \sin\omega t \qquad (2)$$

The mass leaves the target at time, t_f, when

$$X(t_f) = 0 = -\frac{V_0}{\omega} e^{-\xi\omega_0 t_f} \sin\omega t_f$$

or, when

$$\omega t_f = \pi = \omega_0 (1-\xi^2)^{1/2} t_f \qquad (3)$$

During the time $0 \le t \le t_f$ when the mass impacts the target, compresses the spring and damper, and reverses direction to return to the starting point, the velocity of the mass is

$$\dot{X}(t) = -\frac{V_0}{\omega}\left[-\xi\omega_0 e^{-\xi\omega_0 t} \sin\omega t + \omega e^{-\xi\omega_0 t} \cos\omega t\right]$$

so that at time, t_f, when $\omega t_f = \pi$

$$\dot{X}(t_f) = V_1 = V_0 e^{-\xi\omega_0 t_f} \qquad (4)$$

Since $\frac{V_1}{V_0} = e$ = coefficient of restitution, and

$$\omega_0 = \frac{\omega}{\sqrt{1-\xi^2}}$$

Then

$$\boxed{e = e^{-\frac{\xi\pi}{(1-\xi^2)^{1/2}}}} \qquad (5)$$

The above equation is plotted in the following

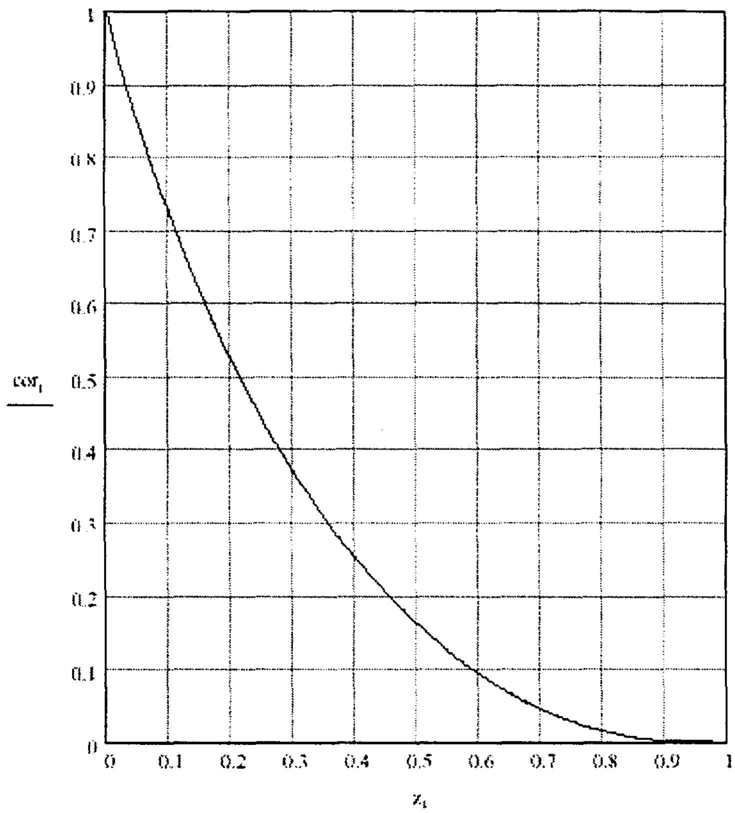
$i := 1 .. 40$

$$z_i := \frac{(i-1)}{40}$$

$$g_i := \frac{-z_i}{\left[1-(z_i)^2\right]^{.5}} \cdot \pi \qquad cor_i := e^{g_i}$$

$i =$	$cor_i =$	$z_i =$
1	1	0
2	0.924	0.025
3	0.854	0.05
4	0.79	0.075
5	0.729	0.1
6	0.673	0.125
7	0.621	0.15
8	0.572	0.175
9	0.527	0.2
10	0.484	0.225
11	0.444	0.25
12	0.407	0.275
13	0.372	0.3
14	0.34	0.325
15	0.309	0.35
16	0.281	0.375
17	0.254	0.4
18	0.229	0.425
19	0.205	0.45
20	0.183	0.475
21	0.163	0.5

Project 90428



STONE & WEBSTER, INC.

5010.64

CALCULATION SHEET

CLIENT & PROJECT	PAGE 1 OF 115
PRIVATE FUEL STORAGE, LLC – PFSF	+ 22 pp of ATTACHMENTS

CALCULATION TITLE	QA CATEGORY (✓)
STABILITY ANALYSES OF CASK STORAGE PADS	☑ I NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ (other)

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

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

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Geotechnical	PJTrudeau – Stoughton/3	☑			

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STONE & WEBSTER, INC.

CALCULATION SHEET

CLIENT & PROJECT	PAGE 2
PRIVATE FUEL STORAGE, LLC – PFSF	
CALCULATION TITLE: **STABILITY ANALYSES OF CASK STORAGE PADS**	QA CATEGORY (✓) ☑ I NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ (other)

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

APPROVALS - SIGNATURE & DATE			REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED ☑	
PREPARER(S)/DATE(S)	REVIEWER(S)/DATES(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
Original Signed By: PJTrudeau / 5-31-01	Original Signed By: TYChang / 5-31-01	Original Signed By: TYChang / 5-31-01	8	7		✓
PJTrudeau / 7-26-01 Paul J. Trudeau	TYChang / 7-26-01 Thomas Y. Chang	TYChang / 7-26-01 Thomas Y. Chang	9	8		✓

DISTRIBUTION

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Geotechnical	PJTrudeau – Stoughton/3	☑			

TABLE OF CONTENTS

RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1

Revision 1 was prepared to incorporate the following:

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

REVISION 2

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

REVISION 3

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

REVISION 4

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

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

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

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

Modified/updated conclusions.

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

Remaining pages prepared by DLAloysius and reviewed by SYBoakye.

REVISION 5

Major re-write of the calculation.

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

REVISION 6

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

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

REVISION 7

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

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

REVISION 8

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

REVISION 9

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

OBJECTIVE OF CALCULATION

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

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

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

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

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

ASSUMPTIONS/DATA

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

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

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

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

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

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

GEOTECHNICAL PROPERTIES

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

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

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

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

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

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

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

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

The results of these tests are presented in Attachment 7 of the Appendix 2A of the SAR and they are plotted in Figure 7 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C). Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, in the sliding stability analyses of the cask storage pads, included below, the shear strength of the silty clay/clayey silt equals the shear strength measured in these direct shear tests for a normal stress equal to the vertical stress under the fully loaded cask storage pads prior to imposition of the dynamic loading due to the earthquake. As shown in Figure 7 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C), this shear strength is 2.1 ksf and the friction angle is set equal to 0°.

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

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

Case IA Static using undrained strength: [illegible] = 0° & c = 2.2 ksf.

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

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

METHOD OF ANALYSIS

DESCRIPTION OF LOAD CASES

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

The following load combinations are analyzed:

Case I Static

Case II Static + dynamic horizontal forces due to the earthquake

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

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

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

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

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

OVERTURNING STABILITY OF THE CASK STORAGE PADS

The factor of safety against overturning is defined as:

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

The resisting moment is calculated as the resultant weight of the pad and casks x the distance from one edge of the pad to the center of the pad in the direction of the minimum width. The weight of the pad is calculated as 3 ft x 67 ft x 30 ft x 0.15 kips/ft³ = 904.5 K, and the weight of 8 casks is 8 x 356.5 K/cask = 2,852 K. The moment arm for the resisting moment equals ½ of 30 ft, or 15 ft. Therefore,

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

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

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

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

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

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

SLIDING STABILITY OF THE CASK STORAGE PADS

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

FS = resisting force ÷ driving force

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

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

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

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

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

B = 30 feet

L = 67 feet

DESIGN ISSUES RELATED TO SLIDING STABILITY OF THE CASK STORAGE PADS

Figure 3 presents a detail of the soil cement under and adjacent to the cask storage pads. Figure 8 presents an elevation view, looking east, that is annotated to facilitate discussion of potential sliding failure planes. The points referred to in the following discussion are shown on Figure 8.

1. Ignoring horizontal resistance to sliding due to passive pressures acting on the sides of the pad (i.e., Line AB or DC in Figure 8), the shear strength must be at least 1.60 ksf (11.10 psi) at the base of the cask storage pad (Line BC) to obtain the required minimum factor of safety against sliding of 1.1.
2. The static, undrained strength of the clayey soils exceeds 2.1 ksf (14.58 psi). This shear strength, acting only on the base of the pad, provides a factor of safety of 1.27 against sliding along the base (Line BC). This shear strength, therefore, is sufficient to resist sliding of the pads if the full strength can be engaged to resist sliding.
3. Ordinarily a foundation key would be used to ensure that the full strength of the soils beneath a foundation are engaged to resist sliding. However, the hypothetical cask tipover analysis imposes limitations on the thickness and stiffness of the concrete pad that preclude addition of a foundation key to ensure that the full strength of the underlying soils is engaged to resist sliding.
4. PFS will use a layer of soil cement beneath the pads (Area HITS) as an "engineered mechanism" to bond the pads to the underlying clayey soils.
5. The hypothetical cask tipover analysis imposes limitations on the stiffness of the materials underlying the pad. The thickness of the soil cement beneath the pads is limited to 2 ft and the static modulus of elasticity is limited to 75,000 psi.

6. The modulus of elasticity of the soil cement is directly related to its strength; therefore, its strength must be limited to values that will satisfy the modulus requirement. This criterion limits the unconfined compressive strength of the soil cement beneath the pads to 100 psi.

7. Therefore, the pads will be constructed on a layer of soil cement that is at least 1-ft thick, but no thicker than 2-ft, that extends over the entire pad emplacement area, as delineated by Area HITS.

8. The unconfined compressive strength of the soil cement beneath the pads is designed to provide sufficient shear strength to ensure that the bond between the concrete comprising the cask storage pad and the top of the soil cement (Line BC) and the bond between the soil cement and the underlying clayey soils (Line JK) will exceed the full, static, undrained strength of those soils. To ensure ample margin over the minimum shear strength required to obtain a factor of safety of 1.1, the unconfined compressive strength of the soil cement beneath the pads (Area HITS) will be at least 40 psi.

9. DeGroot (1976) indicates that this bond strength can be easily obtained between layers of soil cement, based on nearly 300 laboratory direct shear tests that he performed to determine the effect of numerous variables on the bond between layers of soil cement.

10. Soil cement also will be placed between the cask storage pads, above the base of the pads, in the areas labeled FGBM and NCQP. This soil cement is NOT required to resist sliding of the pads, because there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement (Line BC) and between that soil-cement layer and the underlying clayey soils (Line JK) that the factor of safety against sliding exceeds the minimum required value.

11. The pads are being surrounded with soil cement so that PFS can effectively use the eolian silt found at the site to provide an adequate subbase for support of the cask transporter, as well as to provide additional margin against any potential sliding.

12. The actual unconfined compressive strength and mix requirements for the soil cement around the cask storage pads will be based on the results of standard soil-cement laboratory tests.

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

The analysis presented on the following pages demonstrates that the static, undrained strength of the in situ clayey soils is sufficient to preclude sliding (FS = 1.27 vs minimum required value of 1.1), provided that the full strength of the clayey soils is engaged. The soil-cement layer beneath the pads provides an "engineered mechanism" to ensure that the full, static, undrained strength of the clayey soils is engaged in resisting sliding forces. It also demonstrates that the bond between this soil-cement layer and the base of the concrete pad will be stronger than the static, undrained strength of the in situ clayey soils and, thus, the interface between the in situ soils and the bottom of the soil-cement layer is the weakest link in the system. Since this "weakest link" has an adequate factor of safety against sliding, the overlying interface between the soil cement and the base of the pad will have a greater factor of safety against sliding. Therefore, the factor of safety against sliding of the overall cask storage pad design is at least 1.27.

Sliding Stability at Interface Between In Situ Clayey Soils and Bottom of Soil Cement Beneath the Pads

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

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

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

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

ACTIVE EARTH PRESSURE

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

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

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

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

DYNAMIC EARTH PRESSURE

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

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

where :

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

β = slope of ground behind wall,
α = slope of back of wall to vertical,
α_H = horizontal seismic coefficient, where a positive value corresponds to a horizontal inertial force directed toward the wall,
α_V = vertical seismic coefficient, where a positive value corresponds to a vertical inertial force directed upward,
δ = angle of wall friction,
ϕ = friction angle of the soil,
g = acceleration due to gravity.

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

$P_{AE} = \frac{1}{2}\gamma H^2 K_{AE}$, where :
γ = unit weight of soil,
H = wall height, and
K_{AE} is calculated as shown above.

Sliding Stability at Interface Between In Situ Clayey Soils and Bottom of Soil Cement Beneath the Pads

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

$\beta = \alpha = 0$

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

$\phi = 40^\circ$

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

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

With these approximations,

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

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

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

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

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

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

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

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

WEIGHTS

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

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

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

EARTHQUAKE ACCELERATIONS – PSHA 2,000-YR RETURN PERIOD

a_H = horizontal earthquake acceleration = 0.711g

a_V = vertical earthquake acceleration = 0.695g

CASK EARTHQUAKE LOADINGS

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

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

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

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

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

Note:

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

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

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

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

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

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

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

The driving force, V, is defined as:

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

The factor of safety against sliding is calculated as follows:

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

For this analysis, the value of the horizontal driving force due to the earthquake, EQhc, is limited to the upper-bound value of the coefficient of friction, $\mu = 0.8$, x the cask normal load, because if EQhc exceeds this value, the cask will slide. The factor of safety exceeds the minimum allowable value of 1.1; therefore the pads plus 2-ft block of soil cement beneath them are stable with respect to sliding for this load case. The factor of safety against sliding is higher than this if the lower-bound value of μ is used (= 0.2), because the driving forces due to the casks would be reduced.

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

When the earthquake forces act in the downward direction:

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

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

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

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

The driving force, V, is defined as:

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

The factor of safety against sliding is calculated as follows:

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

The factor of safety against sliding is higher than this if the lower-bound value of μ is used (= 0.2), because the driving forces due to the casks would be reduced.

Ignoring the passive resistance acting on the sides of the pad, the resistance to sliding is the same in both directions; therefore, for this analysis, the larger value of EQhc (i.e., acting in the E-W direction) was used. Even with these conservative assumptions, the factor of safety exceeds the minimum allowable value of 1.1; therefore the pads overlying 2 ft of soil cement are stable with respect to sliding for this load case, assuming the strength of the cement-treated soils underlying the pad is at least as high as the undrained strength of the underlying soils.

MINIMUM SHEAR STRENGTH REQUIRED AT THE BASE OF THE PADS TO PROVIDE A FACTOR OF SAFETY OF 1.1

The minimum shear strength required at the base of the pads to provide a factor of safety of 1.1 is calculated as follows:

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

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

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

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

ADHESION BETWEEN THE BASE OF PAD AND UNDERLYING CLAYEY SOILS

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

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

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

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

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

DeGroot (1976) indicates that this bond strength can be easily obtained between layers of soil cement. He performed nearly 300 laboratory direct shear tests to determine the effect of numerous variables on the bond between layers of soil cement. These variables included the length of time between placement of successive layers of soil cement, the frequency of watering while curing soil cement, the surface moisture condition prior to construction of the next lift, the surface texture prior to construction of the next lift, and various surface treatments and additives. His results demonstrated that, with the exception of treating the surface of the lifts with asphalt emulsion, asphalt cutback, and chlorinated rubber compounds, the bond strength nearly always exceeded 11.10 psi, the minimum required value of shear strength of the bond between the base of the pads and the underlying material. The minimum bond strength he reports, other than for the asphalt and chlorinated rubber surface treatments identified above, is 7.7 psi. This value applied for only one test (Sample No. 15R-149, Series No. 3, Spec. No. 12) that was performed on a sample that had no special surface treatment along the lift line. This test, however, was anomalous, since all of the other specimens in this series had bond strengths in excess of 38.5 psi. He reports that nearly all of the specimens that used a cement surface treatment broke along planes other than along the lift lines, indicating that the bond between the layers of soil cement was stronger than the remainder of the specimens. Excluding the specimens that did not use the cement surface treatment, the minimum bond strength was 47.7 psi, which greatly exceeds the bond strength (11.10 psi) required to obtain an adequate factor of safety against sliding of the pads without including the passive resistance acting on the sides of the pads.

DeGroot reached the following conclusions:

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

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

DeGroot's direct shear test results demonstrate that the specimens having a cement surface treatment all had bond strengths that ranged from 47.7 psi to 198.5 psi, with the average bond strength of 132.5 psi. Even the minimum value of this range greatly exceeds the bond strength (11.10 psi) required to obtain a factor of safety against sliding of 1.1, conservatively ignoring the passive resistance available on the sides of the pads. Therefore, when required due to unavoidable time delays, the techniques DeGroot describes for enhancing bond strength will be used between the top of the soil cement and succeeding lifts or between the top of the soil cement and the concrete cask storage pads, to assure that the bond at the interfaces are greater than the minimum required value. These techniques will include roughening and cleaning the surface of the underlying soil cement, proper moisture conditioning, and using a cement surface treatment.

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

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

LIMITATION OF STRENGTH OF SOIL CEMENT BENEATH THE PADS

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

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

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

Where E = Young's modulus

q_u = Unconfined compressive strength

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

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

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

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

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

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

$$\mathbf{FS_{Pad\ to\ Soil\ Cement}} = \overset{T}{5{,}789\ K} \div \underbrace{(\overset{F_{AE\ E\text{-}W}}{65.3\ K} + \overset{EQhp}{643\ K} + \overset{EQhc_{E\text{-}W}}{2{,}212\ K})}_{(2{,}920.3\ K)} = \mathbf{1.98}$$

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

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

Soil Cement Above the Base of the Pads

Soil cement also will be placed between the cask storage pads, above the base of the pads. Earlier versions of this calculation demonstrated that this soil cement could be designed such that its compressive strength alone would be sufficient to resist all of the sliding forces due to the design earthquake. However, as shown above, this soil cement is NOT required to resist sliding of the pads, because there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement and between that soil cement and the underlying clayey soils that the factor of safety against sliding exceeds the minimum required value. The pads are being surrounded with soil cement so that PFS can effectively use the eolian silt found at the site to provide an adequate subbase for support of the cask transporter. The eolian silt, otherwise, would be inadequate for this purpose and would require replacement with imported structural fill. The soil cement surrounding the pad may also help to spread the seismic load into the clayey soil outside the pad area to engage additional resistance against sliding of the pad. This effect would result in an increase in the factor of safety against sliding.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

SLIDING RESISTANCE OF ENTIRE N-S COLUMN OF PADS

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

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

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

Inertial forces due to Pads + Soil Cement:

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

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

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

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

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

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

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05996.02	G(B)	04 - 9		

$T_{N\text{-}S}$ = 10 pads x 30 ft x 67 ft x 2.1 ksf + 9 zones between the pads (Soil cement) x 30 ft x 5 ft x 1.4 ksf,

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

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

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

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

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

The resulting FS against sliding in the E-W direction will be even higher, because the soil cement zone between the pads is much wider (35 ft vs 5 ft) and longer (67 ft vs 30 ft) between the pads in the E-W direction than those in the N-S direction. The cask driving forces in the E-W direction are slightly higher than in the N-S direction, 10 pads x 2,212 K = 22,120 K vs 10 pads x 2,102 K = 21,020 K, resulting in an increased driving force of 22,120 K – 21,020 K = 1,100 K. The resistance to sliding in the E-W direction is increased much more than this, however. The increased resistance to sliding E-W = 35 ft x 67 ft x 1.4 ksf = 3,283 K / area between pads in the E-W row, compared to 5 ft x 30 ft x 1.4 ksf = 210 K / area between pads in the N-S column. Thus, the factor of safety against sliding of a row of pads in the E-W is much greater than that shown above for sliding of a column of pads in the N-S direction.

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

In this analysis, the resistance to sliding in the N-S direction includes the full passive resistance at the far end of the column of pads, which acts on the 2'-4" height of soil cement along the 30-ft width of the pad in the E-W direction.

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

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

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

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

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

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

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

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

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

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

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

The resulting FS against sliding in the E-W direction will be even higher, since there is much greater length available to resist sliding in that direction. The cask driving forces in the E-W direction are slightly higher than in the N-S direction, 10 pads x 2,212 K = 22,120 K vs 10 pads x 2,102 K = 21,020 K, resulting in an increased driving force of 22,120 K - 21,020 K = 1,100 K. The resistance to sliding in the E-W direction is increased more than this, including only the difference between the length vs the width of the pad. The soil cement adjacent to the pad provides (67 ft ÷ 30 ft) x 2,516 K, or 5,619 K of resistance based on the full passive pressure acting on the length of the pad, which is an increase of 5,619 K - 2,516 K = 3,103 K compared to the resistance provided by the soil cement to sliding in the N-S direction. This is greater than the increase in driving forces in the E-W direction; therefore, the factor of safety against sliding will be higher in the E-W direction. The soil cement zone between the pads also is much wider and longer between the pads in the E-W direction; therefore, there will be even more resistance to sliding E-W than N-S.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

The factor of safety against sliding is calculated as follows:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

and $H = 5$ ft

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

20 Pads in N-S Row

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

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

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

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

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

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

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

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

$$\phi = 17°$$

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

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

Acceleration in N-S direction, A = 0.284g

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

ϕ = 17°

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

Acceleration in N-S direction, A = 0.711g

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

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

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

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

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

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

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

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

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

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

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

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

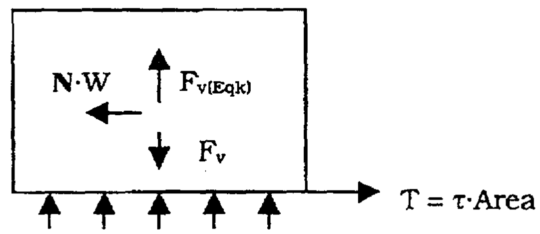
EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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

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

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

ESTIMATION OF HORIZONTAL DISPLACEMENT USING NEWMARK'S METHOD



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

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

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

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

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

$\sigma_n =$ Normal Stress

$\phi =$ Friction angle of cohesionless layer

$\sigma_n =$ Net Vertical Force/Area

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

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

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

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

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

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

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

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

MAXIMUM GROUND MOTIONS

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

LOAD CASES

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

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

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

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

GROUND MOTIONS FOR ANALYSIS

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

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

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

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

$$\phi = 30^\circ$$

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

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

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

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

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

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

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

where g is in units of inches/sec².

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

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

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

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

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

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

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

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

$$\phi = 30°$$

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

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

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

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

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

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

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

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

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

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

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

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

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with $\phi = 30°$, the estimated relative displacement of the pads due to the design basis ground motion based on Newmark's method of estimating displacements of embankments and dams due to earthquakes ranges from ~1.9 inches to 2.2 inches. Because there are no connections between the pads or between the pads and other structures, displacements of this magnitude, were they to occur, would not adversely impact the performance of the cask storage pads. There are several conservative assumptions that were made in determining these values and, therefore, the estimated displacements represent upper-bound values.

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

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

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

ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS

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

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

where

q_{ult} = ultimate bearing capacity

c = cohesion or undrained strength

q = effective surcharge at bottom of foundation, = γD_f

γ = unit weight of soil

B = foundation width

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

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

i_c, i_q, i_γ = load inclination factors

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

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

BEARING CAPACITY FACTORS

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

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

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

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

SHAPE FACTORS (FOR *L>B*)

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

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

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

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

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

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

$$d_\gamma = 1$$

INCLINATION FACTORS

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

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

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

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

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

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

STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

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

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

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

Allowable Bearing Capacity of Cask Storage Pads

Static Analysis: **Case IA - Static**

Soil Properties:

$c =$ **2,200** Cohesion (psf)

$\phi =$ **0.0** Friction Angle (degrees)

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

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

Foundation Properties:

$B' =$ **30.0** Footing Width - ft (E-W) $L' =$ **67.0** Length - ft (N-S)

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

0 $g = a_H$

$FS =$ **3.0** Factor of Safety required for $q_{allowable}$ **0** $g = a_V$

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

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

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.82	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.04	Eq 3.27

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

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

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STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

Allowable Bearing Capacity of Cask Storage Pads

Static Analysis: **Case IB - Static**

Soil Properties:	$c =$	**0** Cohesion (psf)	
Effective Stress Strengths	$\phi =$	**30.0** Friction Angle (degrees)	
	$\gamma =$	**80** Unit weight of soil (pcf)	
	$\gamma_{surch} =$	**100** Unit weight of surcharge (pcf)	
Foundation Properties:	$B' =$	**30.0** Footing Width - ft (E-W)	$L' =$ **67.0** Length - ft (N-S)
	$D_f =$	**3.0** Depth of Footing (ft)	
			0 $g = a_H$
	$FS =$	**3.0** Factor of Safety required for $q_{allowable}$	**0** $g = a_V$
	$F_{V\ Static} =$	**3,757** k & $EQ_V =$ **0** k →	**3,757** k for F_V
	$EQ_{H\ E\text{-}W} =$	**0** k & $EQ_{H\ N\text{-}S} =$ **0** k →	**0** k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, but $= 5.14$ for $\phi = 0$	=	30.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	18.40	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	22.40	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.27	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.26	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.82	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.03	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1 - d_q)/(N_q \tan\phi)$	=	1.03	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	N/A	Eq 3.27

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

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

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

Dynamic Bearing Capacity of the Cask Storage Pads

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

Based on Inertial Forces

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

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

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

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

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

Determine forces and moments due to earthquake.

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

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

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

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

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

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

Combine these horizontal components to calculate F_H:

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

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb.

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

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

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

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

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

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

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case II

Based on Inertial Forces Combined: 100 % N-S, 0 % Vert, 100 % E-W

Soil Properties:

$c =$ 2,200 Cohesion (psf)
$\phi =$ 0.0 Friction Angle (degrees)
$\gamma =$ 80 Unit weight of soil (pcf)
$\gamma_{surch} =$ 100 Unit weight of surcharge (pcf)

Footing Dimensions:

$B =$ 30.0 Width - ft (E-W)
$L =$ 67.0 Length - ft (N-S)

Foundation Properties:

$B' =$ 15.6 Effective Ftg Width - ft (E-W) $L' =$ 52.6 Length - ft (N-S)
$D_f =$ 3.0 Depth of Footing (ft)

0.711 $g = a_H$

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

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

$EQ_{H\ E\text{-}W} =$ 2,671 k & $EQ_{H\ N\text{-}S} =$ 2,671 k → 3,777 k for F_H

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

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

$N_c = (N_q - 1)\ \cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\,(N_q + 1)\ \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.06	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.88	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\ D_f/B$	=	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 $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.08	Eq 3.27
$m_B = (2 + B/L) / (1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B) / (1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	=	0.79 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.39	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	5,338	psf =	5,038	+	300	+	0

$q_{all} =$ 4,850 psf $= q_{ult} / FS$

$q_{actual} =$ 4,565 psf $= (F_{v\ Static} + EQ_v) / (B' \times L')$

$FS_{actual} =$ 1.17 $= q_{ult} / q_{actual}$ > 1.1 Hence OK

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

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

Determine forces and moments due to earthquake.

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

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

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

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

⇒ Nc = 870 K

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

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

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

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

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

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

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

Determine moments acting on pad due to casks.

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

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

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

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

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

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

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

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

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case IIIA

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

Soil Properties:	$c =$	**2,200** Cohesion (psf)	Footing Dimensions:		
	$\phi =$	**0.0** Friction Angle (degrees)	B = **30.0**	Width - ft (E-W)	
	$\gamma =$	**80** Unit weight of soil (pcf)	L = **67.0**	Length - ft (N-S)	
	$\gamma_{surch} =$	**100** Unit weight of surcharge (pcf)			
Foundation Properties:	$B' =$	**18.3** Effective Ftg Width - ft (E-W)	L' = **55.3**	Length - ft (N-S)	
	$D_f =$	**3.0** Depth of Footing (ft)			
				0.711 $g = a_H$	
	$FS =$	**1.1** Factor of Safety required for $q_{allowable}$		**0.695** $g = a_V$	

$F_{V\,Static} =$ **3,757** k & $EQ_V =$ **-2,611** k → **1,146** k for F_V

$EQ_{H\,E\text{-}W} =$ **749** k & $EQ_{H\,N\text{-}S} =$ **749** k → **1,060** k for F_H

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

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

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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.06	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.87	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1 - d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.07	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.79 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.86	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**11,344**	**psf =**	**11,044**	+	**300**	+	**0**
q_{all} =	**10,310**	**psf = q_{ult} / FS**					
q_{actual} =	**1,132**	**psf = $(F_{v\,Static} + EQ_v)/(B' \times L')$**					
FS_{actual} =	**10.02**	**= q_{ult}/q_{actual}**			**> 1.1 Hence OK**		

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

Determine forces and moments due to earthquake.

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

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

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

⇒ Nc = 2,059 K

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

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

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

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

Using 40% of N-S:

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

Using 100% of E-W:

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

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

Determine moments acting on pad due to casks.

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

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

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

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

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

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

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

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

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case IIIB

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

Soil Properties:

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

Foundation Properties:

$B' =$	**15.7**	Effective Ftg Width - ft (E-W)	L' = **59.0**		Length - ft (N-S)
$D_f =$	**3.0**	Depth of Footing (ft)			
					0.711 $g = a_H$
FS =	**1.1**	Factor of Safety required for $q_{allowable}$			**0.695** $g = a_V$
$F_{V\ Static} =$	**3,757** k	& $EQ_V =$	**-1,044** k →	**2,712**	k for F_V
$EQ_{H\ E\text{-}W} =$	**2,077** k	& $EQ_{H\ N\text{-}S} =$	**1,068** k →	**2,336**	k for F_H

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

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

$N_c = (N_q - 1)\ \cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\,(N_q + 1)\ \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.05	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.89	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\ D_f/B$	=	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 $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.08	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	=	1.10 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.61	Eq 3.18c
$i_q = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.64	Eq 3.16a

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

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

Determine forces and moments due to earthquake.

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

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

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

$\Rightarrow$ Nc = 2,059 K

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

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

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

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

Using 100% of N-S:

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

Using 40% of E-W:

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

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

Determine moments acting on pad due to casks

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

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

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

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

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

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

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

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

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

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case IIIC

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

Soil Properties:	$c =$	**2,200** Cohesion (psf)	Footing Dimensions:	
	$\phi =$	**0.0** Friction Angle (degrees)	B = **30.0**	Width - ft (E-W)
	$\gamma =$	**80** Unit weight of soil (pcf)	L = **67.0**	Length - ft (N-S)
	$\gamma_{surch} =$	**100** Unit weight of surcharge (pcf)		
Foundation Properties:	$B' =$	**22.0** Effective Ftg Width - ft (E-W)	L' = **52.7**	Length - ft (N-S)
	$D_f =$	**3.0** Depth of Footing (ft)		
				0.711 $g = a_H$
	FS =	**1.1** Factor of Safety required for $q_{allowable}$		**0.695** $g = a_V$

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

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

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.08	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.83	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.48 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.39	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.75	Eq 3.16a

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

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

Determine forces and moments due to earthquake.

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

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

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

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

$\Rightarrow$ Nc = 4,834 K

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

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

2,028 K 3,867K

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

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

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

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

Determine moments acting on pad due to casks.

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

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

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

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

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

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

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

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

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case IVA

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

Soil Properties:

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

Foundation Properties:

$B' =$	**26.6**	Effective Ftg Width - ft (E-W)	L' = **63.6**	Length - ft (N-S)
$D_f =$	**3.0**	Depth of Footing (ft)		

0.711 $g = a_H$

FS =	**1.1**	Factor of Safety required for $q_{allowable}$		**0.695**	$g = a_V$
$F_{V\ Static} =$	**3,757** k &	$EQ_V =$ **2,611** k	→	**6,368**	k for F_V
$EQ_{H\ E\text{-}W} =$	**1,068** k &	$EQ_{H\ N\text{-}S} =$ **1,068** k	→	**1,511**	k for F_H

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.08	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.83	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1 - d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	=	0.79 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.88	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	11,567	psf =	11,267	+	300	+	0
q_{all} =	10,510	psf = q_{ult} / FS					
q_{actual} =	3,762	psf = $(F_{V\ Static} + EQ_V) / (B' \times L')$					
FS_{actual} =	3.07	= q_{ult} / q_{actual}		>	1.1	Hence OK	

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

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

Determine forces and moments due to earthquake.

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

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

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

$\Rightarrow$ Nc = 3,645 K

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

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

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

Using 40% of N-S:

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

Using 100% of E-W:

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

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

Determine moments acting on pad due to casks

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

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

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

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

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

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

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

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

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

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case IVB

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

Soil Properties:

$c =$ **2,200** Cohesion (psf)

$\phi =$ **0.0** Friction Angle (degrees)

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

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

Footing Dimensions:

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

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

Foundation Properties:

$B' =$ **18.8** Effective Ftg Width - ft (E-W) $L' =$ **62.5** Length - ft (N-S)

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

0.711 $g = a_H$

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

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

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

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.06	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.88	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.06	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	1.19 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.64	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.64	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	8,508	psf =	8,208	+	300	+	0

$q_{all} =$ **7,730** psf = q_{ult} / FS

$q_{actual} =$ **4,095** psf = $(F_{V\,Static} + EQ_V)/(B' \times L')$

$FS_{actual} =$ **2.08** = q_{ult}/q_{actual} > **1.1 Hence OK**

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

Determine forces and moments due to earthquake.

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

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

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

$\Rightarrow$ Nc = 3,645 K

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

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

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

Using 100% of N-S:

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

Using 40% of E-W:

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

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

Determine moments acting on pad due to casks

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

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

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

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

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

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

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

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

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

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case IVC

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

Soil Properties:

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

Foundation Properties:

$B' =$	**25.5**	Effective Ftg Width - ft (E-W)	L' = **55.8**		Length - ft (N-S)
$D_f =$	**3.0**	Depth of Footing (ft)			
					0.711 $g = a_H$
FS =	**1.1**	Factor of Safety required for $q_{allowable}$			**0.695** $g = a_V$
$F_{V\ Static} =$	**3,757** k	& $EQ_V =$	**1,044** k →	**4,801**	k for F_V
$EQ_{H\ E\text{-}W} =$	**1,068** k	& $EQ_{H\ N\text{-}S} =$	**2,671** k →	**2,877**	k for F_H

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

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

	$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	= 5.14	Eq 3.6 & Table 3.2
	$N_q = e^{\pi\tan\phi}\tan^2(\pi/4 + \phi/2)$	= 1.00	Eq 3.6
	$N_\gamma = 2(N_q + 1)\tan(\phi)$	= 0.00	Eq 3.8
	$s_c = 1 + (B/L)(N_q/N_c)$	= 1.09	Table 3.2
	$s_q = 1 + (B/L)\tan\phi$	= 1.00	"
	$s_\gamma = 1 - 0.4(B/L)$	= 0.82	"
For $D_f/B \le 1$:	$d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2\,D_f/B$	= 1.00	Eq 3.26
	$d_\gamma = 1$	= 1.00	"
For $\phi > 0$:	$d_c = d_q - (1-d_q)/(N_q\tan\phi)$	= N/A	
For $\phi = 0$:	$d_c = 1 + 0.4(D_f/B)$	= 1.05	Eq 3.27
	$m_B = (2 + B/L)/(1 + B/L)$	= 1.69	Eq 3.18a
	$m_L = (2 + L/B)/(1 + L/B)$	= 1.31	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$:	$\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	= 0.38 rad	
	$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	= 1.36	Eq 3.18c
	$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	= 1.00	Eq 3.14a
	$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	= 0.00	Eq 3.17a
For $\phi = 0$:	$i_c = 1 - (m\,F_H/B'L'c\,N_c)$	= 0.76	Eq 3.16a

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

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

BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

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

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

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

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

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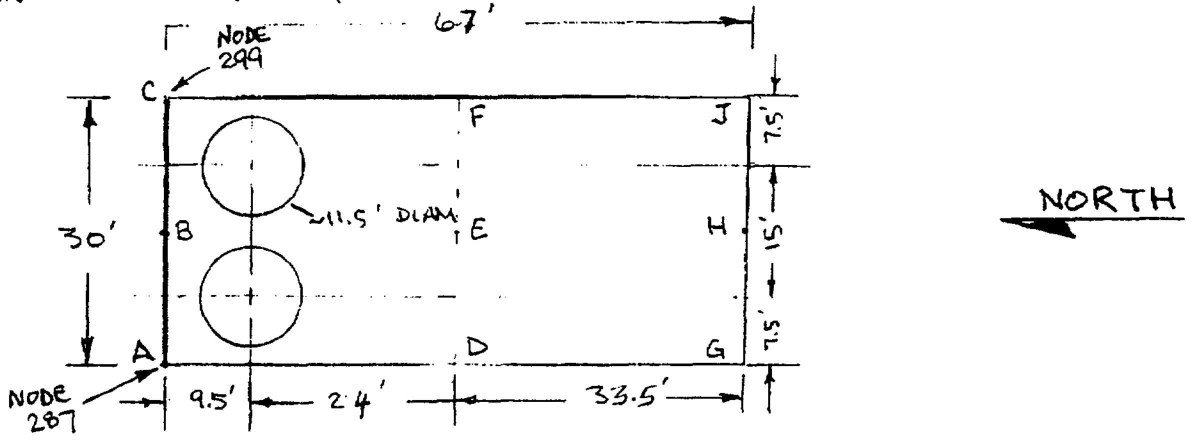
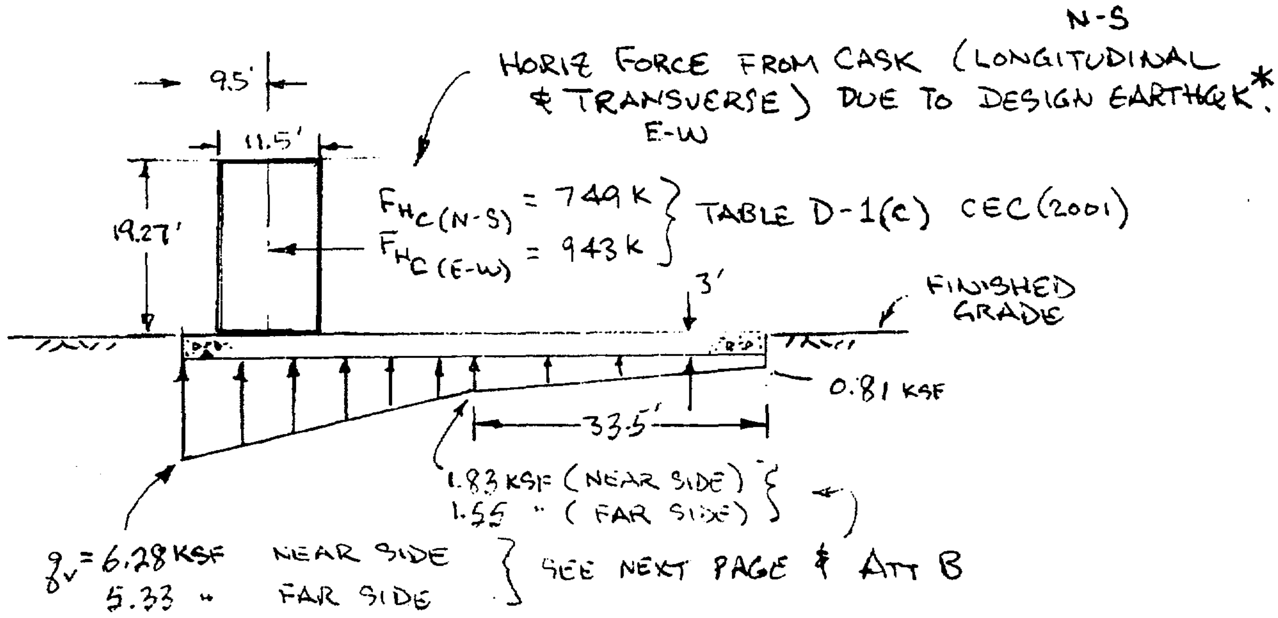
DYN BEARING CAPACITY OF PAD: 2-CASK CASE



PLAN



ELEV

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

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05996.02	G(8)	04-9		

DYN BEARING CAPACITY OF PAD: 2-CASK CASE

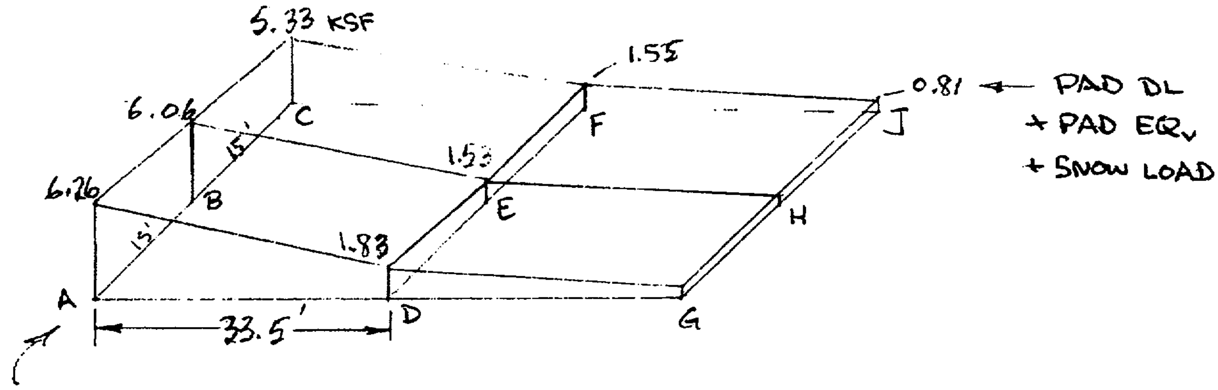
SOIL BEARING PRESSURES ARE BASED ON INFO FROM CEC(2001) INCLUDED IN ATT B AND ARE SUMMARIZED IN TABLE 1.

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

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

CASK EQ PRESSURES ARE SHOWN ON TABLE 1.

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



FOR LOADED HALF OF PAD:

$$F_v \simeq \left[\overbrace{\frac{15'}{2}\times(6.26+2\times6.06+5.33)^{ksf}}^{177.82}+\overbrace{\frac{15'}{2}(1.83+2\times1.53+1.55)^{ksf}}^{48.30}\right]\frac{33.5}{2}$$

$F_v \simeq 3790$ K FOR LOADED 1/2 OF PAD

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

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

~~0086~~

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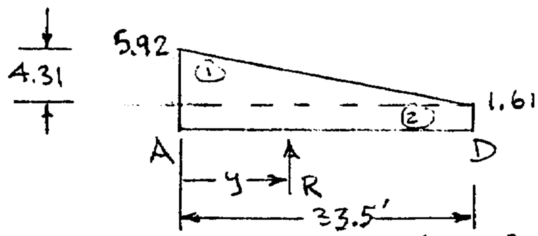
DYN BEARING CAPACITY OF PAD: 2-CASK CASE

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



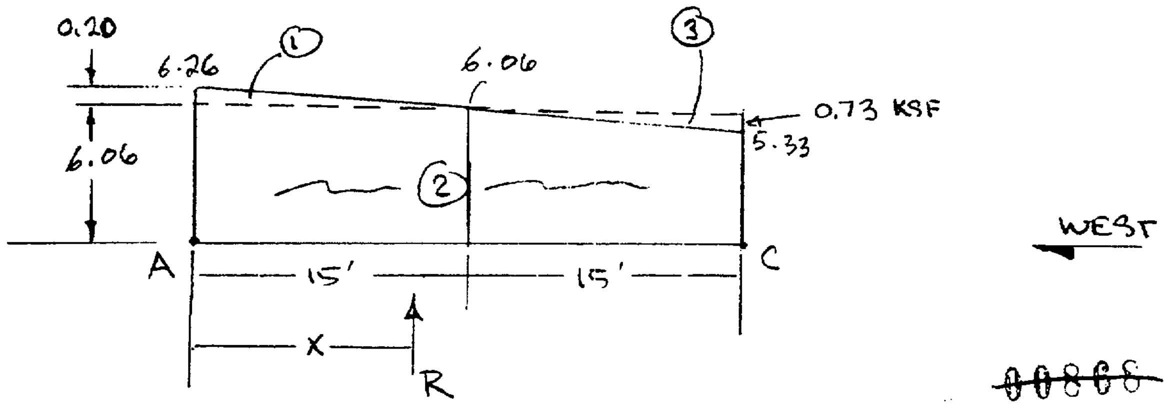
ΣM_A TO FIND LOC'N OF F_V

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

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

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

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



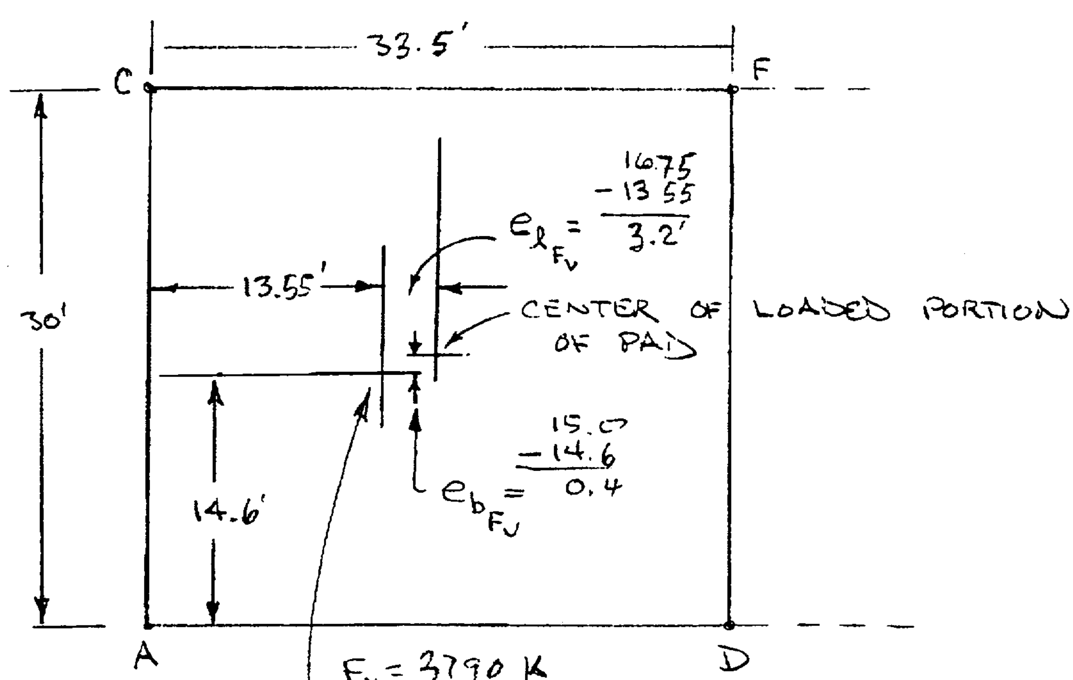
~~00868~~

DYN BEARING CAPACITY OF PAD : 2-CASK CASE

ΣM_A

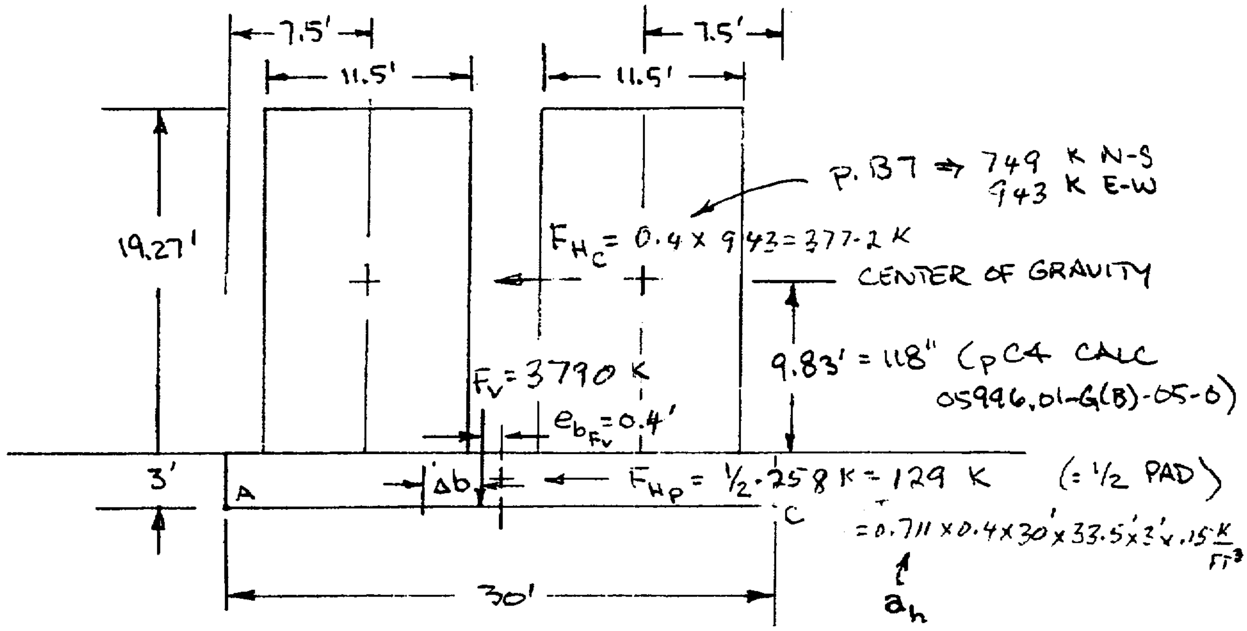
	AREA (K/FT)	MOMENT ARM (FT)	MOMENT K-FT/FT
1	$\frac{1}{2}$ 0.20 KSF × 15' = 1.5	$\frac{1}{3} \cdot 15' = 5'$	7.5
2	6.06 KSF × 30' = 181.8	$\frac{1}{2} \cdot 30' = 15'$	2,727
3	$-\frac{1}{2}$ 0.73 KSF × 15' = −5.48	$15' + \frac{2}{3} 15 = 25'$	−136.88
	$\Sigma F_V = R = 177.8$ K/FT		2597.6

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



POINT OF APPLICATION OF F_V DUE TO PAD (DL+EQ) & CASKS (LL+EQ) ~~00889~~
FOR 2-CASK CASE

DYN BEARING CAPACITY OF PAD: 2-CASK CASE



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

½ PAD *

2-CASKS TRANSVERSE LOADING (E-W)

$$R \Delta b = 1.5' \times \frac{129^K}{2} + (3' + 9.83')\ 377.2^K$$

$R = F_V = 3790$ K

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

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

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

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

~~00870~~

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DYN BEARING CAPACITY OF PAD: 2-CASK CASE

CALCULATE L' SIMILARLY FOR LONGITUDINAL DIRECTION

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

ΣM_{F_V}

$R\,\Delta\ell = 1.5' \times \frac{129}{2}\text{ K} + (3' + 9.83')(300\text{ K})$ (½ PAD; 2-CASKS LONGITUDINAL LOADS)

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

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

ADD $e_{\ell_{F_V}} = 3.2' \Rightarrow e_\ell = 1.07' + 3.2' = 4.27'$

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

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

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

$F_{H_{E\text{-}W}} = 377.2\text{K} + 129\text{ K} = 506.2\text{ K}$

2-CASK EQ_h (p B7); ½ PAD EQ_h

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

FS = 1.1

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

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

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

~~00871~~

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

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

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 2 CASKS

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

Soil Properties:
- $c =$ **2,200** Cohesion (psf)
- $\phi =$ **0.0** Friction Angle (degrees)
- $\gamma =$ **80** Unit weight of soil (pcf)
- $\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Footing Dimensions:
- B = **30.0** Width - ft (E-W)
- L = **67.0** Length - ft (N-S)

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

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

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

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

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.18	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.62	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.87 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.53	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.86	Eq 3.16a

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

[geot]j05996\calc\brng_cap\Pad\Wint_Fang-8.xls Sheet 2-Cask

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CALCULATION SHEET

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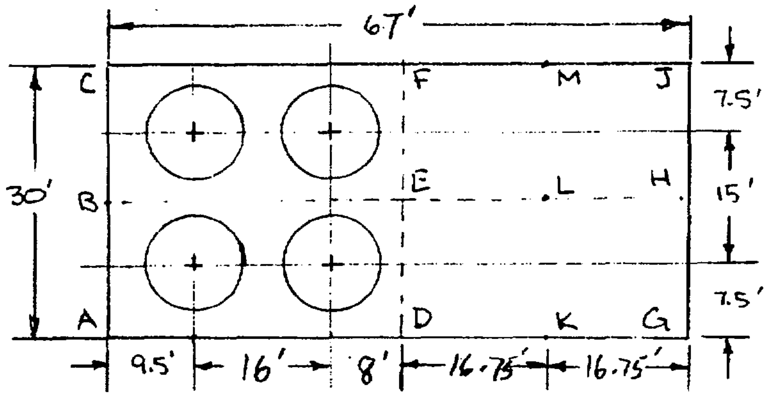
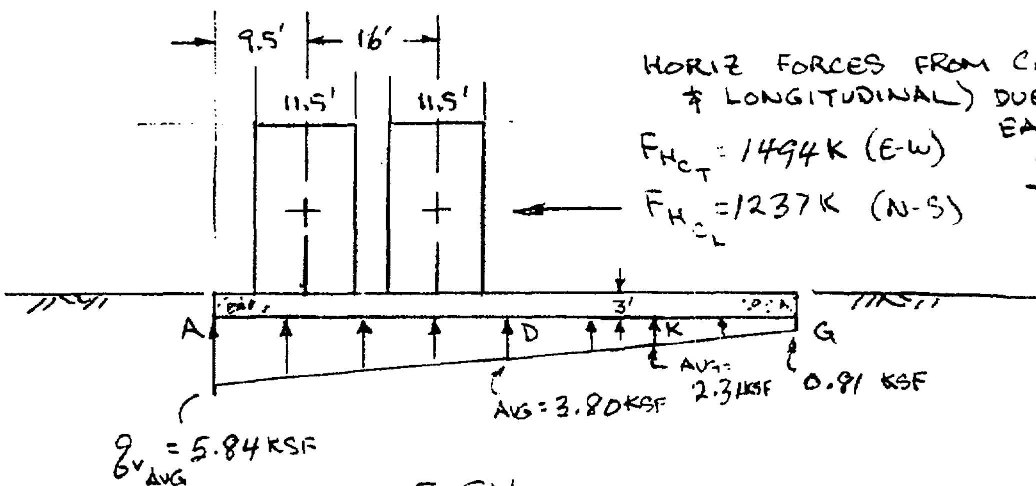
DYN BEARING CAPACITY OF PAD: 4-CASK CASE



PLAN



HORIZ FORCES FROM CASK (TRANSVERSE & LONGITUDINAL) DUE TO DESIGN EARTHQUAKE (FROM p B7)

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

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

TABLE D-1(c) CEC (2001)

ELEV

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

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DYN BEARING CAPACITY OF PAD: 4-CASK CASE

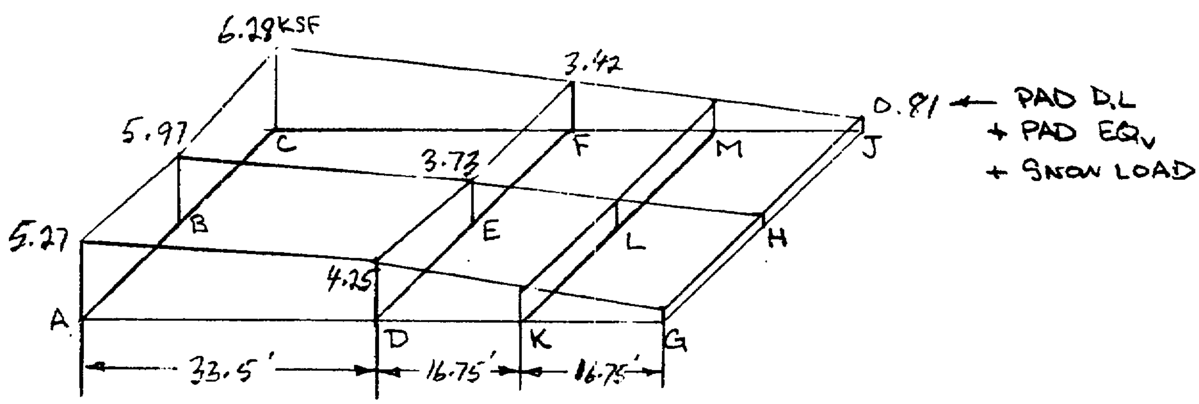
SOIL BEARING PRESSURES ARE BASED ON INFO FROM CEC (2001) INCLUDED IN ATT B AND ARE SUMMARIZED IN TABLE 1.

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

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

CASK EQ PRESSURES ARE SHOWN ON TABLE 1

RESULTING PRESSURE DISTRIBUTION:



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

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

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

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

~~00875~~

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

CALCULATE F_V

ALONG LINE — AREA = K/FT

AC $\frac{15'}{2}(5.27 + 2\times5.97 + 6.28)$ KSF $= 176.18$ K/FT

DF $\frac{15'}{2}(4.25 + 2\times3.73 + 3.42) = 113.48$

KM $\frac{15'}{2}(2.53 + 2\times2.27 + 2.12) = 68.93$

$F_V \sim \frac{33.5}{2}(176.18 + 113.48)^{K}/_{1} + \frac{16.75'}{2}(113.48 + 68.93)^{K}/_{1}$

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

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

NOTE AVG VERT STRESS ALONG LINES.

LINE

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

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

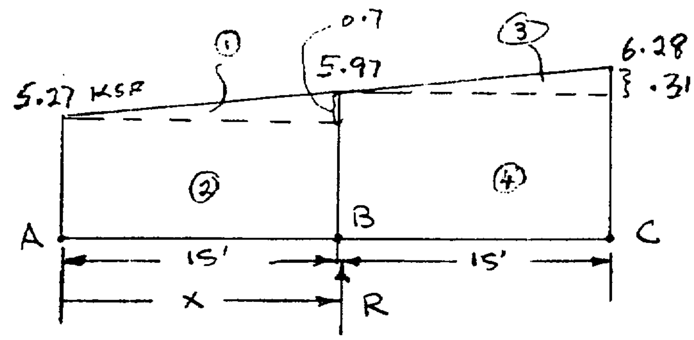
KM $= \frac{68.93\ K/FT}{30'} = 2.30$ KSF

~~00876~~

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

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

ALONG LINE AC



ΣM_A

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

$F_V = \Sigma = 176.175$ $\qquad R_x = \Sigma = 2718.38$

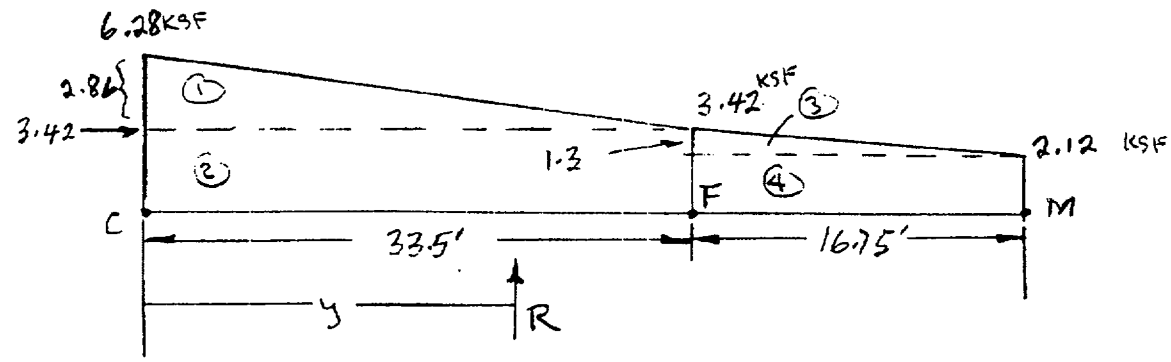
$$\therefore x = \frac{2718.38 \text{ K-FT/FT}}{176.175 \text{ K/FT}} = 15.4'$$

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

~~00877~~

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

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



ΣM_A TO FIND y

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

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

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

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

AS SHOWN ON NEXT PAGE

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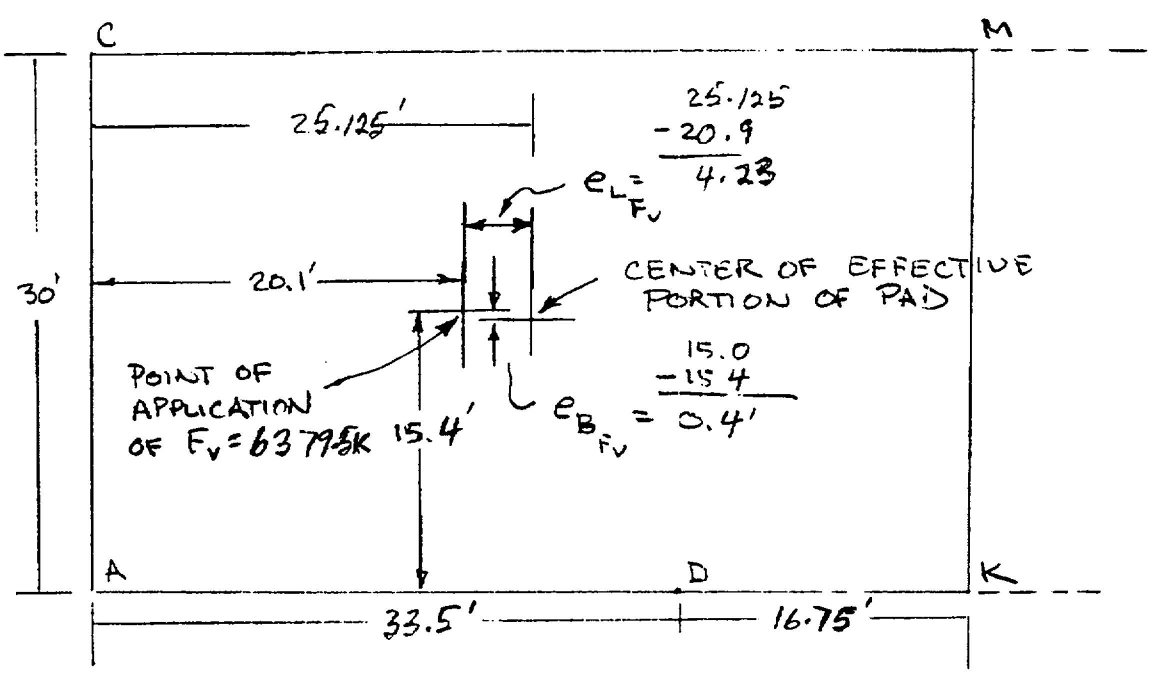
DYN BEARING CAPACITY OF PAD: 4-CASK CASE

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



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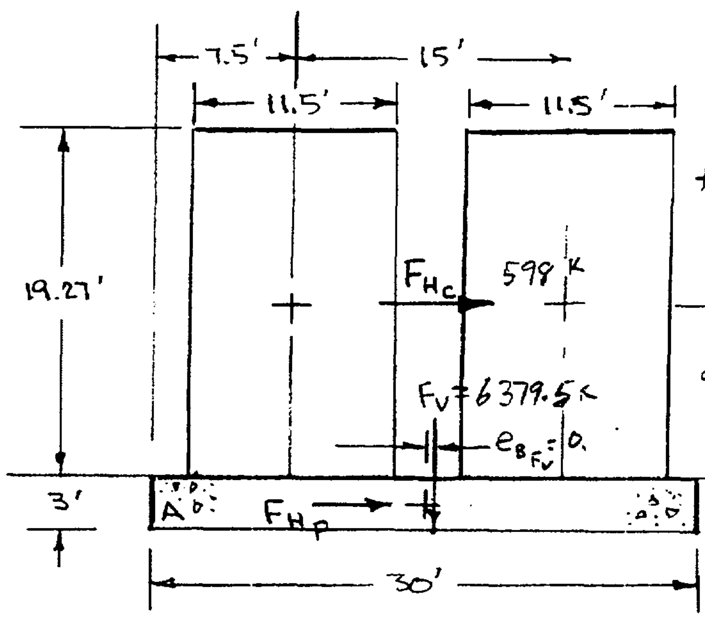
DYN BEARING CAPACITY OF PAD: 4-CASK CASE



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

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

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

CENTER OF GRAVITY OF CASK

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

$40\% F_{H_P} = 257^K$ FOR $30' \times 67'$

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

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

3/4 PAD — 40% 4-CASK TRANSVERSE F_H

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

$\uparrow$ = $F_V = 6379.5^K$ ON $30 \times 50.25'$ PORTION OF PAD

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

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

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

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DYN BEARING CAPACITY OF PAD: 4-CASK CASE

CALCULATE L' SIMILARLY FOR LONGITUDINAL F_H

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

$F_{H_{CL}}$ ← 4 CASKS

$$+\tfrac{3}{4}\times 40\% F_{HP} = \frac{3}{4}\,257 \Rightarrow EQ_{H\ N-S} = 698\ K$$

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

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

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

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

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

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

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

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

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

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

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

$\phi = 0°$ $C = 2.2$ KSF

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

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 4 CASKS

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

Soil Properties:	$c =$	**2,200**	Cohesion (psf)	Footing Dimensions:		
	$\phi =$	**0.0**	Friction Angle (degrees)	B =	**30.0**	Width - ft (E-W)
	$\gamma =$	**80**	Unit weight of soil (pcf)	L =	**67.0**	Length - ft (N-S)
	$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)			
Foundation Properties:	$B' =$	**26.7**	Effective Ftg Width - ft (E-W)	L' =	**39.7**	Length - ft (N-S)
	$D_f =$	**3.0**	Depth of Footing (ft)			
	$FS =$	**1.1**	Factor of Safety required for $q_{allowable}$.			
	$F_V =$	**6,380**	k (Includes EQ_V)			
	$EQ_{H\ E\text{-}W} =$	**791**	k & $EQ_{H\ N\text{-}S} =$ **688** k → **1,048** k for F_H			

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

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

Equation		Value	Reference
$N_c = (N_q - 1)\ \cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\ (N_q + 1)\ \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.13	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\ (B/L)$	=	0.73	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\ D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)\ /\ (N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\ (D_f/B)$	=	1.04	Eq 3.27
$m_B = (2 + B/L)\ /\ (1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)\ /\ (1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}\ /\ EQ_{H\ N\text{-}S})$	=	0.85 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.53	Eq 3.18c
$i_q = \{\ 1 - F_H\ /\ [(F_v + EQ_v) + B'\ L'\ c \cot\phi]\ \}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{\ 1 - F_H\ /\ [(F_v + EQ_v) + B'\ L'\ c \cot\phi]\ \}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\ F_H\ /\ B'\ L'\ c\ N_c)$	=	0.87	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**11,879**	**psf =**	**11,579**	+	**300**	+	**0**
q_{all} =	**10,790**	**psf = q_{ult} / FS**					
q_{actual} =	**6,017**	**psf = $(F_v + EQ_v)\ /\ (B' \times L')$**					
FS_{actual} =	**1.97**	**= $q_{ult}\ /\ q_{actual}$**			**> 1.1**	**Hence OK**	

[geot]j05996\calc\brng_cap\Pad\Wint_Fang-8.xls Sheet 4-Cask

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CALCULATION SHEET

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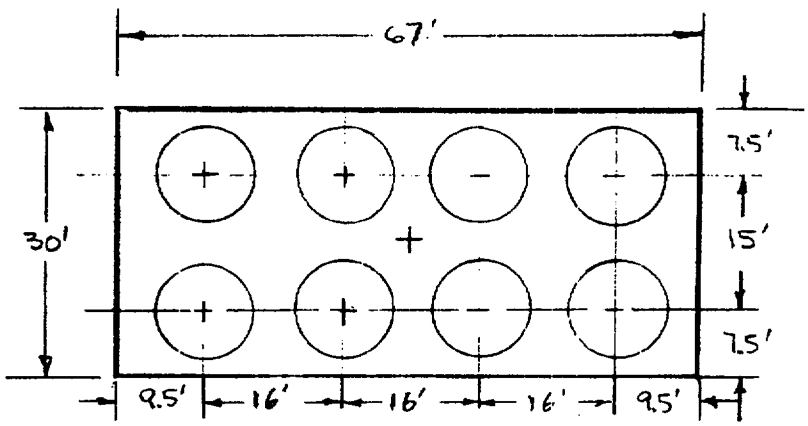
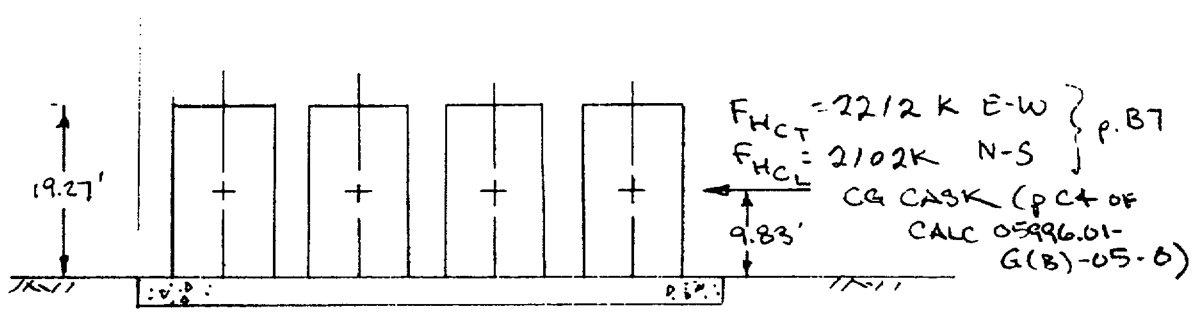
DYN BEARING CAPACITY OF PAD: 8-CASK CASE



PLAN



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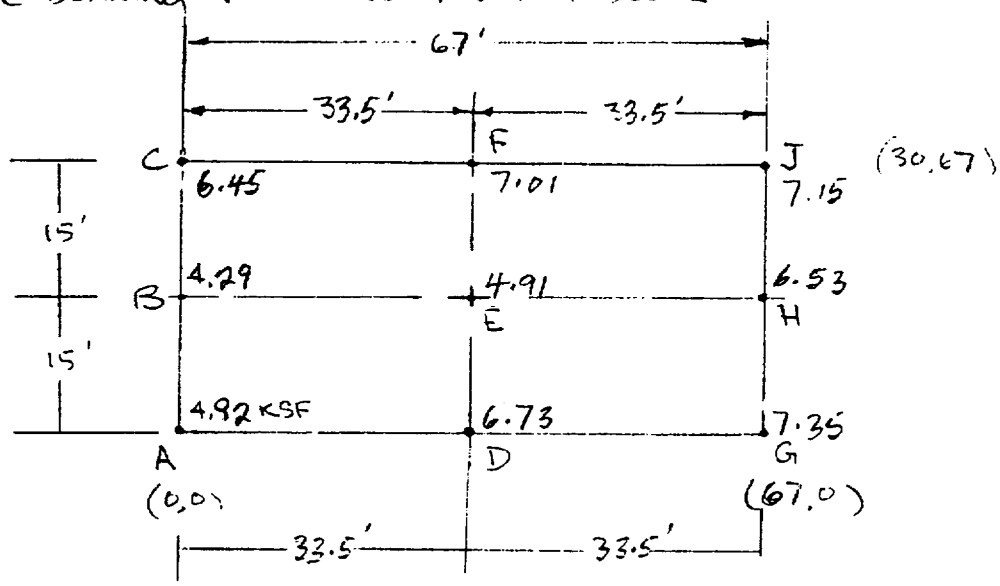
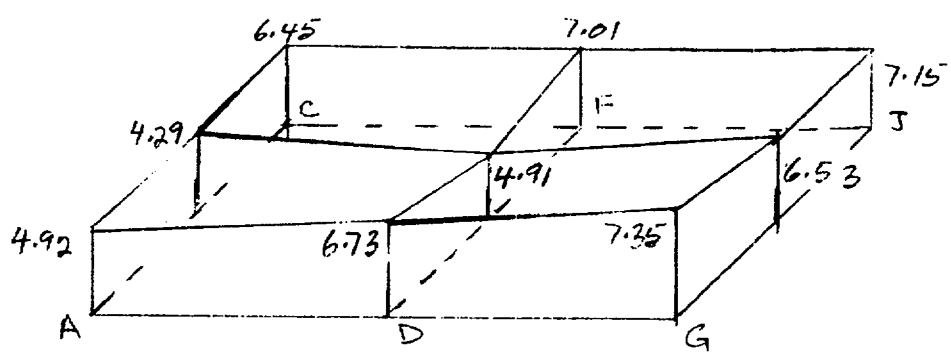
DYN BEARING CAPACITY OF PAD: 8-CASK CASE

SOIL BEARING PRESSURES FROM TABLE 1





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DYN BEARING CAPACITY OF PAD: 8-CASK CASE

CALCULATE F_V :

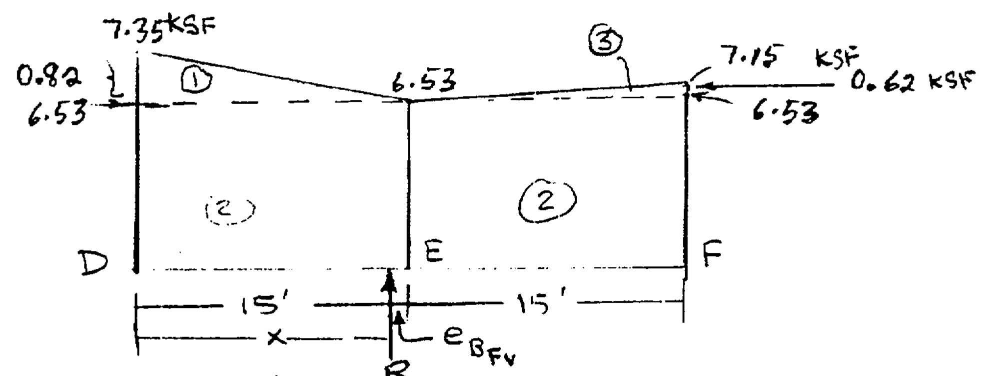
ALONG LINE	AREA = K/FT	F_V (K/FT)	$\bar{q}$ AVG (KSF)
AC	$\frac{15}{2}(4.92 + 2\times 4.29 + 6.45) =$	149.63	4.99
DF	$\frac{15}{2}(6.73 + 2\times 4.91 + 7.01) =$	176.70	5.89
GJ	$\frac{15}{2}(7.35 + 2\times 6.53 + 7.15) =$	206.7	6.89

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

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

ALONG LINE GJ, WHICH HAS THE GREATEST STRESSES

ΣM_D



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

$$\therefore x = \frac{Rx}{R} = \frac{3085.5\ \text{K-FT/FT}}{206.7\ \text{K/FT}} = 14.92'$$

$$e_{B_{F_V}} = \frac{30}{2} - 14.92 = 0.08'$$

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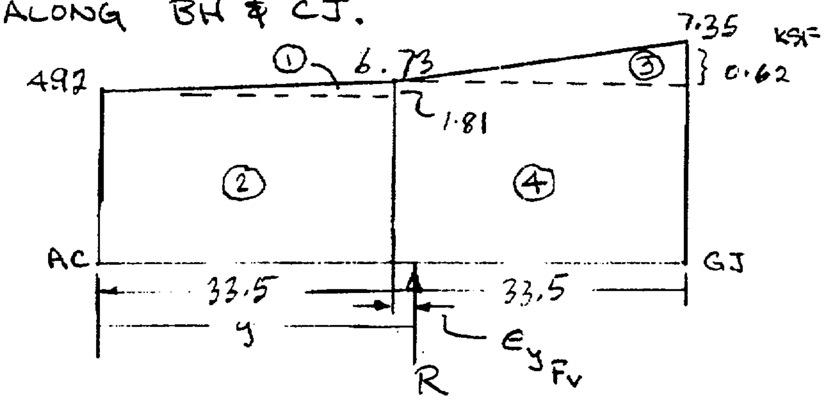
DYNAMIC BEARING CAPACITY OF PAD - 8-CASK CASE

DETERMINE ECCENTRICITY OF F_V WRT L, $e_{L_{F_V}} = \frac{L}{2} - y$
USE AVERAGE VALUES ALONG LINE AG, WHICH ARE GREATER THAN ALONG BH & CJ.



ΣM_{AC}

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

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

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

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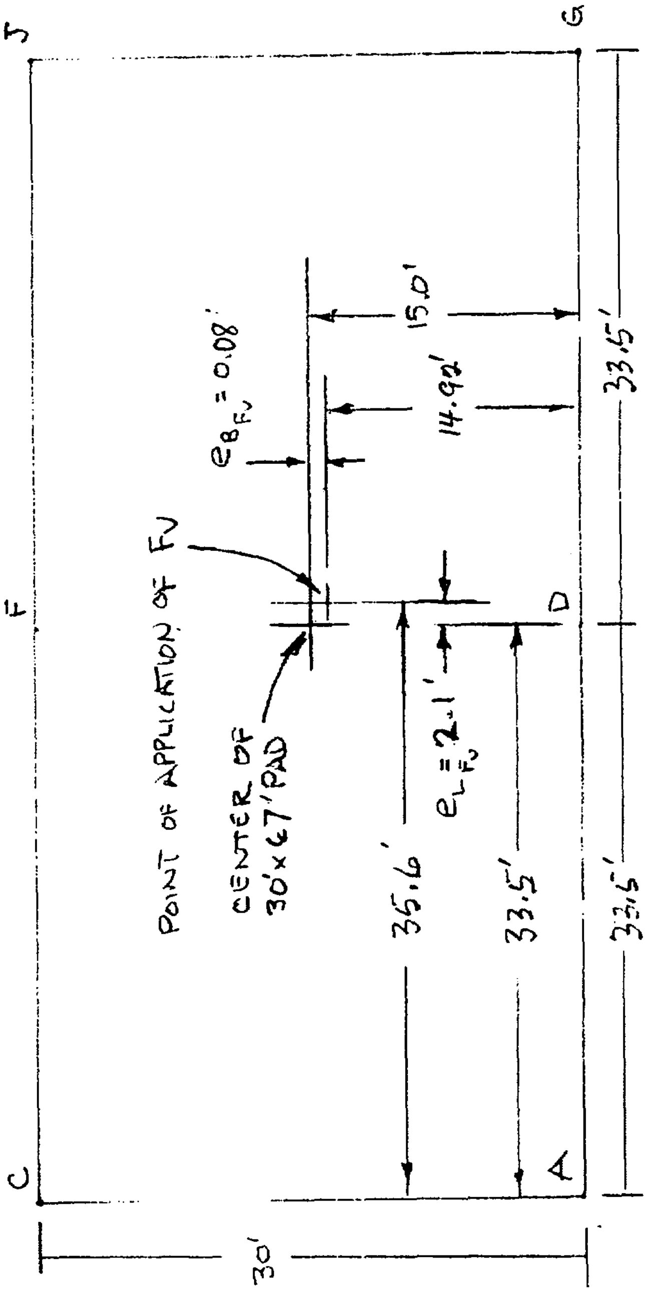
PLAN VIEW OF PAD SHOWING LOCATION OF VERTICAL FORCE DUE TO VERTICAL STRESSES FROM STATIC AND DYNAMIC LOADS FOR 8-CASK LOADING CASE



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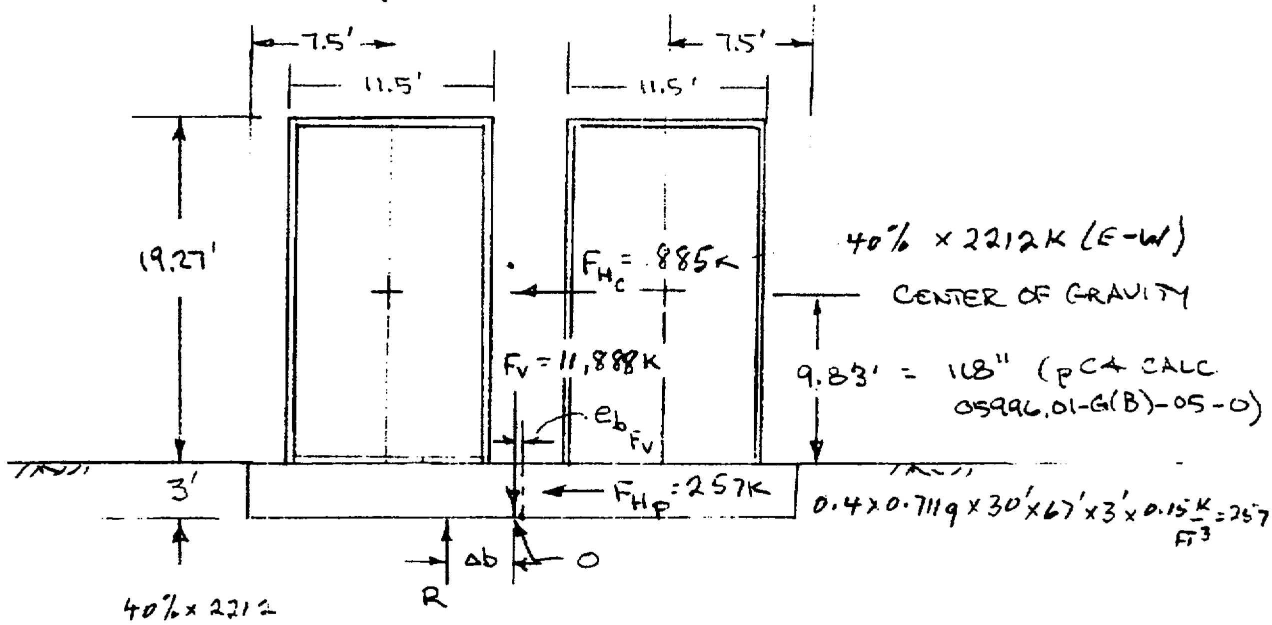
DYN BEARING CAPACITY OF PAD: 8-CASK CASE



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

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

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

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

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

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

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

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

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DYN BEARING CAPACITY OF PAD: 8-CASK CASE

SIMILARLY FOR LONGITUDINAL DIRECTION

40% of 2102K

$F_{H_C} = 841 \text{ K}$ ADD $F_{HP} = 257 \text{K} \Rightarrow EQ_{H_{N-S}} = 1098 \text{ K}$

↑ ℄ p B7

PAD CASKS

R = F_V

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

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

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

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

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

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

$F_V = 11{,}888$ K (STATIC + DYN 8 CASKS)

$EQ_{H\,E\text{-}W} = \overset{F_{HP}}{257} \text{ K} + \overset{F_{HC}}{885} \text{ K} = 1142 \text{ K}$

$EQ_{H\,N\text{-}S} = 257 \text{ K} + 841 \text{ K} = 1098 \text{ K}$

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

$\phi = 0°$ $C = 2.2$ KSF

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

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 8 CASKS

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

Soil Properties:
- $c =$ **2,200** Cohesion (psf)
- $\phi =$ **0.0** Friction Angle (degrees)
- $\gamma =$ **80** Unit weight of soil (pcf)
- $\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Footing Dimensions:
- B = **30.0** Width - ft (E-W)
- L = **67.0** Length - ft (N-S)

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

- FS = **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **11,888** k (Includes EQ_V)
- $EQ_{H\ E\text{-}W} =$ **1,142** k & $EQ_{H\ N\text{-}S} =$ 1,098 k → **1,584** k for F_H

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

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

Expression		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
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.82	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.04	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.81 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.51	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.88	Eq 3.16a

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

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

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

CONCLUSIONS

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

Case I Static

Case II Static + dynamic horizontal forces due to the earthquake

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

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

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

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

OVERTURNING STABILITY OF THE CASK STORAGE PADS

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

SLIDING STABILITY OF THE CASK STORAGE PADS

The cask storage pads will be constructed on and within soil cement, as shown in Figure 3. Analyses presented above demonstrate that the static, undrained strength of the in situ clayey soils is sufficient to preclude sliding (FS = 1.27 vs minimum required value of 1.1), provided that the full strength of the clayey soils is engaged. The soil-cement layer beneath the pads provides an "engineered mechanism" to ensure that the full, static, undrained strength of the clayey soils is engaged in resisting sliding forces. This soil cement will be designed to have a minimum unconfined compressive strength of 40 psi. The bond between this soil-cement layer and the base of the concrete pad will be stronger than the static, undrained strength of the in situ clayey soils. The factor of safety against sliding between the concrete at the base of the pad and the surface of the underlying soil cement is greater than 1.98, which exceeds the factor of safety between the bottom of the soil cement and the underlying clayey soils. Therefore, the minimum factor of safety against sliding of the overall cask storage pad design is at least 1.27.

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

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

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

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