CONDENSED 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. WEN-SHOU TSENG**
	) (Utah Contention L/QQ)

Tuesday, March 12, 2002 - 9:55 a.m.

Location: Utah Attorney General's Office
160 East 300 South
Salt Lake City, Utah

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

State's
Exhibit 113

1 calculation, and he indicated that the differential is
2 the order of .001 second. And at .001 second one would
3 consider, you know, for that small pad the effect would
4 be very, very high frequencies, something on the order
5 greater than 50 Hz, which is beyond the frequency range
6 of our interest, actually. So I would expect this
7 effect would be insignificant.
8 Q. What about for a quadrant of pads in soil
9 cement that would act as one integrated pad?
10 A. Would you say that again?
11 Q. What about for a quadrant of pads, which I
12 believe is 125 pads, and the soil cement in between
13 that act as an integrated structure?
14 MR. GAUKLER: Objection, lack of foundation.
15 Answer if you can.
16 A. Well, even though you have soil cement in
17 between the pad, but there is no structure continuing
18 through -- in other words, there is no rebar going
19 through it. So even though it may appear to be
20 integrated, that -- structurally they are still
21 separate. So I wouldn't consider that to be a credible
22 case of integrating all the pads together.
23 Q. So do you believe the pads will act
24 independently of each other, out of phase in their
25 motions?

1 A. Well, each pad of course is by itself a
2 structural element. It may have some minor interaction
3 between pad to pad through the underlying soil, but
4 again, that I believe will be a quite secondary effect.
5 Now, if you consider all the pads lying in
6 one big quadrant, from one end to the other there may
7 be bigger variation, but since each pad is structurally
8 not connected, there is no effect from one to the
9 other, especially from one extreme end to the other
10 extreme end. And they separate so far apart and they
11 won't impact much, either. Only when the pad, they are
12 structurally integrated together in such a large
13 dimension that such time differential would have bigger
14 effect.
15 Q. So do you believe the soil cement will not
16 have an impact in integrating the motion of the
17 different pads together?
18 A. It stiffens up the soil, certainly, and that
19 effect has been included in this. But structurally you
20 don't have really positive connections. Eventually I
21 don't think they would behave as an integrated
22 structure.
23 Q. Did you evaluate the adverse effect of
24 inclined waves on the seismic response of the pads?
25 A. Would you say that again?

1 Q. Did you evaluate any adverse effect of
2 inclined -- strike that. Did you evaluate any adverse
3 effects of inclined waves on the seismic response of
4 the pads?
5 A. We -- in our calculation for pad design we
6 did not explicitly consider that. But this incline is
7 a small angle, which Bob Youngs had estimated to be the
8 case, X minus like 11 degree from vertical, which is
9 very small. The effect of that will be very small, as
10 I stated earlier.
11 And in addition, you know, if you look at
12 the ASCE's 4-86, some of this variation of soil
13 property from best estimate to lower bound to upper
14 bound with a large factor, a factor of two in modulus,
15 is partially to cover some of these uncertainties, I
16 would call, nonvertical propagating wave, some slide
17 possibility differentials, and many other uncertainties
18 which cannot be quantified.
19 So that variation of soil property from
20 lower to upper bound in the right range is partially
21 intended to cover some of this effect.
22 Q. Do you anticipate conducting any additional
23 analyses or calculations on the PFS project?
24 A. As far as pad design and analysis is
25 concerned, I do not foresee other than if we need to

1 answer certain questions, we may do a few calculations
2 to back up our response.
3 MS. NAKAHARA: If you'll excuse us for one
4 moment.
5 We have -- I have no more questions. Thank
6 you.
7 MR. GAUKLER: I may have a few questions.
8 We'll take a break.
9 (Recess from 1:58 to 2:00 p.m.)
10 EXAMINATION
11 BY MR. GAUKLER:
12 Q. I have a couple quick questions.
13 At one point in time in the questions asked
14 by Ms. Nakahara you were talking about Table D-1(d) on
15 sheet 234 of your calculation. Correct?
16 A. Yes.
17 Q. And that sheet shows vertical displacements?
18 A. (Witness nods head.)
19 Q. Are those vertical displacements all at one
20 time, or what are the vertical displacements displayed
21 in that table?
22 A. The vertical displacement displayed in this
23 table for various nodal points are maximum for the
24 individual nodes. They may or may not be at the same
25 time. So the number in each of these loads would show

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	)	ASLPB No. 97-732-02-ISFSI
PRIVATE FUEL STORAGE L.L.C.	)	
	)	DEPOSITION OF:
(Private Fuel Storage Facility)	)	**PAUL J. TRUDEAU**
	)	(Utah Contention L/QQ)

March 6, 2002 - 1:06 p.m.

Location: Office of the Attorney General
160 East 300 South, 5th Floor
Salt Lake City, Utah

Reporter: Susette M. Snider, RPR, CRR
Notary Public in and for the State of Utah

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A. **We would need to see the calc to find out for sure.**
Q. Okay. That's just fine.
A. **I could hazard a guess.**
Q. Sure, yes.
MR. TRAVIESO-DIAZ: Do not guess.
MS. CHANCELLOR: Yes, we'd love you to guess.
MR. TRAVIESO-DIAZ: Instructing the witness not to guess.
Q. (By Ms. Chancellor) Okay. I'll get those settlement calculations during the next break, and we can go over that.
In your opinion will the dynamic forces imparted to the foundation and foundation soils include bending and torsional stresses?
MR. TRAVIESO-DIAZ: Do you understand that question?
THE WITNESS: This is with respect to the Canister Transfer Building foundation and the pads, I assume.
Q. (By Ms. Chancellor) That's correct. The foundations that we're talking about in this contention. So if I mention foundations, unless I'm specific, it's either the pads --

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A. **And the question is will those foundations be exposed to bending stresses due to the earthquake.**
Q. Bending and torsional stresses due to the dynamic forces from an earthquake, yes.
A. **In my opinion, that's likely, yes.**
Q. And have you performed calculations to determine the magnitude and orientation of these stresses?
A. **I have no need to do that. I'm not designing those structures. The structural designer needs to take those into consideration.**
Q. And is that the ICEC calcs?
A. **Yes, for the pads.**
Q. And what about the CTB?
A. **That would be one of Bruce Ebbeson's calcs, the structural -- the structural engineer in Cherry Hill.**
Q. Would that be the calculation, Development of Soil Impedance Function for the Canister Transfer Building, or Seismic Analysis of the CTB? There are two different calculations that Mr. Ebbeson was involved with --
A. **It's likely the latter.**
Q. The latter?
A. **I don't know for sure, but --**

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Q. Okay.
A. **-- it's one of the SC calcs, probably 4 or 5, but I'm not positive.**
Q. In your opinion will soil cement undergo bending and torsion also?
A. **Well, I believe it will be subjected to bending stresses. I'm not sure about the torsional stresses.**
Q. And why don't you think it will undergo any torsional stress?
A. **My opinion.**
Q. The soil cement testing program, will that be looking at all at bending stresses?
A. **No, not that I -- I don't consider that the bending stresses and the soil cement surrounding the pads are of any interest because that soil cement around the pads doesn't do anything for our design basis except enhance the sliding capability.**
Q. What about under the pads?
A. **Under the pads? Under the pads the material's going to be bonded to the concrete so --**
Q. Provided that your soil cement testing program shows that you can do that; is that correct?
A. **Correct.**
Q. So you're not going to look at the bending

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or torsional stresses in the cement-treated soil under the pad?
A. **Correct.**
(A discussion was held off the record.)
Q. (By Ms. Chancellor) Will there be bending and/or torsional stresses in the soil cement around the CTB?
A. **Well, I believe there will be bending for sure. Again, not convinced where the torsional stresses are going to come from for that plate-like material around the building.**
Q. And what effect, if any, will these bending stresses have on the passive resistance of the soil cement around the CTB?
A. **I don't think they'll have any effect on it.**
Q. Why not?
A. **Why would they?**
Q. I'm asking you. You don't get to ask questions.
MR. TRAVIESO-DIAZ: Well, I believe he said --
MS. CHANCELLOR: I mean is there a basis for his opinion?
Q. I mean is it just your gut feeling that

Pg. 40-41

that's what's going to happen or is there a basis for your opinion?

A. If it -- if it bends in excess of the amount that it can tolerate, then it will crack, and if it cracks, it will be a vertical crack in response to this bending. As the waves pass through this material, if it cracks, it -- it's really not going to crack it, I don't think. It's going to end up opening an existing shrinkage crack. And when the wave goes by, the crack will be closed up again when the wave -- you know, when it's on the downside of it, it's going to close back up, and then when the waves fully pass, you're going to end up with the same kind of shrinkage crack you had when you began.

Now, the passive resistance is not diminished by the presence of a crack. It just means that the building needs to strain a little -- displace a little bit further to close up that little crack before you get the full resistance again. So I don't think that this bending stress issue is a concern for the soil cement surrounding the CTB.

Q. Well, how much can the CTB slide?

A. There is absolutely nothing safety related connected to this building, so it could slide tens of feet and it wouldn't affect any safety-related systems.

But certainly a few inches or even feet is not going to cause problems for safety of on-site or off-site personnel or the public.

This is -- this is a little different from normal nuclear power plant structures. On normal nuclear power plants, you have typically very Category I piping systems that you have to worry about being severed during an earthquake. So if the building were to slide, in a situation like that, you'd have to make sure that your connections were designed to sustain those kinds of movement. Here we don't have any Category I piping connections. We don't have any Category I electrical connections. We don't have any Category I gas lines or fuel lines or -- there's nothing Category I connected to this building, so if it were to slide a few inches it's not of any consequence to any safety-related thing.

(A discussion was held off the record.)

Q. (By Ms. Chancellor) In your opinion, is it possible that tensile strength of soil cement may be considerably less than that determined in unconfined or triaxial compression?

A. Would you please repeat the question?

Q. In your opinion, is it possible that tensile strength of soil cement may be considerably

less than that determined in unconfined or triaxial compression?

A. Yes.

MR. TRAVIESO-DIAZ: Sorry. I have problems with the form of the question. Two problems I have. First is what do you mean by tensile strength? You mean actual, in the actual condition, as opposed to test? That's my first question. And what do you mean by considerable? So I object to the form.

Q. (By Ms. Chancellor) Under test conditions -- well, just let's say less than rather than considerably.

A. Yes.

Q. How much? How much less?

A. I don't know.

Q. Let's see here. And when the state submitted its Contention Utah QQ, submitted a declaration by Dr. Mitchell, and in that declaration -- and I'll give you a copy of it, but let me just read it for the record. In paragraph 11 of that declaration, Dr. Mitchell states, The cement-treated soil will be subject to tensile stresses from static loading, from freeze/thaw and wet/dry, from shrinkage and from dynamic loading. The tensile strength of cement-treated soil is typically only about a fifth to

a third of the unconfined compressive strength, so even rather low tensile stresses can cause cracking.

Here's the -- here's paragraph 11 of Dr. Mitchell's declaration. And I'd like to ask you whether you have an opinion on whether this range of tensile strength is possible?

A. I have no reason to doubt Dr. Mitchell's statement.

MR. TRAVIESO-DIAZ: I'm sorry. What -- I should have spoken sooner. What range are you talking about here?

MS. CHANCELLOR: The three to five times less than the unconfined compressive strength.

MR. TRAVIESO-DIAZ: Is that what you understood the question to be?

MS. CHANCELLOR: To the yellow highlighted area.

THE WITNESS: Um-hum.

MS. CHANCELLOR: And I asked him whether he had an opinion on whether this range of tensile strength is possible, and he -- Mr. Trudeau answered that he had no reason to challenge Dr. Mitchell's assumption.

Q. And my next question is if you do have this tensile strength being three to five times less than

NRC Project on
Seismic Behavior of Spent Fuel Storage Cask Systems

Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage Facility

March 8, 2002

Vincent K. Luk
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David A. Aube

Nuclear Technology Programs Department
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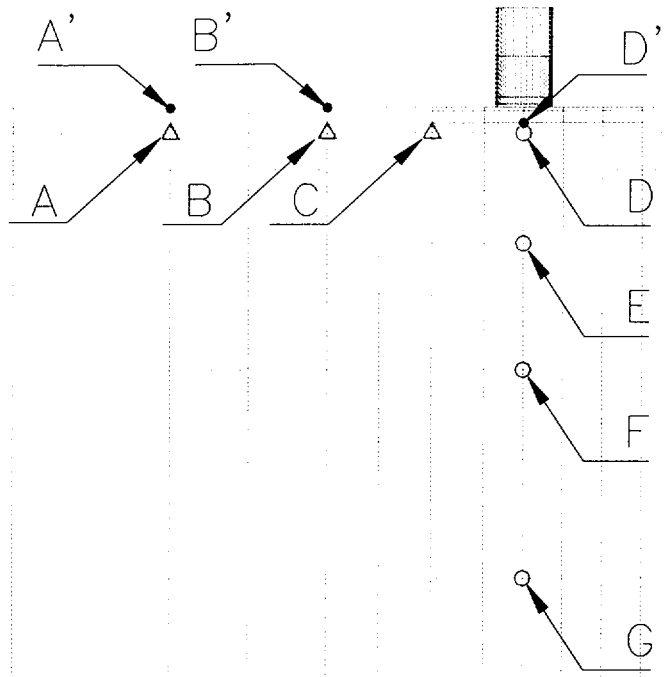


Designation	Description
A	Point at free field location (top of soil surface)
B	10' from edge of pad (top of soil surface)
C	Edge of pad (top of soil surface)
D	Center of pad (top of soil surface)
A'	Point at free field location (free surface)
B'	10' from edge of pad (free surface)
D'	Center of pad (top of soil-cement layer)
E	Depth of 12' below free surface
F	Depth of 26' below free surface
G	Depth of 50' below free surface

Figure 16. Location definition of designation points

NRC-02345

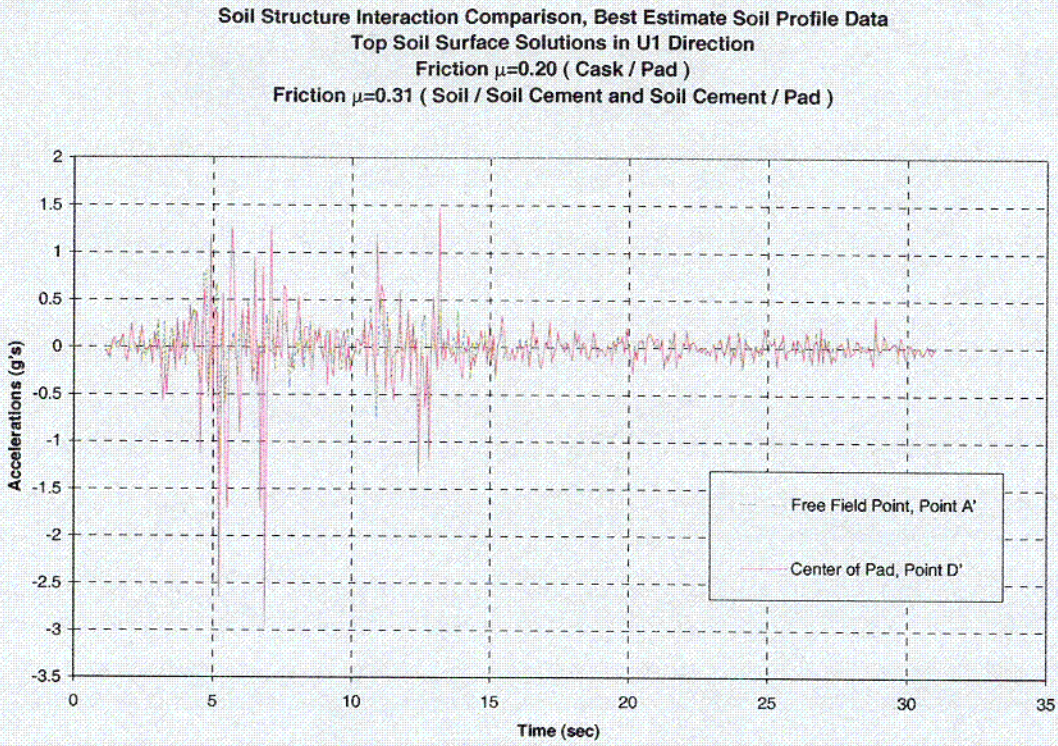


Figure 17. Time histories of accelerations in U1 direction at points A' and D' to demonstrate the SSI effect

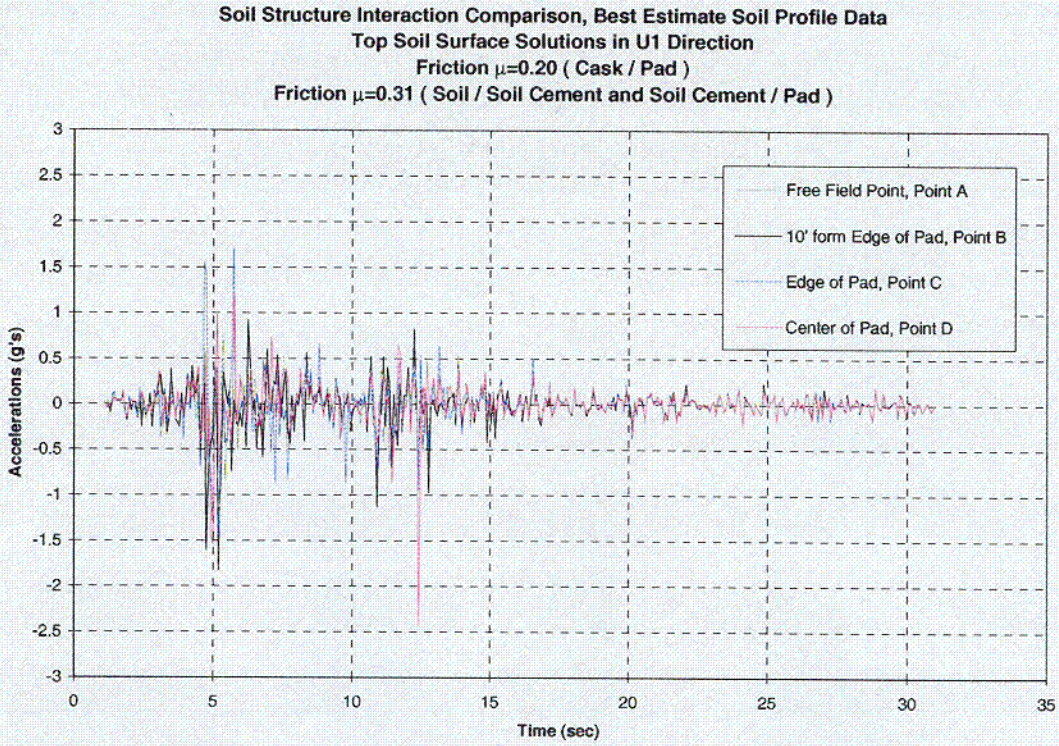


Figure 18. Time histories of U1 accelerations at four locations on top of soil surface to demonstrate the SSI effect

C01

NRC-02346

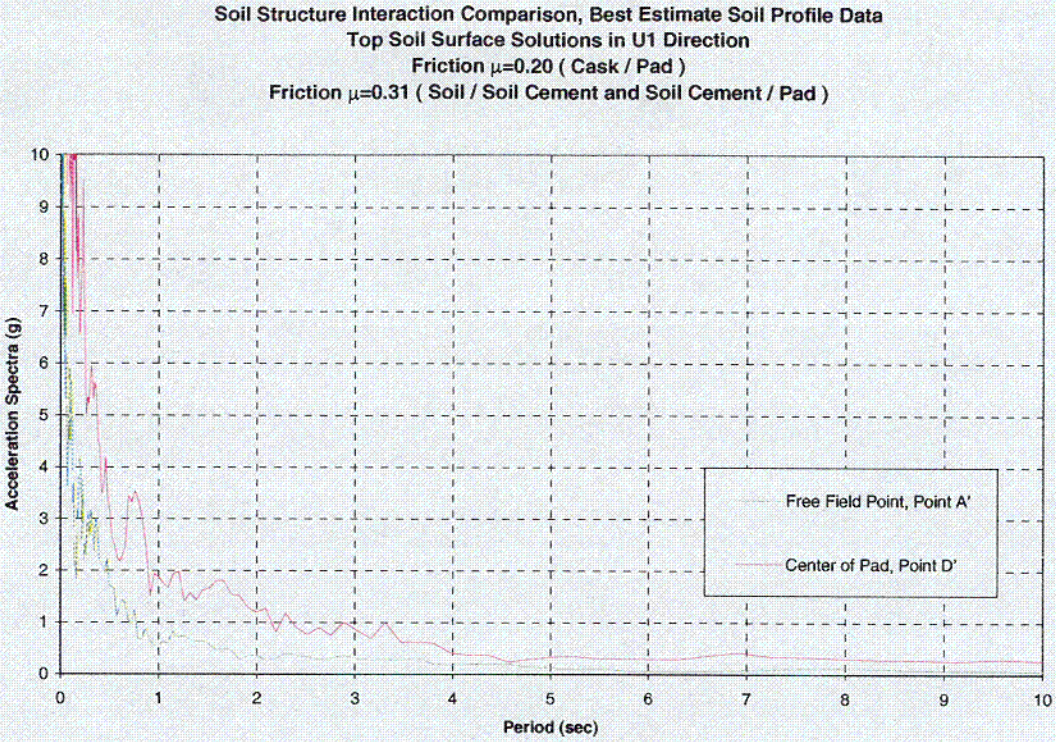
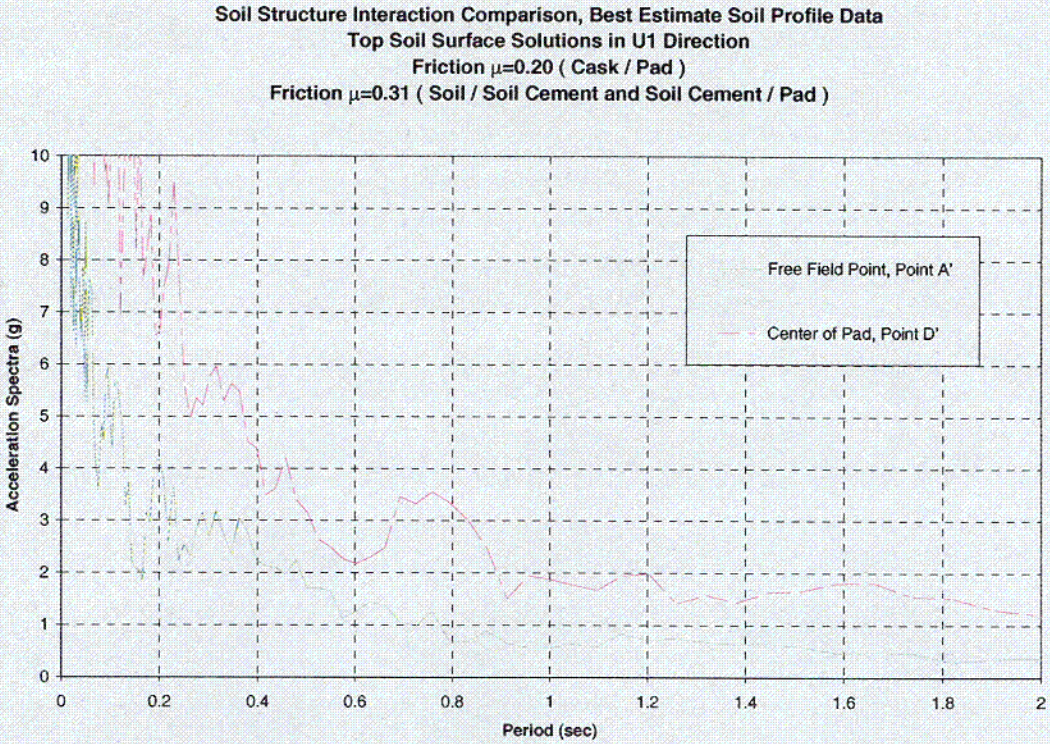


Figure 20a



C02

Figure 20b.

Response spectra in U1 direction at points A' and D' to demonstrate the SSI effect

NRC-02348

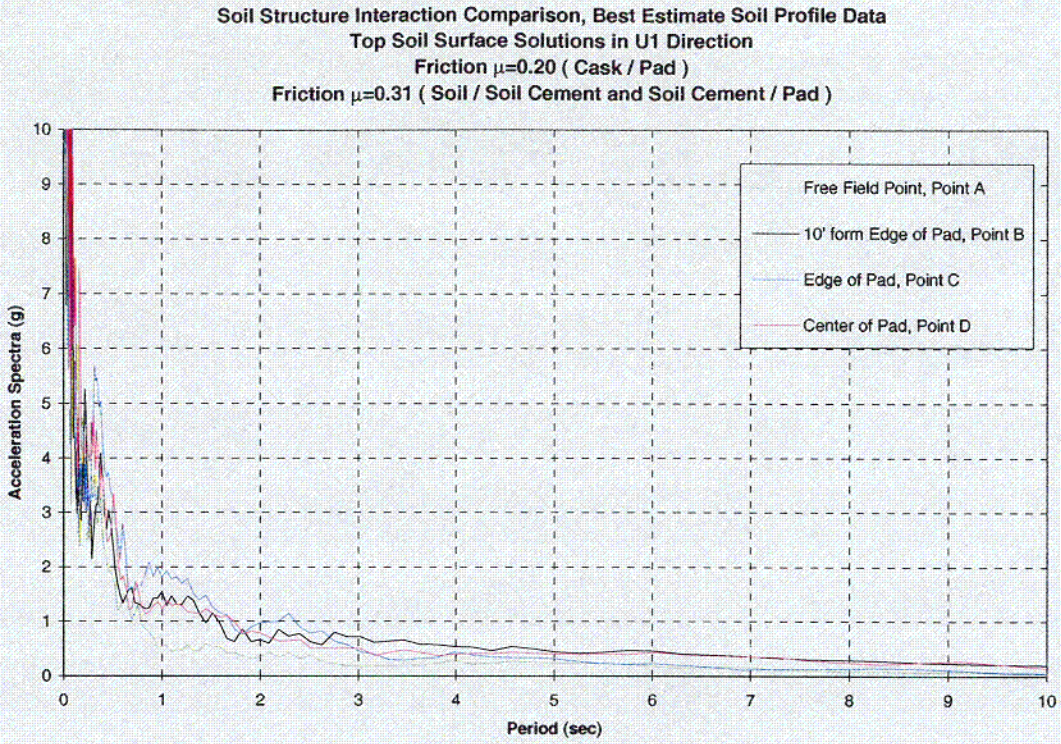


Figure 21a.

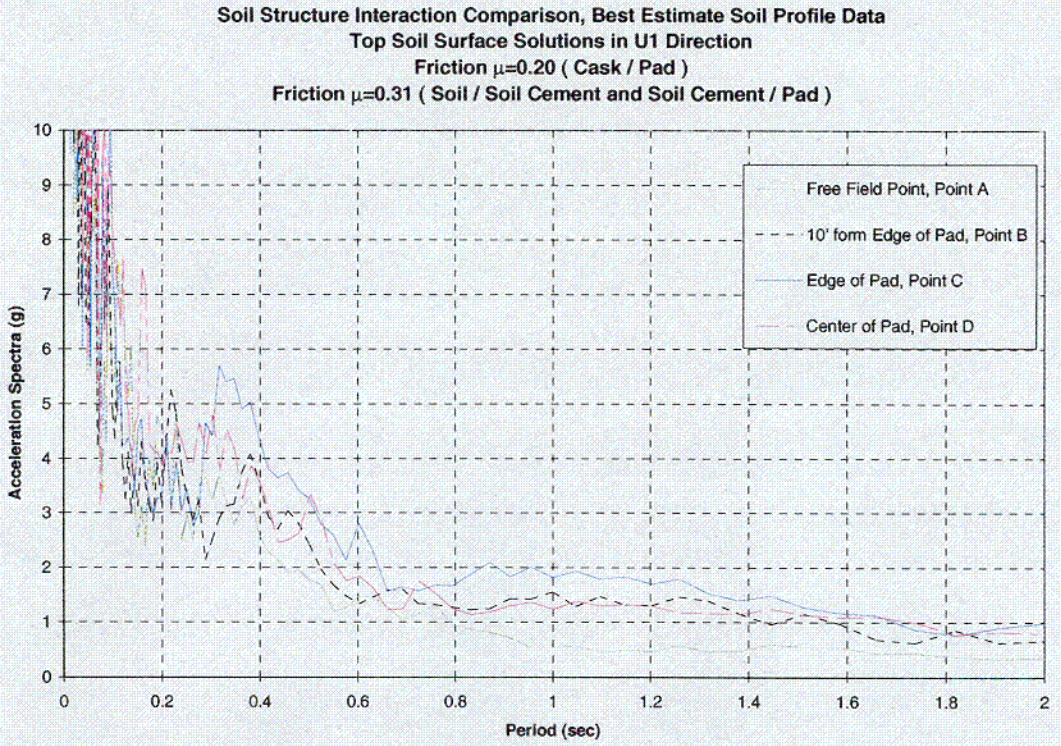


Figure 21b.

Response spectra in U1 direction at four locations on top of soil surface to demonstrate the SSI effect

C03

NRC-02349

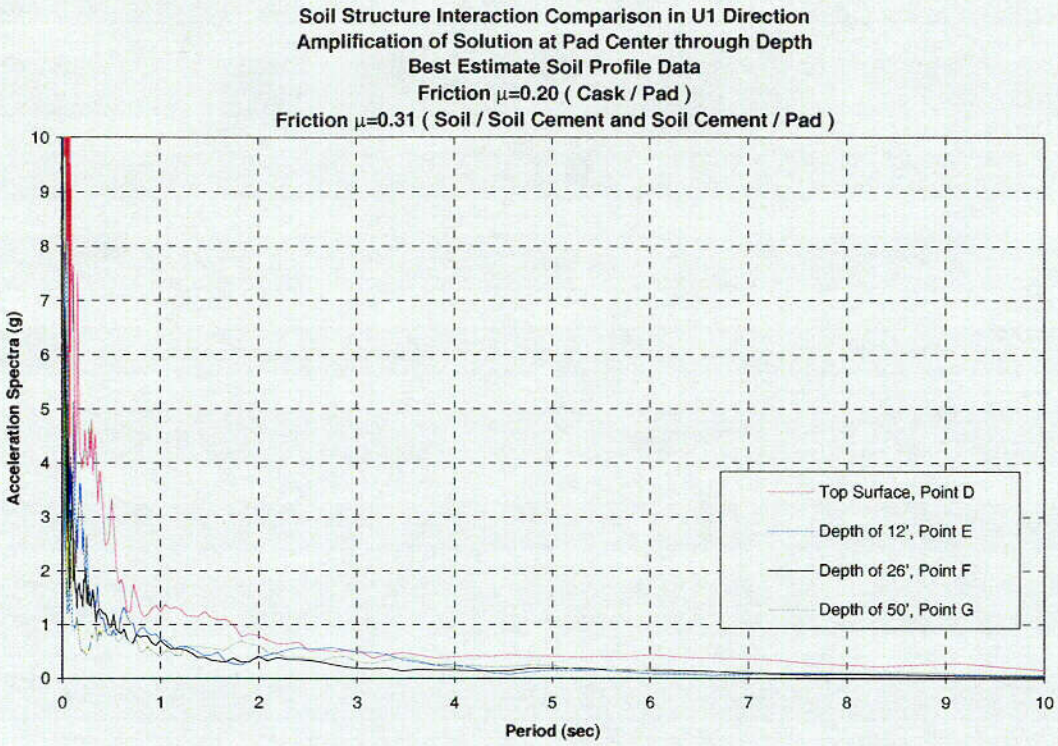


Figure 22a.

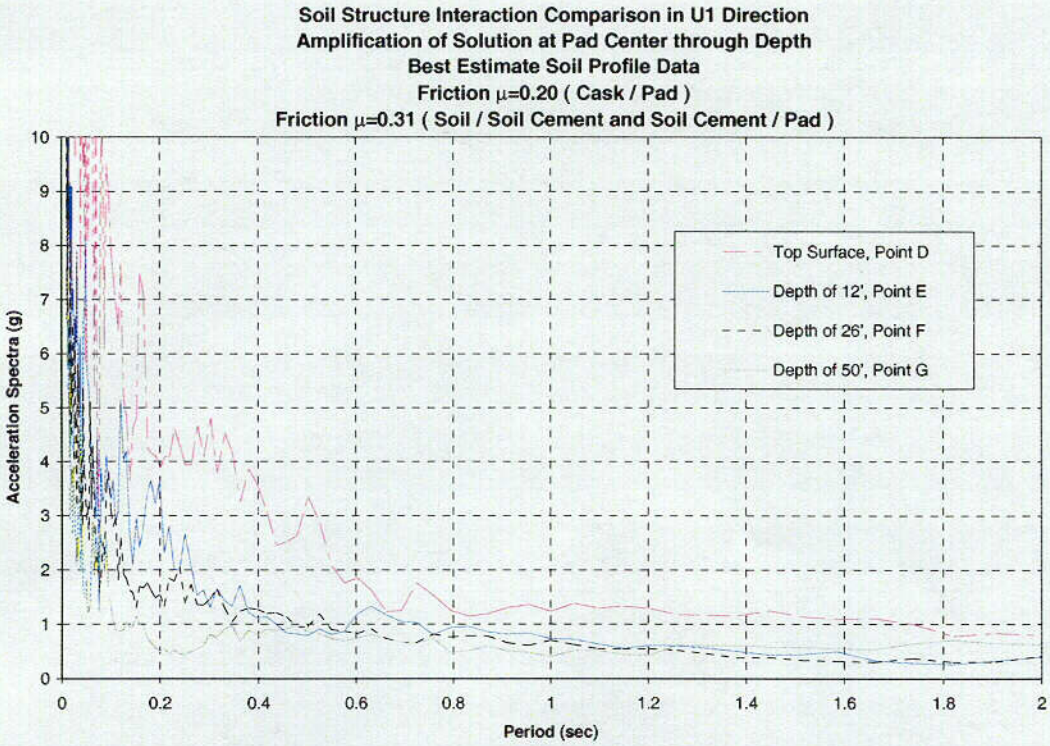


Figure 22b.

C04

Response spectra in U1 direction at four depths along the central axis of pad to demonstrate the amplification effect

NRC-02350

STONE & WEBSTER, INC.

5010.64

CALCULATION SHEET

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

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

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

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State's Exhibit 116

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

SLIDING STABILITY OF THE CASK STORAGE PADS

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

FS = resisting force ÷ driving force

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

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

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

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

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

B = 30 feet

L = 67 feet

DESIGN ISSUES RELATED TO SLIDING STABILITY OF THE CASK STORAGE PADS

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

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

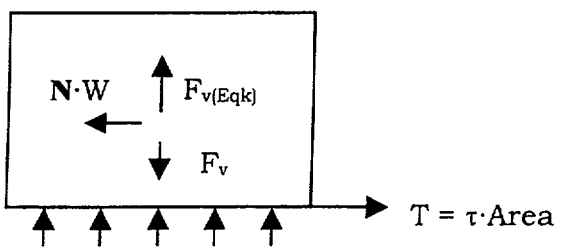
EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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

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

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

ESTIMATION OF HORIZONTAL DISPLACEMENT USING NEWMARK'S METHOD



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

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

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

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

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

$\sigma_n =$ Normal Stress

$\phi =$ Friction angle of cohesionless layer

$\sigma_n =$ Net Vertical Force/Area

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

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

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

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

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

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

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

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

MAXIMUM GROUND MOTIONS

The maximum ground accelerations used to estimate displacements of the cask storage pads were those due to the PSHA 2,000-yr return period earthquake; i.e., a_H = 0.711g 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, $F_{v\,Eqk} = a_V \times W/g = 0.695g \times 3{,}757\ K/g = 2{,}611\ K$

$\phi = 30°$

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

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

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

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

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

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 K x 0.40 = 1,044 K

ϕ = 30°

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

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

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

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

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

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

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

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

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

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

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

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

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with ϕ = 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.

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CALCULATION SHEET

CLIENT & PROJECT	PAGE 1 OF 59
PRIVATE FUEL STORAGE, LLC – PFSF	+ 6 pp of ATTACHMENTS
CALCULATION TITLE **STABILITY ANALYSES OF CANISTER TRANSFER BUILDING**	QA CATEGORY (✓) ☑ I NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ (other)

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

APPROVALS - SIGNATURE & DATE			REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED ☑	
PREPARER(S)/DATE(S)	REVIEWER(S)/DATES(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
Original Signed By: LPSingh / 12-9-98	Original Signed By: DLAloysius / 12-10-98	Original Signed By: DLAloysiu / 12-10-98	0	N/A		✓
Original Signed By: DLAloysius / 9-3-99 SYBoakye / 9-3-99 *See page 2-1 for ID of*	Original Signed By: SYBoakye / 9-3-99 DLAloysius / 9-3-99 *Prepared / Reviewed By*	Original Signed By: TYChang / 9-3-99 TYChang / 9-3-99	1	G(C)-13 Rev. 0		✓
Original Signed By: PJTrudeau / 1-21-00	Original Signed By: TYChang / 1-21-00	Original Signed By: TYChang / 1-21-00	2	1		✓
Original Signed By: PJTrudeau / 6-19-00	Original Signed By: TYChang / 6-19-00	Original Signed By: TYChang / 6-19-00	3	2		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	4	3		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	5	4		✓
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the state of stress existing under the Canister Transfer Building mat. Note, that the average post-peak strength reduction for normal stress of 1.5 ksf for the three direct shear tests is only 15.6% for these very high shear displacements in the direct shear tests. The maximum value of the average the post-peak strength reductions for normal stress of 1.5 ksf occurred for Sample U-3B&C in CTB-6, and it equaled 20.8%. If the results of this test were used to define the residual strength of these soils, the analyses would be performed at c = 1.5 ksf, the average of the post-peak strengths measured at the maximum shear displacements in these tests for normal stresses of 1 ksf and 2 ksf. This would result in higher factors of safety than are calculated and presented in Table 2.6-14, based on c = 1.36 ksf.

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

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

Average = 15.6

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

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Seismic Analysis of Safety-Related Nuclear Structures and Commentary

This document uses both Système International (SI) units and customary units.

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State's
Exhibit 118

Ex 34

Seismic Analysis of Safety-Related Nuclear Structures

1.0 GENERAL

1.1 INTRODUCTION

1.1.1 Purpose

This standard provides minimum requirements and acceptable methods for the seismic analyses of safety-related structures of a nuclear facility. This standard provides a methodology for calculating seismic responses in structures and to derive input motions for use in the seismic qualification of electrical and mechanical systems and components.

The purpose of the analytical methods is to provide only small levels of conservatism to account for uncertainties. The intentional conservatism is contained in the following three areas:

1. For soil-structure interaction, three cases are analyzed using different soil modulus values and the results use the envelope of the three cases.
2. For in-structure response spectra, the peaks are broadened.
3. For structural damping, conservative values are specified.

As a result, the output from the analyses using these methods will be at a slightly greater probability of non-exceedance than that of the input. For example, the seismic responses will have about a 90% chance of not being exceeded for an input response spectrum specified at the 84th percentile non-exceedance level.

1.1.2 Scope

1.1.2.1 Types of Structures Covered by This Standard

This standard is intended for use in the seismic analysis of all safety-related structures of nuclear facilities including, but not limited to, above and below ground structures, buried piping, above ground vertical tanks and structures with seismic isolation systems. Analysis of caisson and pile-supported foundations, unlined tunnels, and floating structures are not covered by this standard. However, nothing in this standard should be considered to preclude the use of these structures and structural elements.

1.1.2.2 Foundation Material Stability

The analysis procedures provided herein assume that the structures analyzed are adequately supported by their foundation materials and that no soil or rock failure occurs that would modify or void the seismic analysis.

1.1.3 General Requirements

1.1.3.1 Use of Analysis Results

The seismic responses determined from the analyses prescribed herein are to be combined with responses due to dead load and other prescribed loads.

1.1.3.2 Alternative Methodologies

Techniques other than those specified in this standard, including experience gained from past earthquakes, special analyses, and testing, may be used in lieu of the requirements specified herein. However, such alternative methodologies shall be properly substantiated and shall conform to the intent of this standard as expressed in the preface.

1.2 DEFINITIONS

The following terms are defined for general use in this standard. Specialized definitions also appear in some individual sections.

Apparent wave propagation velocity: The apparent propagation velocity of seismic waves through the ground relative to a fixed local coordinate system on the object analyzed.

Competent soil: Any natural or improved soil that has a shear wave velocity, $V_s \geq 1{,}000$ fps (300 m/s).

Coupled: A descriptive term for mathematical models of structures and components that are interconnected and which influence the dynamic response of each other.

"Cut-off" frequency: The highest frequency which is adequately represented in the model for the soil structure interaction analysis procedure. It may be taken as twice the highest dominant frequency of the coupled soil-structure system for the direction under consideration, but not less than 10 Hz.

Design (or evaluation) ground acceleration: The value of the acceleration which corresponds to acceleration at zero period in the design ground-response spectrum.

Design (or evaluation) response spectrum: A smooth response spectrum of the free-field input mo-

$\{\dot{X}\}$ = column vector of relative velocities ($n \times 1$);
$\{\ddot{X}\}$ = column vector of relative accelerations ($n \times 1$);
$\{U_b\}$ = influence vector; displacement vector of the structural system when the support undergoes a unit displacement in the direction of the earthquake motion ($n \times 1$);
n = number of dynamic degrees of freedom;
$\ddot{u}_g$ = ground acceleration.

(b) Eq. 3.2-1 may be solved using the modal superposition or direct integration time history methods.

3.2.2.2.1 Modal superposition

(a) The modal-superposition method may be used when the equations of motion (Eq. 3.2-1) can be decoupled using the transformation:

$$\{X\} = [\phi]\{Y\} \qquad \text{(Eq. 3.2-2)}$$

where

$[\phi]$ = normalized mode shape matrix; $[\phi]^T[M][\phi] = [I]$ [This is an ($m \times m$) identity matrix];
$\{Y\}$ = vector of normal, or generalized, coordinates ($m \times 1$);
m = number of modes considered.

(b) The transformation of Eq. 3.2-2 will decouple the equation of motion (Eq. 3.2-1) when terms like $\{\phi_i\}^T[C]\{\phi_j\}$, $i \neq j$, are small and can be neglected. This approximation is used in most practical cases including the structural systems with composite damping described in Sections 3.1.5.2 and 3.1.5.3. When experience shows that such an approximation is inappropriate, or a more accurate analysis is desired, a method which accounts for nonclassically damped systems may be used.

(c) The decoupled equation of motion for each mode may be written as:

$$\ddot{Y}_j + 2\lambda_j\omega_j\dot{Y}_j + \omega_j^2 Y_j = -\Gamma\ddot{u}_g \qquad \text{(Eq. 3.2-3)}$$

where

Y_j = generalized coordinate of jth mode;
λ_j = damping ratio for the jth mode expressed as fraction of critical damping;
ω_j = circular frequency of jth mode of the system (rad/s);
Γ_j = modal participation factor of the jth mode;

$$= \frac{\{\phi_j\}^T[M]\{U_b\}}{\{\phi_j\}^T[M]\{\phi_j\}} = \{\phi_j\}^T[M]\{U_b\} \qquad \text{(Eq. 3.2-4)}$$

(when mass normalized so denominator equals one).

The single-degree-of-freedom equations shall be integrated using a proven technique, such as those listed in Table 3.2-1.

(d) The techniques used for determining mode shapes and frequencies shall have convergence checks to ensure accuracy.

(e) It shall be sufficient to include all the modes in the analysis having frequencies less than the ZPA frequency, provided that the residual rigid response due to the missing mass is calculated from Eq. 3.2-5 and is combined algebraically with the response from Eqs. 3.2-2 and 3.2-3.

$$[K]\{X_o\} = -[M]\left\{\{U_b\} - \sum_{i=1}^{m} \Gamma_i\{\phi_i\}\right\}\ddot{u}_g \qquad \text{(Eq. 3.2-5)}$$

(f) Alternatively, the number of modes included in the analysis shall be sufficient to ensure that inclusion of all remaining modes does not result in more than 10% increase in total responses of interest.

3.2.2.2.2 Direct integration

(a) Direct integration of the equations of motion (Eq. 3.2-1) may be used. Either implicit or explicit methods of numerical integration may be used to solve the equations of motion.

3.2.2.3 Nonlinear Methods

(a) When performing a nonlinear analysis, the following shall be considered:

1. Geometric nonlinearities that significantly alter the effective system geometry, such as large displacements or significant gaps;
2. Material nonlinearities, such as plasticity or friction, in the range of response under consideration.

(b) The direct-integration and modal-superposition procedures (when appropriate) are acceptable methods to use for solution.

(c) Nonlinear analyses, shall, in general, consider all three components of earthquake motion, which shall be considered to act simultaneously unless it can be shown that individual component responses are uncoupled.

(d) In general, more than one set of acceleration time histories, meeting the requirements of Section

2.3, should be used, and the results of the analyses shall be averaged.

3.2.3 Response Spectrum Method

3.2.3.1 Linear Methods

(a) When the response spectrum method is used, the basic equations of motion given by Eq. 3.2-1 shall be uncoupled using the linear coordinate transformation of Eq. 3.2-2 and represented by the uncoupled, individual equation for each mode as given by Eq. 3.2-3.

(b) The generalized response of each mode shall be determined from:

$$Y_j(\max) = \Gamma_j \left(\frac{S_{aj}}{\omega_j^2}\right) \qquad \text{(Eq. 3.2-6)}$$

where S_{aj} is the spectral acceleration corresponding to frequency ω_j.

(c) The maximum displacement of node i relative to the base due to mode j is:

$$X_{ij}(\max) = \phi_{ij} Y_j(\max) \qquad \text{(Eq. 3.2-7)}$$

(d) In performing the calculations using Eqs. 3.2-6 and 3.2-7, and in calculation of the response quantities, the signs of the participation factor, Γ_j, the maximum generalized coordinate, $Y_j(\max)$, the maximum displacement of node i relative to the base due to mode j, $X_{ij}(\max)$, and other response quantities, shall be retained.

(e) Include all the modes in the analysis having frequencies less than the ZPA frequency or cutoff frequency, provided that the residual rigid response due to the missing mass calculated from Eq. 3.2-8 is added.

$$[K]\{X_o(\max)\} = M \times \left\{\{U_b\} - \sum_{i=1}^{m} \Gamma_i\{\phi_i\}\right\} S_{A\max} \qquad \text{(Eq. 3.2-8)}$$

where

$S_{A\max}$ = highest spectral acceleration in the interval between the cut-off frequency and ZPA.

Alternatively, the number of modes to be included in the analysis shall be determined as in Section 3.2.2.2.1(f).

(f) For modal combination purposes the residual rigid response $\{X_o(\max)\}$ shall be considered as an additional mode having a frequency equal to the ZPA or cutoff frequency.

(g) Individual modal and component responses shall be combined in accordance with the requirements of Section 3.2.7.

3.2.3.2 Nonlinear Methods

The response spectrum method cannot be applied in a rigorous manner to nonlinear multi degree-of-freedom systems because superposition of modes is no longer valid; however, there are approximate methods which may be used with adequate accuracy.

3.2.4 Complex Frequency Response Method

3.2.4.1 General Requirements

When the complex frequency response method is used for seismic time history analysis, the following requirements shall be met:

(a) The time interval for the input time history shall be chosen so that the maximum frequency of interest is retained.
(b) The frequency interval for calculation of transfer functions shall be selected to accurately define the transfer functions at structural frequencies.
(c) A quiet zone (trailing zeros) shall be added to the excitation time history. The quiet zone shall be long enough to damp out the transient response to ensure zero initial conditions.
(d) The transfer functions shall be established at a minimum 150 points in the 0 to ZPA frequency range unless the use of a lesser number of points or a lower upper frequency limit is justified.

3.2.4.2 Response Time History

When the complex frequency response method is used, the response time history, $R(t)$, may be expressed as:

$$R(t) = \frac{1}{2\pi}\int_{-\infty}^{+\infty} R(\omega)e^{i\omega t}\, d\omega \qquad \text{(Eq. 3.2-9)}$$

where $R(\omega)$ is the response in the frequency domain and is given by:

$$R(\omega) = T(\omega)\ddot{u}_g(\omega) \qquad \text{(Eq. 3.2-10)}$$

where

$T(\omega)$ = transfer function for the structure at circular frequency ω;
ω = circular frequency;

TABLE 3.3-1. Lumped Representation of Structure-Foundation Interaction at Surface for Circular Base

Motion	Equivalent Spring Constant	Equivalent Damping Coefficient
Horizontal	$k_x = \dfrac{32(1-\nu)GR}{7-8\nu}$	$c_x = 0.576 k_x R\sqrt{\rho/G}$
Rocking	$k_\psi = \dfrac{8GR^3}{3(1-\nu)}$	$c_\psi = \dfrac{0.30}{1+B_\psi} k_\psi R\sqrt{\rho/G}$
Vertical	$k_z = \dfrac{4GR}{1-\nu}$	$c_z = 0.85 k_z R\sqrt{\rho/G}$
Torsion	$k_t = 16GR^3/3$	$c_t = \dfrac{\sqrt{k_t I_t}}{1+2I_t/\rho R^5}$

Notes: ν = Poisson's ratio of foundation medium; G = shear modulus of foundation medium; R = radius of circular basemat; ρ = mass density of foundation medium; $B_\psi = 3(1-\nu)I_o/8\rho R^5$; I_o = total mass moment of inertia of structure and basemat about the rocking axis at the base; and I_t = polar mass moment of inertia of structure and basemat.

fixed base analysis of the flexible structure representation.

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. The consideration of an accidental eccentricity of 5% of the structure's plan dimension, as discussed in Section 3.1.1, will fully account for the torsional effects.

(b) Variation of amplitude and frequency content with depth may be considered for embedded structures.

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 o tion 3.3.1.9, secondary nonlinearities, including 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 Section 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 performed to establish seismic responses.

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 low strain soil shear modulus. Low strain 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. If sufficient, adequate soil investigation data are available, the mean and standard deviation of the low strain shear modulus shall be established for every soil

STATE OF UTAH's PREFACE TO TESTIMONY OF DR. MOHSIN R. KHAN AND DR. FARHANG OSTADAN ON CONTENTION UTAH L/QQ, Cask Stability Analysis

I. PFS's Unprecedented and Unconservative Design Requires Comprehensive and Qualitative Seismic Analysis.

A. PFS's design to allow the unanchored casks and pads to excessively slide and rotate in a seismically active area is unprecedented and unconventional.

B. It is contrary to good engineering practice to rely only on analytical analyses to justify PFS's unconventional design and not include redundancies in design.

C. Conservatism is critical with an unconventional design, thus, the seismic analysis/testing must account for all potential cumulative effects of ground motion that may amplify the movement of the casks and storage pads.

II. Portions of Cask Stability Independently Modeled.

A. To check the PFS cask stability analysis, Dr. Khan modeled aspects of the cask behavior subject to a 2,000-year DE at the PFS site.

1. Estimating the maximum, accumulative cask displacement was a key objective, so the model was designed to not allow the cask to over turn.
2. Cask displacement varies significantly with the contact stiffness value used.
3. Non-linear mathematical models predicting cask movement are highly sensitive to the contact stiffness parameter selected for the analysis.
4. The high stiffness values may cause the code to treat the model as linear.
5. In the cask model, high contact stiffness values artificially absorb energy and could reduce instantaneous velocities used in the next successive iteration in non-linear time history analysis thus, underestimating the cask displacement.
6. Cask displacement varies significantly with the amount of damping used.
7. Damping should be insignificant for a rigid cask because only friction should be the primary energy dissipation mechanism.
8. Using a range of reasonable contact stiffness values the relative cask displacements may be significantly higher than those reported in Holtec analyses of HI-STORM 100 cask system in all three directions of motion.
9. Based on the necessary velocity and uplift to over turn the cask, the HI-STORM 100 may tip over under a PFS 2,000-year DE.

III. PFS Seismic Analysis of HI-STORM 100 Unconservative and Inconclusive.

A. Based on Dr. Khan's analysis, PFS's cask stability analysis results are unreliable.

1. PFS used a high contact stiffness which may underestimate cask displacement.
2. PFS used 5% damping which may underestimate cask displacement.
3. Without benchmark test data, the PFS's results are questionable.

B. The lump mass mathematical code, Dynamo, used to model cask behavior for the 2,000-year DE, may generate invalid results.

1. Dynamo's limitation have not been quantified, however, Holtec suspects it is a small deformation code not capable of handling rotations in excess of 15 degrees. Holtec has not defined the basis for this 15 degrees.
2. Dynamo has not been previously used for equivalent or greater ground motion than PFS 2,000-year DE for cask analyses.
3. Dynamo has not been benchmarked against any full or bench scale data for ground motions greater than 0.4 g for sliding and uplift type of problems.

4. If cask rotations are too high, Dynamo would produce erroneous results which could appear to have physical meaning.
5. Dynamo was not used to predict cask behavior for PFS 10,000-year DE because Dynamo can not handle the expected cask rotations.

C. Must validate analytical seismic analysis displacements with shake table test data.

IV. Conclusion.

A. While Dr. Khan's results cannot be verified without benchmarking against shake table data they are sufficiently accurate to question the validity of PFS's analytical seismic results.

B. The questionable validity of Holtec's results support the Dynamic Analysis Testimony, that PFS has not demonstrated adequate factor of safety in its seismic analysis

C. Without an accurate cask behavior analysis it is impossible to estimate the magnitude and cumulative effect of PFS's failure to conservatively account for the flexibility of the storage pads; pad to pad interaction, and other inputs to its seismic analysis.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of:	)	Docket No. 72-22-ISFSI
PRIVATE FUEL STORAGE, LLC (Independent Spent Fuel Storage Installation)	)	ASLBP No. 97-732-02-ISFSI
	)	April 1, 2002

STATE TESTIMONY OF DR. MOHSIN R. KHAN AND DR. FARHANG OSTADAN ON UNIFIED CONTENTION UTAH L/QQ, PART D (Cask Stability)

I. Purpose of Testimony.

Q. 1: Dr. Ostadan, please state your name for the record.

A. 2: (FO) My name is Dr. Farhang Ostadan.

Q. 2: Dr. Khan, please state your name for the record.

A. 2: (MRK) My name is Dr. Mohsin R. Khan.

Q. 3: Dr. Ostadan, have you given other testimony describing your concerns with PFS's seismic analyses?

A. 3: Yes. In accompanying testimony jointly given with Dr. Steven Bartlett, I described PFS's inconsistent approach to analysis of the storage pads. *See* Dynamic Analysis Testimony. My testimony with Dr. Bartlett also addresses, to some extent, PFS's failure to consider non-vertically propagating waves, pad flexibility, cold bonding and to use multiple time histories.

The most serious concern I have is that PFS's design concept is unprecedented and unproven. I have discussed this at some length in my Dynamic Analysis testimony with Dr. Bartlett. Suffice it to say that there are no redundancy built in PFS's seismic design, which means that any misstep in PFS's seismic analyses could cause its design to fail.

Also, in testimony concurrently filed with Dr. Bartlett relating to Lack of Conservative Design, I discuss that the appropriate design basis earthquake must be inextricably linked to the performance of structures systems and components important to

safety at the PFS facility.

Q. 4: What is the purpose of your testimony?

A. 4: (MRK, FO) The purpose of our joint testimony is to explain the basis for our individual professional opinions that PFS's analysis of the stability of the storage casks are not performed in a comprehensive way to adequately demonstrate that the casks and storage pads will safely withstand a 2,000-year design basis earthquake ("DBE") at the PFS site. The assumptions and parameters used in PFS's cask stability analysis for the Holtec International Company ("Holtec"), HI-STORM 100 cask system are unconservative and may underestimate potential cask reaction under seismic ground motion at the PFS facility. Without an accurate cask behavior analysis, it is impossible to estimate the magnitude and cumulative effect of PFS's failure to conservatively account for the frequency dependency of spring and damping values; the flexibility of the storage pads; pad-to-pad interaction during seismic excitation and sliding; the variation in phasing of time histories in a non-linear analysis; the potential for cold bonding; and non-vertically propagating waves.

(MRK) To demonstrate the inadequacy of PFS's seismic analysis, I will testify that PFS's HI-STORM 100 cask stability analysis results have not been verified or benchmarked with shake table test data to determine their accuracy or usefulness under the high seismic environment. My independent analysis shows that the HI-STORM 100 cask system may excessively slide, uplift, and potentially tipover when subject to the ground motions for a 2,000-year DBE at the PFS facility

Q. 5: Explain why you are testifying together.

A. 5: (MRK, FO) We have different individual expertise with respect to seismic analysis. Our individual roles with respect to evaluating PFS's seismic analysis of the casks and storage pads are interconnected. Dr. Khan's cask stability and tipover analysis is a supporting subset of Dr. Ostadan's evaluation of the PFS seismic design and analysis. Our collective testimony comprised of our individual opinions will aid the Licensing Board in assessing this issue and make our testimony more cohesive and easier to understand.

II. Qualifications and Background.

Q. 6: Dr. Ostadan, have you previously provided your qualifications with respect to pre-filed testimony in support of this contention?

A. 6: Yes. Please refer to my Dynamic Analysis Testimony with Dr. Steven F. Bartlett. My curriculum vitae is included with that testimony as State Exh. 110.

Q. 7: Dr. Khan, please provide your qualifications and affiliation.

A. 7: (MRK) I hold a Ph.D. in solid mechanics (structures) from Clarkson College of Technology, Potsdam, N.Y., am currently employed by Altran Corporation, and am a consultant and registered professional engineer. My curriculum vitae listing my qualifications, experience, training, and publications is in State's Exhibit 119.

I have more than 22 years experience using response spectral data and finite element analysis to predict seismic performance of various structures, systems, and components. Throughout my career, I have used finite element models to analyze and predicte the seismic response of structures and equipment at nuclear facilities, including the analysis of free standing spent nuclear fuel racks and rigid blocks in a high seismic area. I have evaluated various seismic analyses of free standing dry storage casks for the high seismic areas. Additionally, I have performed analyses and seismic studies and review of numerous equipment and structures for several nuclear power plants.

Currently, I am the Engineering Manager of Altran's Structural Mechanics group in San Francisco, which is involved in equipment, piping, and structural analysis and design, including developing seismic specifications for testing and design review support. I also reviewed and developed a design basis criteria document for a dry cask ISFSI project in a high seismic area.

I have experience in linear and non-linear finite element analysis and design of structure, systems, or component performance under high seismic conditions. I have used structural analysis finite element codes such as STRUDL, ANSYS, BSAP, SAP90, and SAP2000 and in-house computer programs. Occasionally I have used the NASTRAN and STARDYNE computer programs. I am familiar with the use of AISC Code and ASME Code, Section III and VIII, and IEEE-344 Standard for dynamic qualification of equipment. I am currently a member the of IEEE-344 working group. As an example of my experience, I was involved in the review and development of finite element models to qualify equipment and structures at a nuclear power plant in a high seismic environment.

I have substantial experience designing shake table tests and interpreting and utilizing shake table data to calibrate and refine finite element models for equipment and structural designs. I have been involved in incorporating shake table data into probabilistic risk assessments and fragility evaluations.

When I was employed by Pacific Gas and Electric, I was a team member of the Diablo Canyon and Humbolt Bay Power Plants' Dry Cask Projects. I am familiar with Holtec International Inc.'s HI-STAR 100 and HI-STORM 100 design features and limitations. Further, I managed the equipment and structural seismic design and dynamic testing of many other projects when I was employed at Pacific Gas and Electric Company and Bechtel.

While employed at Clarkson College of Technology as an Assistant Professor, I

taught several graduate level courses, as well as developed special finite element computer programs, structural optimization programs, and kinematic analysis programs.

III. PFS Seismic Analysis of HI-STORM 100 Unconservative and Inconclusive.

Q. 8: Are you familiar with the HI-STORM 100 cask system proposed for use at the PFS facility?

A. 8: (MRK, FO) Yes, we have reviewed relevant documents describing the specification and performance of the HI-STORM 100 cask system.

(MRK) I also am familiar with the HI-STORM 100 cask system as a result of other work experience.

Q. 9: Dr. Khan, please describe how Holtec modeled the behavior of the cask.

A. 9: (MRK)

In an abundance of caution, the State if filing A.9 to my testimony as a proprietary filing in the event that Holtec claims any information therein as proprietary.

Q. 10: What mathematical code did Holtec use to perform its cask stability analysis?

A. 10: (MRK) In its cask stability analysis of the 2,000-year DBE at PFS, Holtec models the cask using beam finite elements with an in-house non-linear analysis program called Dynamo. Singh/Soler 2002 Tr. at 24-28 (State's Exh. 120); Singh/Soler 2001 Tr.[1] at 100. Holtec modified a published "general lump mass analysis" code to create an earlier version of Dynamo to analyze wet storage or spent nuclear fuel racks. Singh/Soler 2002 Tr. at 24-25. Holtec then modified the wet storage code for use in dry cask storage analysis. Id.

Q. 11: Please describe any limitations, if any, in using Dynamo to perform a cask stability analysis?

A. 11: (MRK) Dr. Soler stated in his deposition, Dynamo "does not alter the equilibrium equation step by step," but it "assumes to always satisfy equilibrium based on the original configuration." Id. at 43, State's Exh. 120. According to Dr. Soler, Dynamo is a

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[1] Excerpts from deposition transcript of Dr. Krishna P. Singh and Dr. Alan I. Soler (November 5-6, 2001) included as State's Exhibit 121.

"small deformation code" that is not currently capable of processing "large" cask rotations as a result of ground motion. Id. at 27.

Although Holtec expects Dynamo cannot handle rotations greater than 15 degrees, Holtec has not quantified the maximum peak ground accelerations or degree of rotations in which the results generated from Dynamo are valid. *See* Id. at 29-30. Other than at the PFS site, Holtec has not used Dynamo to perform a seismic analysis on unanchored casks with ground motion equal or greater than the PFS 2,000-year DBE. Holtec has only used Dynamo for three other unanchored, dry cask seismic analyses where the zero period acceleration was above 0.4 g but less than the PFS 2,000-year DBE. Id. at 19, 20, 29.

Holtec used Dynamo in a generic license with "seismic loads well in excess of" 0.5 or 0.6 g. Singh/Soler 2002 at 20. Additionally, Holtec used Dynamo to analyze the HI-STORM 100 cask response at Northwest Energy with zero period accelerations of about 0.5 g (id. at 18, 29) and Tennessee Valley with zero period accelerations about 0.5 to 0.6 g (id. at 29; Singh/Soler 2001 at 38 (State's Exh. 121). Furthermore, Dynamo's results for ground motions greater than 0.4 g have not been benchmarked against any full or bench scale data. Singh/Soler 2001 Tr. at 93-95, State's Exh. 121.

Q. 12: To your knowledge, has Holtec used a code other than Dynamo to analyze cask stability?

A. 12: (MRK) Yes, it has. Because Holtec expected Dynamo to produce "erroneous results" due to its inability to analyze "large" rotations of the cask, Holtec used VisualNastran for the dynamic analysis of casks under a 10,000-year earthquake at PFS. Id., Singh/Soler 2002 Tr. at 27-28. Although the potential cask rotations will be restricted because the casks will be anchored at Diablo Canyon (Singh/Soler 2001, Tr. at 35-36), Holtec relied on VisualNastran, not its in house code Dynamo, to conduct a seismic analysis for Diablo Canyon (Singh/Soler 2002 at 16-17, 28-29, State's Exh. 120).

Q. 13: What are the zero period accelerations at Diablo Canyon?

A. 13: (MRK) The zero period accelerations at Diablo Canyon are between 0.9 and 1 g. *See* Singh/Soler 2001 Tr. at 36, State's Exh. 121.

Q. 14: Generally describe your conclusions of Holtec's seismic analysis of the HI-STORM 100 cask system?

A. 14: (FO, MRK) The design and operational concept to allow unanchored spent nuclear fuel ("SNF") casks and storage pads to slide and rotate in a high seismically active area such as the PFS site is unprecedented and unconventional. In our joint opinion, it is not sound engineering practice to combine this highly unconventional practice of allowing SNF casks to slide without a sound technical justification and by using only analytical means to justify the design basis. Furthermore, the design has no redundancy, which means that

every component must perform its required function or the failure of a single component could cause the entire system to fail. If unanchored SNF casks are stored in high seismic areas, then it is critical that PFS conservatively account for the cumulative effects of potential ground motion in its design. However, PFS has not conducted a thorough analysis and testing to prove it has a conservative seismic design. Holtec's seismic analysis of the HI-STORM 100 cask system fails to thoroughly address potential conditions that may amplify the seismic response of the cask and storage pads. Additionally, Holtec's analysis uses input parameters which may significantly underestimate cask behavior.

(FO) In my Dynamic Analysis testimony, I provide a detailed discussion of how PFS failed to consider in its dynamic analysis and design the influence of the frequency dependency of spring and damping values, and pad-to-pad interaction, non-vertically propagating waves, multiple phasing of time histories, and cold bonding. *See* Bartlett and Ostadan Dynamic Analysis Testimony. Although PFS has failed to perform a qualitative analysis, PFS and the NRC Staff discount the individual effects of these dynamic analysis omissions and unconservative assumptions. As a result, Dr. Khan ran a number of cases to evaluate the performance of a simple model of the HI-STORM 100 cask at the PFS site to check portions of the PFS cask stability analysis.

Q. 15: Dr. Khan, what did the State of Utah ask you to do with respect to the PFS proposal?

A. 15: (MRK) I was hired to evaluate the Holtec's seismic cask stability (report numbers: HI-971631, HI-2012653, HI-2012640) results by independently modeling portions of the sliding and tipover phenomenon of the HI-STORM 100 cask under seismic motion for a 2,000 year earthquake at the PFS site. *See Analytical Study of HI-STORM 100 Cask System Under High Seismic Condition*, Technical Report No. 01141-TR-000, Revision 0 (Dec. 11, 2001) ("Altran Report"), State's Exhibit 122.

Q. 16: What general conclusions did you reach from your analysis?

A. 16: (MRK) As a result of my analysis, I determined that cask movement predicted by mathematical models are highly sensitive to the assumed contact stiffness between the cask and the pad. Barring any test data to validate the mathematical model results, Holtec's cask stability analysis results are questionable. Further, contrary to Holtec's conclusions, my analysis indicates that the HI-STORM 100 cask system may significantly uplift and slide. Further, because of the large displacements and high velocities observed for a few cases, the HI-STORM 100 may tipover under 2,000-year DBE ground motions at the PFS site.

Q. 17: Dr. Ostadan, what did you conclude from Dr. Khan's analysis?

A. 17: (FO) Because Dr. Khan determined that Holtec's cask stability analysis does

not provide unique results and the HI-STORM 100 may excessively slide and uplift, the cumulative effect of these unconservative omissions and assumptions discussed in the Dynamic Analysis Testimony cannot even be qualitatively estimated. Due to the small design margins and lack of redundancy, it is critical that PFS thoroughly and quantitatively address all potential conditions.

A. Independent Seismic Analysis of Cask Behavior Shows Cask May Tip Over.

Q. 18: Dr. Khan, was the objective of your analysis to perform a thorough seismic analysis that could be used for design purposes?

A. 18: (MRK) No. My understanding is that the State is not responsible for the design of the PFS facility. However, in the absence of any verifying test data or analysis at the time, I was asked to model the cask seismic reaction in a manner that would allow portions of Holtec's results to be checked for reasonableness.

Q. 19: Are you familiar with NRC quality assurance requirements for nuclear projects?

A. 19: (MRK) Yes.

Q. 20: Although your seismic analysis was not for design purposes, did you comply with NRC quality assurance requirements?

A. 20: (MRK) Yes, I conducted my analysis under Altran's quality assurance program for nuclear projects, which was developed in accordance with the requirements of 10 CFR Part 50, Appendix B, and was approved by NRC.

Q. 21: Dr. Khan, please describe how you conducted your seismic analysis of the HI-STORM 100?

A. 21: (MRK) To mathematically simulate the cask behavior, I modeled the HI-STORM 100 cask as beam elements. The cask base is connected to the ISFSI pad using non-linear elements. The mathematical model predicts the movement of the cask under inertial forces that exceed the frictional force or μW, where μ is the coefficient of friction and W is the mass of the cask. My modeling was based on the same cask design parameters used by Holtec in its analysis. I used the SAP2000 structural analysis code to perform a non-linear time history analysis of the cask response. I designed the model to calculate the total relative displacement of the cask.

I conducted case studies using three mathematical cask models with varying degrees of complexity. To evaluate results obtained by Holtec, I varied some of its input parameters, such as contact stiffness, coefficient of friction, and damping.

The purpose of the first case was to obtain horizontal sliding displacements for the HI-STORM 100 cask without any vertical excitation in order to benchmark my results against another accepted industry standard structural analysis code, ANSYS, by comparing the maximum sliding displacement values and the nodal displacement time history traces.

The second case study included the effect of vertical excitation without any rocking effects due to cask height. In this second case, to evaluate the sensitivity of the dynamic analysis to various input parameters, I conducted eleven runs where I varied the contact stiffness and coefficient of friction.

In the third case study, I modeled a three-dimensional cask with vertical and horizontal rigid beam finite elements. The mass density of the vertical beam elements is adjusted to obtain approximately the weight of a fully loaded HI-STORM100 cask. In the third case, I conducted nine runs where I varied the contact stiffness, coefficient of friction, and damping.

A complete description of my analysis and my results are found in my report, *Analytical Study of HI-STORM 100 Cask System Under High Seismic Condition*, Technical Report No. 01141-TR-000, Revision 0 (Dec. 11, 2001), included as State's Exhibit 122.

Q. 22: Dr. Khan, please describe SAP2000, your experience using the codes, and the applications for the codes.

A. 22: (MRK) SAP2000 is lump mass mathematical code commonly used to perform seismic analysis in structural design. SAP2000 uses nonlinear spring elements that can accommodate large deformation or large rotational movements of the structure to predict cask displacement. For this evaluation, I chose SAP2000 because 1) it is a widely available code that others could check my results; 2) it has been benchmarked with known analytical solutions to provide adequate results for dynamic analyses, and 3) I have used it to obtain reliable seismic analysis results in the past.

Q. 23: Did you attempt to verify your results?

A. 23: (MRK) I did not have any actual test data (*e.g.*, shake table test data) to compare with my results. However, I compared my SAP2000 results with another general purpose structural analysis code, ANSYS, for a two dimensional model. Like SAP2000, ANSYS has been benchmarked with known analytical solutions to provide adequate results for dynamic analyses. ANSYS is an accepted and widely used structural analysis code in the engineering arena. It is a general purpose analysis code for solving a variety of engineering problems. ANSYS can perform fluid, thermal, electromagnetic, impact and seismic engineering design calculations.

The results for SAP2000 and ANSYS were identical, which validates the SAP2000 results. *See* Comparison of SAP2000 and ANSYS results in Table 1, State's Exh. 122.

Q. 24: Please describe the results of your analysis.

A. 24: (MRK) Excluding any rocking effects due to cask height, Case 2 estimated the accumulative displacements of the HI-STORM 100 cask for the PFS 2,000- year DBE over the 30 second duration of the time history. Case 2 results show that the horizontal displacement varies with the coefficient of friction *See* Table 2, State's Exh. 122. It is expected that the sliding or horizontal displacement will increase with lower coefficient of frictions.

Case 2 also shows that the relative cask displacement significantly varies with the contact stiffness. This variation shows that the HI-STORM 100 cask dynamic analysis is highly sensitive to the local contact stiffness values used as input in the mathematical model. Local contact stiffness is needed in a mathematical simulation before any sliding occurs. However, after sliding occurs, the horizontal displacement is a function of the inertial forces overcoming the coefficient of friction times the mass (μW). The sliding or horizontal displacement of a cask under seismic ground motion should not be very sensitive to the local contact stiffness values. Because the displacement is dependent upon the contact stiffness value used, the dynamic analysis does not give a unique solution and thus is not completely meaningful without any comparison with test data especially under a high seismic environment.

High contact stiffness values also absorb significant amounts of energy before sliding actually occurs. The high stiffness values could tend to reduce instantaneous velocities for the next successive iteration in non-linear time history analysis. The use of a high contact stiffness could underestimate the vertical displacement.

Case 3 estimated the cumulative displacements of the HI-STORM 100 cask for a 2,000-year DBE at the PFS site over the 30 second duration of the time history. The results for the 3 dimensional analysis, Case 3, show that displacement varies significantly with the damping value. In reality, the damping should be small or insignificant for a rigid cask, and only friction should be the primary energy dissipation mechanism. Once the cask starts lifting off the pad for a longer duration during an earthquake, the simultaneous application of horizontal acceleration motion tends to move the cask more in the horizontal direction because the effect of friction is non-existent during the cask uplift. These results are shown in Table 3, State's Exh. 122.

Q. 25: Did you also analyze the potential for cask overturning or tipover?

A. 25: (MRK) Yes. Because I did not design my model to show cask tipover, I conducted a separate analysis, in section 5 of the report, to evaluate cask tipover. *See* State's Exh. 122 at 15-16.

In my analysis, I calculated the minimum vertical uplift and the horizontal velocity required to tip over the HI-STORM 100 cask under seismic ground motions. The HI-STORM 100 cask will over turn if the instantaneous velocity is greater than 77.82 inches per second. In my opinion, there is potential for the HI-STORM 100 to tip over if subject to the 2,000 year design basis earthquake at the PFS site.

B. Validity of PFS's Seismic Analysis Results Are Questionable.

Q. 26: Dr. Khan, what do the results of your analysis reveal with respect to the Holtec seismic analyses?

A. 26: (MRK) Because Holtec has not determined the limits of its mathematical code Dynamo to handle cask rotation in a seismic analysis, it is unknown whether Dynamo generated accurate results in the *Multi Cask Response at the PFS ISFSI from 2000-Yr Seismic Event*. *See* A. 9-11 above. Based on previous results from Holtec's other seismic analyses, Holtec assumed that its analysis for the 2,000- year DBE would not show large rotations and that Dynamo would generate valid results. Singh/Soler 2002 Tr. at 27, State's Exh. 120. However, the zero period accelerations for the previous seismic analysis conducted with Holtec's code Dynamo were all less than the 2,000-year DBE at the PFS site. The projected cask rotation and displacements at other facilities with lower zero period accelerations than the PFS 2,000- year DBE do not ensure that Dynamo will generate accurate results in the PFS analysis.

Furthermore, Holtec has only conducted two other sets of seismic analyses for dry storage casks where the ground motion exceeds the ground motion for the PFS 2,000-year DBE. These two analyses were for the 10,000-year earthquake at PFS and for Diablo Canyon. To analyze the cask response for a 10,000-year earthquake at PFS, Holtec expected Dynamo to generate "erroneous" results because the ground motions would cause cask rotations greater than 15 degrees. Singh/Soler 2002 Tr. at 27-30, State's Exh. 120. In that case, Holtec used VisualNastran to analyze the cask response. Id. at 27. Similarly, Holtec used VisualNastran for its dry storage cask seismic analysis at Diablo Canyon. Id. at 29. Although the seismic analysis presumably modeled anchored dry storage casks at Diablo Canyon which would severely restrict any cask rotation, Holtec still relied upon VisualNastran instead of Dynamo.

Holtec cannot document how to determine from Dynamo's results whether the analyzed casks experienced rotations that Dynamo is not capable of processing and whether, notwithstanding the parameter inputs, Dynamo's results are in fact accurately processed. Holtec states "if you attempt to take a code that is written for small deflections and blindly

just apply it and get a result that would indicate large deflections, either your program will blow up on you or it will just give you ridiculously large results that have no physical meaning, or it will simply give you wrong results that you may think there's a physical meaning to it." Id. 43 (*emphasis added*), State's Exh. 120. Although, anchored casks will experience limited rotation at Diablo Canyon, Holtec did not conduct its analysis at Diablo Canyon with Dynamo because it was concerned with the accuracy of Dynamo. Holtec cannot define the rotational limits of Dynamo. Holtec has not validated its Dynamo results for the 2,000-year DBE at the PFS site with another structural analysis code such as VisualNastran or ANSYS. In my opinion, Holtec cannot document whether the 2,000-year analysis are in fact "wrong results" that they "think" have "physical meaning."

Q. 27: Do the results of Holtec's analyses give a unique solution?

A. 27: (MRK) No. Notwithstanding whether Dynamo could handle the rotations posed by the 2,000-year DBE at the PFS site, the results of the dynamic analysis are not unique and very dependent upon the contact stiffnesses and damping used. *See* A. 24 above. With respect to a storage cask, the contact stiffness is the amount that the cask/pad interface will deflect with the application of a unit load at any given time. Contact stiffness is a function of the boundary conditions and the structural property. The deflection behavior of a pad under a concentrated load or small contact surface area during a dynamic situation could vary significantly. This behavior is shown when the cask starts to rock or pivot.

Q. 28: Please explain Holtec's use of contact stiffness.

A. 28: (MRK) In its analysis, Holtec used a high local contact vertical stiffness of approximately 450 x 106 lbs/in. *See* Singh/Soler Tr. 2002 at 81, State's Exh. 120. High contact stiffness values are generally used to start the mathematical simulation. However, Holtec's contact stiffness of 450 x 106 lbs/inch is too high for an unanchored cask subject to high peak ground acceleration because the contact stiffness makes the vertical frequency of the cask too rigid, thus artificially reducing the vertical displacement. Once sliding begins, the high stiffness values artificially treat the solution as linear without amplifying it in the upward direction and give non-unique or invalid results.

High contact stiffness values also absorb significant amounts of energy before sliding actually occurs. Holtec's use of an initial high local contact vertical stiffness significantly minimizes vertical excitation. It is my opinion that the high values of vertical contact stiffness used by Holtec also artificially and significantly reduce the estimated horizontal sliding displacements.

Q. 29: Is Holtec's model sensitive to contact stiffness values?

A. 29: (MRK) Yes. The sensitivity of a non-linear finite element model, such as Holtec's, to the local vertical stiffness values demonstrates that Holtec's model does not

produce a unique or conservative solution in this case. Holtec's contact stiffness of 450 x 10^6 pounds per inch may be adequate in areas with low seismic activity because the relatively small ground motion will not move the cask and the cask will behave as if anchored. However, the cask are unanchored at the PFS site.

In my opinion, the cask stability results from Holtec's non-linear model stated in HI-2012653 and HI-951312 are not conservative and would lead to an unconservative pad design.

Q. 30: Did Holtec use damping coefficients, and if so, how?

A. 30: (MRK) Holtec assumed BETA damping coefficients corresponding to 5% damping. Singh/Soler Tr. 2001 at 100, State's Exh. 121. My results of Case 3 show that 5 % damping values significantly reduce the estimated cask response,[2] whereas in reality the BETA damping would be small or insignificant for a rigid cask, and only friction should be the primary energy dissipation mechanism. Holtec's use of high BETA damping coefficients also underestimates potential sliding and rocking displacements that may lead to cask collision or tipover during a seismic event.

Q. 31: What would be a reasonable initial contact stiffness to use in the dynamic analysis of the HI-STORM 100 cask?

A. 31: (MRK) The contact stiffness changes with time as the contact between the cask and the pad change with time. A contact stiffness that correlates to frequency ($f = ½\ \theta\pi\ (K/M)^{0.5}$, where f = frequency, K = contact stiffness, M = mass) that falls within the amplified range of the response spectral curve is necessary for the cask to experience any potential movement. This ensures that one has considered all the frequency ranges within the response spectra. If the contact stiffness used is not in the amplified region and is selected so high that the structural code treats the problem as linear, then the code treats the cask as if anchored to the pad. If the contact stiffness is high, it makes the vertical frequency of the cask too rigid to respond to earthquake ground motion. As a result, a contact stiffness that is too high significantly underestimates the response of the cask.

Contrary to PFS's witnesses' opinion, a simple deflection calculation cannot be used to determine the appropriate contact stiffness to use in the dynamic analysis where the cask can potentially rock and uplift. The structural displacement obtained by solving a set of equations of motion could give displacements significantly different than those calculated due to resonance effects. Additionally, this simple deflection formula considers the force or weight of the cask to be acting over the cask base surface area. The deflection behavior of a

[2] *See* Table 3, State's Exh. 122.

pad under a concentrated or small contact surface area during a dynamic situation could vary significantly.

Q. 32: Is it possible to choose a contact stiffness for a non-linear dynamic analysis, such as the cask stability analysis?

A. 32: (MRK) It is essentially impossible to pick the contact stiffness for a non-linear dynamic analysis; therefore, a range of possible contact stiffnesses should be evaluated. The results should be validated with test results.

It is my opinion that a more appropriate contact stiffness value for unanchored casks could be in the range of 1×10^6 lbs/inch to 10×10^6 pounds per inch. This range of stiffness values would correspond to cask frequencies that fall in the amplified spectral range of the input spectra.

Q. 33: In your opinion, would the HI-STORM 100 cask tip over under a 2,000 year design basis earthquake?

A. 33: (MRK) In order to determine the total relative displacement and velocities posed on the cask, I did not design the model to show cask tip over. However, my analysis shows a wide range of relative displacement up to 372.76 inches in the x direction, 229.65 in the y direction, and 27.24 inches in the vertical direction. *See* Table 3 at 13, State's Exh. 122. My analytical study showed that the cask could move excessively and can have velocity greater than what it would take to tip over. With these numerical simulation results, it is possible that a cask could potentially tip over under the 2,000-year DBE at the PFS site.

Q. 34: How would you validate the results of a seismic analysis?

A. 34: (MRK) In my opinion, the only way to validate a seismic analysis in this case 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 have performed shake table tests to benchmark mathematical models. In fact, NRC requires shake table tests in IEEE Std 344-1987 (Institute of Electrical and Electronics Engineers, Inc. *Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations*) for many components and equipment, *e.g.*, electrical pumps. Shake table testing is required to demonstrate the structural integrity and functionality of equipment. It is also used for initial correlation with mathematical models to predict accurate dynamic responses.

Q. 35: What does IEEE Std 344-1987 require?

A. 35: (MRK) NRC Reg. Guide 1.100, Rev. 2, *Seismic Qualification of Electric and Mechanical Equipment for Nuclear Power Plants* endorses use of the IEEE standard 344-1987

(Revision of ANSI/IEEE Std 344-75) for equipment qualification. The introductory note of Section 6, Analysis, of IEEE-344-87 states that "[t]he analysis method is not recommended for complex equipment that cannot be modeled to adequately predict its response. Analysis without testing may be acceptable only if structural integrity alone can ensure the design-intended function." Section 5.3.1 states that "[i]n analysis, a mathematical model is made of the equipment so as to predict the response to the seismic motion. The damping used in this model should correspond to the actual energy dissipation in the equipment to enable the response to be accurately predicted. An alternate approach is to use a conservative value of linear damping to obtain a conservative estimate of response. In any case, there is a need to know the ranges of damping for the specific equipment and the nature of nonlinearities and their effect on the response. Appropriate values of damping may be obtained from tests or other justifiable sources."

Q. 36: Have the HI-STORM 100 cask stability results for PFS been benchmarked with another computer code?

A. 36: (MRK) No. The results of a cask stability analysis using Holtec's mathematical model have not been compared to other computer codes. Singh/Soler Tr. 2001, State's Exh. 121 at 93-94. PFS's witness stated that Holtec's mathematical model was compared with other programs to support wet storage applications. Id.

Q. 37: Dr. Ostadan, what are the limitations of Dr. Khan's cask stability analysis?

A. 37: (FO) As Dr. Khan explained in Answers 15 and 18 above, the intent of his analysis was not to conduct a thorough seismic analysis that could be used for design. The intent of his analysis was to check the validity of Holtec's results. Dr. Khan acknowledges that the accuracy of his results are subject to confirmatory test data. However, Dr. Khan's analysis is sufficient to question the accuracy of Holtec's results, which meets his intended goal to confirm or dispute Holtec's analysis.

As Dr. Khan acknowledges, he did not consider factors that may in fact increase the ground motion and subsequent movement of the casks. For example, Dr. Khan used the same input time histories as Holtec; thus, he did not consider the effects of phasing by using multiple sets of time histories as is the standard engineering practice. *See* State's Exh. 122 at 19.

Q. 38: Dr. Ostadan, do Dr. Khan's conclusions aid you in your evaluation of PFS's dynamic analysis?

A. 38: (FO) Yes. The complex nature of Holtec's seismic analysis of the casks with the limited information offered in the Holtec report supports my opinion described in my Dynamic Analysis testimony, that PFS has not evaluated the cumulative effect of the

applicable range of input parameters in evaluating the stability of the casks. Without a thorough analysis and testing that adequately challenge the unique and unproven features in the PFS design, the stability of the free standing casks on the pad is not proven.

Q. 39: Does this conclude your testimony?

A. 39: (MRK, FO) Yes.

altran

Dr. Mohsin R. Khan

EDUCATION: B.S., Mathematics, Christ Church College, Kanpur, India (1970)
B.S., Mechanical Engineering, Harcourt Butler Technological Institute, Kanpur, India (1974)
M.S., Mechanical Engineering, Clarkson College of Tech., USA (1975)
Ph.D. Solid Mechanics (Structures), Clarkson College of Tech., USA (1978)

REGISTRATION/MEMBERSHIP:

Professional Mechanical Engineer - State of California
American Society of Mechanical Engineer
IEEE-344 Committee (Current Member)
EPRI- STERI (1995-96)

HONORS: Award of Merit from Bechtel Power Corporation
Received several Performance Recognition awards at Pacific Gas & Electric Company

EXPERIENCE: Altran Corporation, San Francisco – Engineering Manager

- Mohsin is the Manager of Altran's Structural Mechanics group in San Francisco. In his current assignments, his responsibilities include both providing technical and project management leadership. The Structural Mechanics group includes structural analysis and design, piping analysis and design, and equipment analysis and design and testing functions.

- Performed Structural Probabilistic Risk Assessment of MHM Crane for the Dry Cask Project. This is for the DOE's Hanford site in Washington.

- Providing equipment seismic specifications for testing and design review support for the CVDF (Cold Vacuum Drying Facility) equipment for the Dry Cask Project. This is also for the DOE's Hanford site in Washington.

- Provided design and analysis for the Indian Point 1 Power Plant's Spent Fuel Pool project. This involved in providing a steel liner and the leak chase system to the existing pool walls to protect it from leakage; designing a partition wall with a water tight steel gate and redesigning the new Fuel Racks. Detailed dynamic analyses of the Spent Fuel Pool Racks using ANSYS program were performed.

- Provided structural design reviews for the Humbolt Bay Power Plant (HBPP) Stack removal project. This involved in assessing the effect of soil pressure on the existing building structural walls due to placement of heavy crane loads during the stack removal.

- Provided design upgrade for the DCPP Refueling Water Storage Tank Project. This involved several dynamic analyses of various piping branch lines, several tank analyses, and structural support evaluations.

- Provided inspection procedures to comply with the ASME section IWE and IWL requirements and calculations for the steel liner and concrete wall threshold thicknesses for the DCPP containment.

- Provided engineering support and structural evaluation and design modification in support of DCPP Seismic Gap program.

- Developing a Design Basis Criteria Document for the HBPP Dry Cask/ISFSI project. This design basis criterion will be used by the HBPP for the bid evaluation to select a Dry Cask vendor for the decommissioning purposes.

- Development of the criteria and the specifications for dynamic shake table tests for the RVLIS system. This involved several cabinet finite element analyses and shaker table testing to meet the seismic design bases requirements for the DCPP site. This involved witnessing the shake table tests, identification of problems which develop during the shake table tests, participation in the resolution of them (e.g., design changes to strengthen the components), and interpretation and acceptance of the test results for documentation in engineering calculations.

Pacific Gas & Electric Company, San Francisco (Nuclear Power Generation)

- Principal Engineer, Company's Structural/Equipment Seismic, Vibration and Dynamic Analysis and design expert

- Supervise and provide technical guidance to a group of engineers in the area of vibration and seismic analysis of Mechanical, Electrical, Instrumentation and Controls, and Heating and Ventilation equipment for Diablo Canyon Nuclear Power Plant (DCPP).

- Developing Finite Element models of equipment and perform computer analyses to qualify the equipment for DCPP design bases seismic events. This involves reviewing the results of the analyses for completeness accuracy, validity and documenting the results in Design Calculations.

- Extensive use of computer programs - STRUDL, ANSYS, BSAP, SAP90,SAP2000 and several in-house programs. Occasional use of NASTRAN and STARDYNE.

- Development of the criteria and the specifications for dynamic shake table tests for instrumentation, electrical devices, and control panels/cabinets to assure that the tests will envelope the specific load conditions that the components would be subjected to during design bases seismic events at DCPP. This involves witnessing the shake table tests, identification of problems which develop during the shake table tests, participation in the resolution of them (e.g., design changes to strengthen the components), and interpretation and acceptance of the test results for documentation in engineering calculations.

- Provide extensive technical support around the clock including weekends to resolve construction constraints/problems during the design implementation and provide alternative design solutions acceptable to client (DCPP) without increasing the outage cost and duration

- Provide technical interface with Westinghouse and other vendors in the area of NSSS equipment, structural dynamic/seismic qualification. This requires monitoring the vendor's technical work, cost and also help in meeting the stringent project schedules.

- Provide technical support including seismic calculations and their interpretation to Diablo Canyon's Long Term Seismic Program consultants for Mechanical, Electrical, I&C, and HVAC equipment. This involves Probabilistic Risk Assessments and Fragility Evaluations.

- Performed structural dynamic analyses to qualify the Manipulator and the Spent Fuel Pool Bridge Cranes for DCPP units 1 and 2. This involved modifying the cranes for the upgrade pupose and to meet high seismic demand loads.

- Provide structural/seismic evaluation for the dedication activities for Replacement Parts Program. This is an extensively tedious and technically involved process and poses a constant challenge to accept commercially procured parts to be used at Diablo Canyon Nuclear Power plant which is in a very high seismic zone.

- Perform LOCA (Loss of Coolant Accident Analysis) and Seismic analyses for Containment Isolation Purge and Exhaust valves.

- Involved in review of Dynamic Qualification of High Density Spent Fuel Racks for Diablo units 1 and 2. Also performed an independent Finite Element analyses to check vendor's work during the licensing hearing process due to outside intervenors. Provided expert testimony on High Density Spent Fuel Rack design.

- Seismic evaluation of as-found conditions of equipment which deviate from the original design bases. For example: electrical cabinets and panels with loose and missing mounting hardware, and damaged equipment and equipment supports. These events are immediately reportable to the NRC if the as-found conditions compromise the public safety. Therefore, prompt resolution of these problems becomes extremely important. The resolution process often times requires unique dynamic non-linear analyses and obtaining test information from various sources.

- Team member of the Diablo Canyon and Humbolt Bay Power Plant's Dry Cask Project. He has unique technical and business experience in the site and vendor selection process for the Dry Cask project. He is familiar with all of the nation's leading Dry Cask vendors, their design features, and their limitations.

- Managed the equipment and structural seismic design and dynamic testing of many capital projects at DCPP. In addition to his work on Dry Casks, he was involved in the Underground Diesel Fuel Tank replacement project, 6th Diesel Generator addition project, Vital 4kv switcher replacement project, Inverter Replacement Project, 125V Battery Replacement Project, Electrical Relays Replacement Project, Various Pump-Motor Replacement Project, Main Annunciator Replacement Project, Main Control Board Modification Project due to Human Factor Effects, Post Accident Monitoring Equipment Replacement Project, and replacing the

buried piping for the ultimate heat sink. Each of these projects was in millions of dollar, and posed significant technical and licensing challenges.

BECHTEL POWER CORPORATION, San Francisco

- Susquehanna Nuclear Power Plant (Mark II) Design evaluation of Containment Building. Structural analysis of box beam platforms, pipe whip restraints, diaphragm slab, hangers, quenchers, etc., using STRUDL, BSAP, ANSYS and other in-house computer programs.

- Equipment (Dynamic) Qualification Group (EQG) Worked on Limerick, Susquehanna-Mark II, Hope Creek-Mark I, Diablo Canyon-PWR, Nuclear Power Plants.

- In charge of all computer activities (Mainframe/Micros) of the group.

- Developed a computer program "BSEIM" that generates artificial random time histories from a given floor response spectra for UNIVAC 1100 series mainframe computer.

- Extensive use of AISC Code and ASME Code. Section III and VIII, and IEEE-344-75 Standard for dynamic qualification of equipment.

- Use of Vibration Testing to qualify Electrical, HVAC and Mechanical Equipment such as cabinets, terminal blocks, valves, dampers, operators, and pumps, etc., per IEEE-344-75.

- Successfully defended NRC Audits on non-NSSS Mechanical Equipment for Diablo Canyon Unit 1 Project.

- Reviewed equipment dynamic qualification of Westinghouse (NSSS) supplied Mechanical Equipment for Diablo Canyon Project.

- Dynamic Analysis (Response Spectra and Time-History) and Static Analysis (Thermal Load, Dead Weight, Pressure, and Nozzle Loads) for a variety of Class I equipment, i.e., valves, component cooling water surge tanks (horizontal), boric acid tanks (vertical), safety injection pump and motor, auxiliary feedwater pumps and motors, auxiliary feedwater turbine, diesel generators and radiators, silencers, starting air receiver and turbo charger tanks, priming tanks, lube and diesel oil filters, strainers, containment fan cooler boxes, dampers, heat exchangers, electrical cabinets, radwaste compressors, coolers, and decay tanks, containment hydrogen purge exhaust and supply filters, steam generator blowdown tanks and coolers. Portable fire pumps, and condensers, etc.

- Extensive use of computer programs - STRUDL, ANSYS, BSAP, NASTRAN, STARDYNE, and in-house Bechtel programs.

- Involved in finite element model review for Seismic and Impact Analysis of Vacuum Breaker Valves for the Downcomers of Susquehanna, Limerick and Shoreham Nuclear Power Plants (Vendor - AGCO).

- Interaction with vendors and review of vendor analysis or test reports; review and writing of design or test specifications; coordination with the project and field about design modifications;

coordination with procurement in issuing purchase orders to outside vendors for analysis and testing.

- Supervised a group of 25 engineers; responsible for the review technical guidance and approval of calculations; manpower planning and man-hour estimates; scheduling and employee performance evaluations; finalizing the equipment qualification packages for NRC audit.

- Developed a General Finite Element Structural Analysis Code MICROSAC for structural analysis for Microcomputers (Static and Dynamic Analysis Capability).

Clarkson College of Technology – Assistant Professor

- Taught Particle Dynamics, Static, Strength of Materials, Machine Design as undergraduate courses.

- Taught Finite Element Methods, Theory of Elasticity and Structural Dynamics as graduate courses.

- Author of several papers in ASME, AISC, and AIAA (American Institute of Aeronautics and Astronautics) Journals.

- Developed several special finite element computer programs, structural optimization programs, kinematic analysis programs.

- In charge of all computer activities in the department of Civil and Environmental Engineering.

- Supervised three graduate students towards their M.S. thesis. Thesis topics involved development of Structural Optimization techniques to design Beams, Frames, Aircraft wing structures, Transmission Towers, etc., under stress, displacement, and natural frequency constraints. Also, a special finite element was developed to analyze horizontally curved beams under warping effects using flexibility method.

PUBLICATIONS:

KHAN, M. R. and WILLMERT, K. D., "Vibration Analysis of Mechanisms Using Constant Length Finite Elements," ASME, Paper No. 76-WA/DE-21, 1976.

KHAN, M. R. and WILLMERT, K. D., "General Kinematic Analysis Of Planar Mechanisms Consisting of Four Bar Chains and Slider Cranks," A User Manual, Department of Mechanical and Industrial Engineering, Clarkson College of Technology, Potsdam, N.Y., Report No. MIE-030.

KHAN, M. R. and WILLMERT, K. D., and THORNTON, W. A., Automated Analysis/Design of High Speed Planar Mechanisms," Proceedings of the 5th OSU Applied Mechanisms Conference, Oklahoma, pp. 21-1 to 21-9, November 1977.

KHAN, M. R., WILLMERT, K. D., and THORNTON, W. A., "A New Optimality Criterion Method for Large Scale Structures," Proceedings of the AIAA/ASME 19th Structures, Structural Dynamics and Materials Conference, pp. 47-58, April 1978.

WILLMERT, K. D., THORNTON, W. A., and KHAN, M. R., "A Hierarchy of Methods for Analysis of Elastic Mechanisms with Design Application," ASME Paper No. 78-DET-56, 1978.

THORNTON, W. A., WILLMERT, K. D. and KHAN, M. R.,"Mechanism Optimization Via Optimality Criterion Techniques," Trans. ASME, Journal of Mechanical Design, Vol. 101, No. 3, July 1979, pp. 392-397.

KHAN, M. R., THORNTON, W. A., and WILLMERT, K. D.,"Optimality Criterion Techniques for Structures with Multiple Design Variables per Member," Proceedings of the AIAA/ASME/ASCE/AHS 20th Structures, Structural Dynamics, and Materials Conference, pp. 87-95, April 1979.

KHAN, M. R., WILLMERT, K. D., and THORNTON, W. A., "An Optimality Criterion Method for Large Scale Structures," AIAA Journal Vol. 17, No. 7, pp. 753-761, July 1979.

KHAN, M. R., WILLMERT, K. D., and THORNTON, W. A., "A Computer Program Package for Large Scale Structural and High Speed Mechanism Design," Proceedings of the Engineering Software Conference at the University of Southhampton, England, September 1979.

SYED, M. I., WILLMERT, K. D., and KHAN, M. R., "Optimality Criteria Techniques Applied to Structures Composed of Different Element Types," Proceedings of the AIAA/ASME/ASCE/AHS 21st Structures, Structural Dynamics and Materials Conference, May 1980, pp. 345-351.

KHAN, M. R., "Optimality Criterion Techniques Applied to Frames Having Nonlinear Cross-Sectional Properties," Proceedings of the AIAA/ASME/ASCE/AHS 22nd Structures, Structural Dynamics, and Materials Conference, pp. 233-241, 1981.

KHAN, M. R., and WILLMERT, K. D., "An Efficient Optimality Criterion Method for Natural Frequency Constrained Structures," Journal of Computers and Structures, Vol. 14, No. 5-6, pp. 501-507, 1981.

KHAN, M. R., and AINSO, H., "Optimal Design of Nonplanar Diaphragms," A Technical Report for Bendix Corporation, Utica, N.Y., 1979.

AINSO, H., and KHAN, M. R., "Effects of Rim and Hub Flexibility During Axial Loading of Diaphragms," A Technical Report for Bendix Corporation, Utica, N.Y., 1980.

KHAN, M. R., "A General Computer Program for the Beams, Frames and Trusses - A User's Manual," Department of Civil and Environmental Engineering Department, Clarkson College,Potsdam, N.Y. 13676, Report Number 81-3, August 1981.

KHAN, M. R., "A Computer Program for Optimum Design of Natural Frequency Constrained Structures," Department of Civil and Environmental Engineering, Clarkson College, Potsdam, N.Y. 13676, Report Number 81-2, August 1981.

KHAN, M. R., "Optimality Criterion Techniques Applied to Frames Having Nonlinear Cross-Sectional Properties," Department of Civil and Environmental Engineering, Clarkson College, Potsdam, N.Y. 13676, Report Number 81-1, August 1981.

KHAN, M. R., and WILLMERT, K.D., "Finite Element Quasi-static Deformation Analysis of Planar Mechanisms With External Loads Using Static Condensation," ASME Paper No. 81-DET-104.

WILLMERT, K. D., JENG, K. K., MONSEF, M., and KHAN, M. R., "Application of Optimality Criterion Methods to Member Lengths and Cross-Sectional Properties of Frame Structures," Proceedings of the AIAA/ASME/ASCE/AHS 23rdStructures, Structural Dynamics and Materials Conference, AIAA-82-0720-CP, May 1982.

ZACHAROPOULOS, A., WILLMERT, K. D., and KHAN, M. R., "An Optimality Criterion for Structures with Stress, Displacement and Natural Frequency Constraints," Proceedings of the AIAA/ASME/ASCE/AHS 24th Structures, Structural Dynamics and Material Conference, AIAA-83-0939, May 1983.

KHAN, M. R., "BSEIM Computer Program For Generation of Random Synthetic Time Histories From The Floor Spectra," Rev. 1, Civil/Structures Group, Bechtel Power Corporation, January 1984.

KHAN, M. R., "Optimality Criterion Techniques Applied to Frames Having General Cross-Sectional Relationships," AIAA Journal, Vol. 22, No. 5, pp. 669-676, May 1984.

KHAN, M. R., "Improved Method of Generation of Artificial Time Histories, Rich In All Frequencies, From Floor Spectra," Earthquake Engineering and Structural Dynamics Journal, Vol. 15, pp. 985-992, 1987.

KHAN, M. R., "User and Verification Manual For MICRO-SEIM Computer Program," September 1988, Pacific Gas & Electric Company.

KHAN, M. R., "Users' Guide, Theory Verification for GENRRS

Computer Program," February 1989, Pacific Gas & Electric Company.

KHAN, M. R. and SHAKIBNIA, B., "Generic Fragility Seismic Testing of Breakers For Diablo Canyon Power Plant," Proceedings of the 1990 ASME Boiler & Pressure Vessel Conference, Book No. H00616.

KHAN, M. R. and CHEN, WAYNE W.H., "Nonlinear InteractionAnalysis for Seismic Equipment qualification," Proceedings of the ASME, PVP -Vol. 237-2, Seismic Engineering Vol. 2, pp. 115-120, 1992.

KHAN, M. R., CHEN, WAYNE W.H. AND WANG, T. Y.,"Seismic Qualification of Multiple Interconnected Safety Related Cabinets In a High Seismic Zone," Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology,.Vol. K2, 1993.

KHAN, M. R., CHEN, WAYNE W.H., SHAKIBNIA, B, AND DAVIS, R.D. "Torque Test For Bolts/Screws In Maintaining Equipment eismic Qualification," Proceedings of the ASME, PVP Vol. 56-2, Seismic Engineering, Vol. 2, pp. 41-45, 1993.

KHAN, M. R., CHEN, WAYNE W.H. AND CHU, W. S., "Seismic Qualification of a Commercial Grade Diesel Generator In a High Seismic Zone," Proceedings of 4th International Topical Meeting on Nuclear Thermal Hydraulics, Operations and Safety., Vol. 1, pp. 5D-1 - 5D-5, April 1994.

PRESENTATIONS

"Vibrational Analysis of Mechanisms Using Constant Length Finite Elements," ASME Winter Annual Meeting, New York, December 1976.

"Optimality Criterion Techniques Applied to Mechanical Design," ASME Design Engineering Technical Conference, Chicago, September 1977.

"Automated Analysis/Design of High Speed Planar Mechanism," 5th OSU Applied Mechanism Conference, Oklahoma, November 1977.

"A New Optimality Criterion Method for Large Scale Structures," AIAA/ASME 19th Structures, Structural Dynamics, and Materials Conference, Maryland, April 1978.

"A Hierarchy of Methods for Analysis of Elastic Mechanisms with Design Applications," ASME Design Engineering Technical Conference, Minneapolis, September 1978.

"Mechanism Optimization Via Optimality Criterion Techniques," ASME Design Engineering Technical Conference, Minneapolis, September 1978.

"Optimality Criterion Techniques Applied to Structures with Multiple Design Variable per Members," AIAA/ASME/ASCE/AHS 20th Structures, Structural Dynamics, and Materials Conference, St. Louis, April 1979.

"Optimality Criteria Techniques Applied to Structures Composed of Different Element Types," AIAA/ASME/ASCE/AHS 21st Structures, Structural Dynamics, and Materials Conference, Seattle, Washington, May 1980.

"Optimality Criterion Techniques Applied to Frames Having Nonlinear Cross-Sectional Properties," AIAA/ASME/ASCE/AHS 22nd Structures, Structural Dynamic, and Materials Conference, Atlanta, Georgia, April 1981.

"Finite Element Quasi-Static Deformation Analysis of Planar Mechanisms With External Loads Using Static Condensation," ASME Design Engineering Technical Conference, Hartford, Connecticut, September 1981.

"Application of Optimality Criterion Methods to Member Lengths and Cross-Sectional Properties of Frame Structures," AIAA/ASME/ASCE/AHS 23rd Structures, Structural Dynamics, and Materials Conference, New Orleans, Louisiana, May 1982.

"An Optimality Criterion for Structures With Stress, Displacements and Natural Frequency Constraints," AIAA/ASME/ASCE/AHS 24th Structures, Structural Dynamics and Materials Conference, Lake Tahoe, Nevada, May 1983.

"Generic Fragility Seismic Testing of Breakers For Diablo Canyon Power Plant," 1990 ASME Boiler & Pressure Vessels Conference, Nashville, Tennessee, July 1990.

"Nonlinear Interaction Analysis for Seismic Equipment qualification," ASME Pressure Vessels and Piping Conference, New Orleans, Louisiana, June 1992.

"Seismic Qualification of Multiple Interconnected Safety Related Cabinets In a High Seismic Zone," 12th International Conference on Structural Mechanics in Reactor Technology, Stuttgart, Germany, August 1993.

"Torque Test For Bolts/Screws In Maintaining Equipment Seismic Qualification," ASME Pressure Vessels and Piping Conference, Denver, Colorado, July 1993.

"Seismic Qualification of a Commercial Grade Diesel Generator In a High Seismic Zone," 4th International Topical Meeting on Nuclear Thermal Hydraulics, Operations and Safety, Taipei, Taiwan, 1994.

"Failure Analysis of Globe Control Valves with Spring-Diaphragm Actuator for Nuclear Power plant Applications, " Topical Meeting on Nuclear Thermal Hydraulics, Beijing, China, April 1997.

"Seismic Qualification of New Underground Diesel Fuel Tanks at Diablo Canyon Nuclear Power Plant Site, " ASME Pressure Vessels & Piping Division Conference, Orlando, Florida, 1997.

CONDENSED 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.	)
	) TELEPHONE DEPOSITION OF:
(Private Fuel Storage Facility)	) **KRISHNA P. SINGH** and **ALAN I. SOLER**
	) (Utah Contention L/QQ)

Wednesay, March 6, 2002 - 9:12 a.m.

Location: Office of the Attorney General
160 East 300 South, 5th Floor
Salt Lake City, Utah

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

State's
Exhibit 120

1 **cohesiveness, and also checking that the input data was**
2 **correct, that the model followed accepted engineering**
3 **principles and that the results made sense.**
4 Q. And Dr. Soler, what was your role with
5 respect to this document? I'm sorry, Dr. Singh.
6 **A. (DR. SINGH) My role is the same in every**
7 **work that's done in the company in the areas where I**
8 **have direct expertise. I review the work, and the**
9 **engineers who do the work, they can consult with me on**
10 **different aspects of the solution. And I'm generally**
11 **familiar with the work done because of the interaction,**
12 **but I don't do the work myself. And I very seldom**
13 **serve as a direct reviewer of the document.**
14 Q. And the third document, Dr. Soler, do you
15 have in front of you a Holtec document entitled
16 "Dynamic Response of Freestanding HI-STORM 100 Excited
17 by 10,000-Year Return Earthquake at PFS"?
18 **A. (DR. SOLER) Yes.**
19 Q. And are you the principal author for this
20 document?
21 **A. (DR. SOLER) Yes, I am.**
22 Q. And Dr. Singh, what was your involvement
23 with this document?
24 **A. (DR. SINGH) My involvement, Ms. Nakahara --**
25 **do you like to be called Ms. or Mrs.? How do you like**

1 **to be addressed?**
2 MS. NAKAHARA: It doesn't matter -- Ms.
3 **A. (DR. SINGH) My involvement in all these**
4 **documents has been uniform in respect of providing**
5 **consulting assistance, discussing the theoretical and**
6 **methodological aspects of the solution, and providing**
7 **general guidance to the others.**
8 Q. And I forgot to make a copy of a fourth
9 document entitled "Evaluation of the Confinement
10 Integrity of a Loaded Holtec MPC under a Postulated
11 Drop Event."
12 **A. (DR. SINGH) Oh, you do have it.**
13 **A. (DR. SOLER) You already mentioned that or**
14 **asked that.**
15 **A. (DR. SINGH) I don't think so.**
16 Q. And that document -- okay. What about the
17 "PFSF Site Specific HI-STORM Drop Tipover Analysis"? I
18 don't have that document, correct?
19 **A. (DR. SINGH) Correct, you don't.**
20 **A. (DR. SOLER) Yes.**
21 Q. Are you familiar -- Dr. Soler, are you
22 familiar with that document?
23 **A. (DR. SOLER) I'm familiar with that**
24 **document, yes.**
25 Q. Did you have any role with that document?

1 **A. (DR. SOLER) I believe my role was as a**
2 **reviewer of the document, and of course I had some**
3 **discussions early on as to the procedure that we would**
4 **follow.**
5 Q. And Dr. Singh, was your role with respect to
6 the drop tipover analysis the same as your role in the
7 other PFS Holtec documents?
8 **A. (DR. SINGH) That is correct.**
9 Q. Dr. Singh, you stated that you very seldomly
10 do the direct analysis any more. Have you ever been
11 the principal analyst for a HI-STORM cask to analyze
12 the response of a HI-STORM cask to a seismic event?
13 **A. (DR. SINGH) No. I have not been the**
14 **principal analyst for seismic analysis of Holtec's cask**
15 **systems. In recent years I have not done direct**
16 **analysis myself.**
17 Q. When was, approximately, the last time you
18 have performed direct seismic analysis or been the
19 principal analyst?
20 **A. (DR. SINGH) Probably about ten years ago.**
21 Q. And Dr. Soler, do you recall in your
22 deposition in November I asked you questions about
23 other cask stability analyses performed by Holtec for
24 other sites?
25 **A. (DR. SOLER) Yes.**

1 Q. I have a few more questions in that area.
2 Is it correct that Holtec evaluated the cask stability
3 at Diablo Canyon?
4 **A. (DR. SOLER) We have evaluated our -- of**
5 **course still in the process, it's an ongoing project.**
6 **Currently under review by the NRC.**
7 Q. Have you performed multiple analyses for
8 Diablo Canyon -- what I'm trying to ask is, you have a
9 current analysis. Did you have -- did you perform an
10 analysis previously in --
11 **A. (DR. SOLER) Well, if your question deals**
12 **with casks on a pad, there has really been one analysis**
13 **which has been independently reviewed and modified over**
14 **the time period of this project.**
15 Q. Okay. And that was for the HI-STORM 100,
16 and now it's the HI-STORM 100F; is that correct?
17 **A. (DR. SOLER) That is correct.**
18 Q. And did you use the same mathematical --
19 strike that. Is it correct for the PFS cask analysis
20 you used a lump mass mathematical model developed by
21 Holtec?
22 **A. (DR. SOLER) Correct.**
23 Q. And did you in your analysis -- strike that.
24 Were you principally involved in the Diablo Canyon cask
25 stability analysis?

A. (DR. SOLER) Yes.
Q. Did you use the same lump mass mathematical model used for the PFS site in the Diablo Canyon analysis?
A. (DR. SOLER) No. Let me qualify that to some extent. The report on the 10,000-year return earthquake at PFS uses the same computer code that we've used for Diablo Canyon.
Q. Is it correct that Holtec conducted a cask stability analysis for Entergy Northwest?
A. (DR. SOLER) Energy.
Q. Oh, Energy. Sorry.
A. (DR. SOLER) Yes.
Q. Were you involved in the principal analysis?
A. (DR. SOLER) I believe that I acted as a reviewer for that analysis.
Q. And did that analysis use the same lump mass mathematical model used in the PFS 2,000-year return?
A. (DR. SOLER) The same mathematical model was used. There were obviously a different number of casks involved. Dimensions of the pad were different.
Q. When did you perform this analysis, or when was this analysis performed?
A. (DR. SOLER) I can only hazard a guess here, that roughly a year ago, maybe year and a half.

Q. Do you recall what the zero period acceleration for Energy Northwest was?
A. (DR. SOLER) Again, I'm making a guess here without a report in front of me, but I suspect on the pad it was on the order of a half a g horizontal.
Q. Do you recall what the maximum horizontal displacement was at Energy Northwest?
A. (DR. SOLER) No, I don't, but it was on the same order as PFS, meaning that it wasn't ten inches or quarter of an inch.
Q. And do you recall what the maximum vertical uplift, if any, was?
A. (DR. SOLER) I can't recall that, no.
Q. And is it correct that Holtec provides a portion of the cask system, will provide a portion of the cask system at Trojan?
A. (DR. SOLER) That is correct.
Q. Did you do -- did Holtec do a cask stability analysis for the Trojan facility?
A. (DR. SOLER) I do not believe so.
Q. And other than -- is it correct that Holtec performed a cask stability analysis for the Tennessee Valley facility?
A. (DR. SOLER) That's correct.
Q. And when did you conduct this -- strike

that. Were you principally involved in the analysis, cask stability analysis?
A. (DR. SOLER) I was involved as a reviewer and giving some guidance as to the methodology.
Q. And approximately when was this analysis conducted?
A. (DR. SOLER) This analysis is still ongoing.
Q. And do you recall what the zero period acceleration for Tennessee Valley is?
A. (DR. SOLER) I don't recall exactly.
Q. And Dr. Singh, your involvement with -- is it accurate to characterize your involvement with the Diablo Canyon, the Energy Northwest, the Tennessee Valley cask stability analysis the same as your involvement with PFS as a reviewer and consultant available for technical consulting with your staff?
A. (DR. SINGH) That is correct.
Q. Dr. Soler, have you conducted other cask stability analysis for the HI-STORM 100 at other locations than we've discussed that have a peak ground acceleration of above 0.4 g's?
A. (DR. SOLER) Performed some scoping analysis for Humboldt Bay, but that does not involve HI-STORM. Let me correct that. Early in the game before there was a choice by the utility, we did subject the

HI-STORM to the same kind of scoping analysis.
A. (DR. SINGH) Can I supplement that answer?
Q. Yes.
A. (DR. SINGH) We had made generic submittals to the NRC back in 1994-95 time frame where we submitted a complete treatment of seismic loads, generic seismic loads, and that would be defined seismic loads well in excess of .5 g's, actually in excess of .6 g's.
Q. And is that with the Safety Analysis Report?
A. (DR. SINGH) Yes.
Q. Was that part of the Safety Analysis Report?
A. (DR. SINGH) Yes.
Q. Topical Safety Analysis Report?
A. (DR. SINGH) That would be a correct characterization also.
Q. The scoping for Humboldt Bay was originally for the HI-STORM and then for what cask system?
A. (DR. SOLER) It's for the HI-STAR.
Q. And was the seismic analysis for a freestanding cask system or an anchored cask system?
A. (DR. SOLER) Humboldt Bay, it was for a freestanding cask system. Some of the generic work that we did used a bounding earthquake that in effect bounded both Humboldt Bay and Diablo Canyon. So we

1 can't necessarily characterize it as one or the other.
2 Q. And the bounding, when you --
3 A. (DR. SOLER) An earthquake larger than would
4 be expected at either of the sites.
5 Q. But the bounding work, was that in the
6 Diablo Canyon or Humboldt Bay cask stability analysis
7 or in the topical Safety Analysis Report?
8 A. (DR. SOLER) We did bounding work
9 specifically for the utility, and some of the work
10 found its way into the submittal to the NRC.
11 Q. Submittal -- I'm sorry. The submittal to
12 the NRC for Diablo Canyon or for the TSAR, or both?
13 A. (DR. SINGH) Both.
14 A. (DR. SOLER) Yeah, although the submittal to
15 the -- official submittal to the NRC on Diablo Canyon
16 deals only with anchored casks.
17 Q. (BY MS. NAKAHARA) Dr. Singh, are you
18 familiar with the document that's been marked as L/QQ
19 Exhibit 1 entitled "Joint Submittal of Unified
20 Geotechnical Contention, Utah L and Utah QQ"?
21 A. (DR. SINGH) Yes, I am.
22 Q. Dr. Singh, is it correct that you've been
23 named as a witness by PFS with respect to this
24 contention, consolidated contention?
25 A. (DR. SINGH) Yes, I believe I have.

1 Q. Will you please review the contention -- and
2 as a preface to Dr. Soler, I'll be asking you the same
3 thing -- and identify which specific areas you expect
4 to testify about.
5 MR. GAUKLER: Limit the response to Section
6 C and D for the scope of this deposition. Look at
7 Section C and D and answer counsel's question.
8 A. (DR. SINGH) I've marked the ones that I
9 believe I will testify or provide information either
10 completely or in part.
11 Q. Will you identify those sections and briefly
12 describe what portions, if -- strike that. Will you
13 identify the sections, and you mentioned that you may
14 testify in part, describe what parts of those sections
15 you plan to testify with respect to.
16 A. (DR. SINGH) Okay, you've set me up for a
17 speech here. All right, I'll try to be brief.
18 Under "Characterization of Subsurface
19 Soils," broad category upper case C, I will
20 specifically address item 3.e without limitation.
21 Under category D, upper case D, "Seismic
22 Design and Foundation Stability," I will specifically
23 address 1.b, c, e, f, g, h, and i, with the limitation
24 that under h.(ii) my opinion with respect to fault
25 fling would be only from the perspective of a cask

1 dynamic system. That's as brief as I can get.
2 Modify me if --
3 DR. SOLER: I think that's fine.
4 Q. And Dr. Soler, you --
5 A. (DR. SOLER) I would say -- I would answer
6 in the same manner.
7 Q. Dr. Soler, if you can, can you describe how
8 your testimony will differ or supplement Dr. Singh's
9 with respect to these areas?
10 A. (DR. SOLER) I would suspect that my
11 testimony might be more direct to specific points of
12 the report, because I was actively involved in the
13 guidance and review of most of them in addition to
14 being a consultant. So I would have more direct
15 knowledge of the details.
16 Q. And Dr. Singh, do you agree or do you want
17 to supplement how you believe your testimony will
18 coordinate or supplement Dr. Soler's testimony?
19 A. (DR. SINGH) I think Dr. Soler described it
20 quite succinctly.
21 Q. Dr. Soler, what expertise do you bring to
22 this testimony that Dr. Singh does not have?
23 A. (DR. SOLER) For this specific project?
24 Q. Yes.
25 A. (DR. SOLER) I have a direct knowledge of

1 some of the applications of the computer codes
2 involved, hands-on experience, if you will.
3 Q. And Dr. Singh, do you have any expertise
4 with respect to your planned testimony that Dr. Soler
5 does not possess?
6 A. (DR. SINGH) Well, he's quite
7 self-sufficient. He can deal with the issue entirely
8 on his own. But I have the advantage of having some
9 intellectual remove from the problem, and therefore I
10 can provide perhaps perspectives that one would have
11 from not being in the trenches and doing the analysis
12 day in, day out.
13 Q. The next area I'd like to ask you about is
14 the lump mass mathematical model. Dr. Soler, who
15 developed the lump mass mathematical model code that
16 you used for the PFS site?
17 A. (DR. SOLER) It was modified from an
18 existing code in a textbook that was published in 1976,
19 and I adapted it for use in both wet storage and dry
20 storage.
21 Q. And I'm sorry; you did this in 1966 or the
22 original --
23 A. (DR. SOLER) The book -- the original code,
24 which was a general lumped mass analysis code, was
25 published in a book that was published in 1976 that

had -- it was a general dynamic analysis textbook. I took that code over the years and adapted it first for use in wet storage seismic analysis, and later on used it for dry storage seismic analysis.

Q. And when did you adapt it for -- approximately when did you adapt it for wet storage?

A. (DR. SOLER) I believe, but this is not -- let me see if my resume will give me a clue, but I believe we're talking about 1979, '80, '81 time frame.

Q. And when did you adapt it for dry storage?

A. (DR. SOLER) Probably in early 1992.

Q. In general, what type of adaptions did you need to make so the code would work for dry storage?

A. (DR. SOLER) Dry storage, actually we made no direct adaptions or modifications to the code. We developed some what I'll call preprocessors that enabled us to automatically generate the spring constants that are used to simulate the contact phenomena around the periphery of the cask.

I'll qualify that a little bit. We did add some output statements to the code, enabling us to get some information that was directly usable for reports, for instance, to be able to generate information to predict the maximum displacements of one or more casks.

Q. Is there a limit to the number of casks you

can evaluate in your model?

A. (DR. SOLER) There is a current limit set by the dimension statements in the FORTRAN code, but that limit could be changed simply by modifying the dimension statements. With the size capability of computers now, your only restriction is the time it would take to do a much larger system.

Q. The preprocessor that automatically calculates spring constants, has it been verified with another mathematical model?

A. (DR. SOLER) That particular code was simply a tool and was verified by actually doing a sample calculation and checking it manually to see that it gave you the same results that you got from a hand calculation. It was simply a program enabling you to avoid doing a lot of things manually.

Q. Is it correct that you used a different model for the 2000-year analysis at PFS versus the 10,000-year analysis?

A. (DR. SOLER) A different computer code, but essentially the same model. In other words, if your definition of model is that I have an MPC that is inside an overpack and that overpack is sitting on a pad or portion of the pad and that pad or the portion of the pad is being excited, then the models are the

same; the algorithm, the engine by which the program solves the problem is different.

Q. And why did you use a different algorithm?

A. (DR. SOLER) The 10,000-year earthquake was a beyond design basis earthquake. We fully expected from our previous results at other plants that the 2,000-year earthquake would give us what I will loosely call small deformation results, in other words, that we would not show extremely large rotations of the cask during the motion.

The code which you have labeled as a lumped mass model is a small deformation code in that it does not -- it's not capable without modification of modeling the potential for a cask to execute a large rotation.

The 10,000-year earthquake, being beyond the design basis, was a scoping analysis, and therefore it was quite likely to expect that we would experience large rotations of the cask, and therefore we used a program that was capable of managing that kind of a motion.

Q. And what program did you use?

A. (DR. SOLER) It's called Visual NASTRAN Desktop. It used to be called Working Model, but there was a corporate takeover.

Q. And do you have a name for Holtec's code that you developed?

A. (DR. SOLER) It has over the years gone by various names depending upon whether it was in the wet storage arena. In wet storage it was known as DynaRack. In early dry storage work it was sometimes called DynaCask. Lately, to avoid confusion, we have taken to calling it Dynamo.

Q. If I can remember to call it Dynamo, you'll know that I'm talking about your -

A. (DR. SOLER) I'll know what you're talking about.

Q. Okay. And I just have to remember that.

A. (DR. SOLER) Let me add a little bit more just to avoid confusion. Internally and in some references it was also known as MR-2, the MR standing for "multi-rack" at the time.

Q. So is it correct that Dynamo without modification cannot be used to evaluate the 10,000-year ground motions at PFS?

A. (DR. SOLER) I believe that it would give erroneous results.

Q. Is it correct that you did not use Dynamo for the Diablo Canyon cask stability analysis?

A. (DR. SOLER) That is correct.

Q. And was that for all revisions of the cask stability analysis?
A. (DR. SOLER) Yes. We did not use Dynamo for anything at Diablo Canyon that has been submitted to the NRC for their site license.
Q. And did you use Visual MASTRAN?
A. (DR. SOLER) Yes.
A. (DR. SINGH) NASTRAN.
A. (DR. SOLER) Yes. That's N, with an N, NASTRAN, not MASTRAN.
Q. I'm sorry. Thank you. And what model did you use in the Energy Northwest cask stability analysis?
A. (DR. SOLER) Dynamo.
Q. And for Tennessee Valley?
A. (DR. SOLER) Dynamo.
Q. Approximately what range of zero period accelerations do you believe Dynamo is capable of processing?
A. (DR. SOLER) I would say, without having pushed it, but I -- to the extent that zero period accelerations imply a certain cask motion, I would not hazard a guess as to the upper limit on Dynamo. I would have to run it and check on the results. And if those results remained in what I would call the small

rotation range, then I would accept the results from Dynamo. However, the reason for not using Dynamo for the 10,000-year earthquake was simply at the outset I expected large rotations to validate the results from Dynamo.
Q. And when you say "large rotations" --
A. (DR. SOLER) I'm talking qualitatively in the range of, say, 20 degrees, 15 degrees or higher.
Q. In your last deposition I asked whether Holtec's computer code, which I didn't ask it as Dynamo, but --
A. (DR. SOLER) A lumped mass model.
Q. Yes.
A. (DR. SOLER) Okay.
Q. Had been compared to other nonlinear codes. And is it correct to characterize your answer that portions of the NRC had compared it to portions with respect to wet storage?
MR. GAUKLER: Objection. I'd like to have you show the witness the question and answer before you ask whether that is the correct characterization.
A. (DR. SOLER) Do you want me to look at the stuff in yellow?
Q. No. Starting right there and through there (indicating).

Off the record.
(Discussion off the record.)
A. (DR. SOLER) Okay, now ask your question again.
Q. Is it fair to characterize your response in the last deposition, or I guess in general that NRC had compared portions of what I now know as Dynamo for wet storage to other nonlinear codes?
A. (DR. SOLER) Well, I'm not sure what the NRC has done independently, but as part of a number of submissions for particular utilities in wet storage applications, we were of course asked questions by the NRC staff reviewer, and previous to the submittal we were also sometimes asked questions by the utility reviewers before submittal. And if you take all of the submittals that we've made since when we started and through the wet storage period, there have been a whole range of problems considered. And there of course is a validation report that's been issued with different classical problems, both linear and nonlinear. Their "exact" solutions or their numerical solutions from other sources were compared with the results that we would get for the same problem.
So while you could not say that a particular wet storage submittal was completely modeled by another

program and compared with the results that we got, portions of the program were compared by testing the problem that had been done in the literature, or, in one case, a finite element model using ANSYS that was made up by a utility to characterize all of the features like nonlinear springs and gap elements that was in their model.
Q. You mentioned a validation report. Is this a formal document that's submitted to NRC?
A. (DR. SOLER) Yes, I believe it's in the public document now.
Q. And approximately how large is that validation?
A. (DR. SOLER) Like that (indicating). I believe, maybe I'm wrong, but we submitted at one time a table of contents to that report.
Q. Did you submit a validation report with the TSAR?
A. (DR. SOLER) No.
Q. Dr. Singh, do you want to add?
A. (DR. SINGH) May I supplement the response?
Q. Yes.
A. (DR. SINGH) One of the essential undertakings we have in nuclear plant design and analysis activities is to ensure that the computer

predicted displacements in the order of inches.
And to confirm that ANSYS was giving us reasonable results, we modeled the same thing on our Dynamo program and got the same order of magnitude of displacements. We were not at that time trying to check the specific details that we had to put on the client's racks against Dynamo. But we left those details off and just made a general check that our program was not predicting a quarter of an inch when ANSYS was predicting five inches. We got a general level of agreement there, and beyond that we used ANSYS.
Q. You mentioned with respect to this project that you used a large earthquake. Do you recall approximately what the zero period acceleration --
A. (DR. SOLER) No.
MR. GAUKLER: Objection. What project? The one you're talking about that uses ANSYS?
MS. NAKAHARA: Yes.
A. (DR. SOLER) I do not recall what we used to make that check. It was an informal check, what any competent engineer would do when he's developing a new model with a program that he's not used before on that particular application. So we just picked a time history that we had.

Q. With respect to Dynamo, do you directly apply ground acceleration time histories, or do you have to make some modifications to the time histories?
A. (DR. SOLER) We directly apply the acceleration time history with a change of variables.
Q. And what do you mean "with a change of variables"?
A. (DR. SOLER) Well, you can either treat a seismic problem by assuming that the ground is moving with some displacement time history and forcing through the connection to the ground or friction and gap elements, forcing the racks to move, or you can make a change of variables and assume -- not assume, and make everything relative to a fixed ground. And in that case the forces are applied to the rack or to the cask in the form of acceleration time histories, the ones you have, times the mass of the particular component. Based on the questions yesterday, I believe that that's what was done in the Altran report, too.
MS. NAKAHARA: How about if we take a ten-minute break.
(Recess from 10:39 to 10:56 a.m.)
Q. (BY MS. NAKAHARA) Dr. Soler, I have a few more questions to ask about the mathematical codes. What is the difference between Dynamo and Visual

NASTRAN that allowed Visual NASTRAN to accommodate potentially large rotations?
A. (DR. SOLER) Okay. Well, if you write the equations of motion of a system and restrict it to small rotations, you can simplify the equations.
In a nutshell, the Dynamo code does not alter the equilibrium equations step by step. It assumes to always satisfy equilibrium based on the original configuration.
The Visual NASTRAN code was written from the outset to accommodate large motions, falling objects that could tumble, turn over, bounce. Therefore, it did not make internally any simplifications to the equations that are presumably programmed at the site. So if you attempt to take a code that is written for small deflections and blindly just apply it and get a result that would indicate large deflections, either your program will blow up on you or it will just give you ridiculously large results that have no physical meaning, or it will simply give you wrong results that you may think there's a physical meaning to it. So you have to be careful to make sure that you don't pose to a code a problem that has a chance of going outside the range of validity of the code.
Q. And how can you ensure that the results for

the PFS 2,000-year return period using Dynamo are accurate results?
A. (DR. SOLER) If I take, say, the peak displacements that are predicted from any of the runs, we're talking about numbers on the order of three inches, say; and if I take three inches, which is a maximum excursion laterally and assume the worst, that the bottom of the cask was somehow pinned and it was rotating, which is usually the case with a .8 coefficient of friction, if you take three inches and divide by the height of the cask, which is 231 inches, and calculate that angle, that angle is very small and it's a commonly accepted number that would tell you you're still in a small deflection range.
Q. Jumping to the cask stability analysis for Diablo Canyon, were all cask stability analyses for unanchored casks conducted with Visual NASTRAN?
A. (DR. SOLER) Well, all analyses that were submitted to the NRC were for anchored casks and were conducted with Visual NASTRAN.
Q. What about the scoping analysis that you conducted for unanchored casks?
A. (DR. SOLER) The scoping analyses which were most likely a few years ago we're talking were analyzed with Dynamo, perhaps internally Visual NASTRAN, but I

Q. (BY MS. NAKAHARA) Let's go back on the record. Dr. Soler, what total vertical contact stiffness value did you use in the cask stability analysis?

A. (DR. SOLER) Total number, I think was 468 times 108. That's the pounds per inch. That's the sum of all the individual springs.

MR. GAUKLER: That was the total from all the --

MS. NAKAHARA: 468? Off the record.

(Discussion off the record.)

A. (DR. SOLER) It was a little more than that if you take numbers from the actual math book. That's the range, 450 plus times 106.

Q. Will you briefly explain how you calculated that?

A. (DR. SOLER) Okay, that number was calculated -- and this goes back to almost the beginning of our analysis. It had nothing to do with the earthquake. That number was calculated by taking the semi-infinite solution I referred to earlier this morning and looking at the problem of if I just simply took the cask, which has an 11-foot diameter, placed it down on a pad of concrete, knowing the properties of concrete in terms of the Young's modulus and the

radius, you can calculate a spring constant for that section of concrete. Then, since I was modeling 36 springs around the periphery, which I felt was a reasonable number, I simply divided that spring constant by 36 and imposed the result at each one of the vertical springs.

There are, of course, other ways to compute that spring constant, but the underlying rationale would be when you're finished and you can imagine that cask resting on whatever it's resting, that the deflection to predict should be small.

Q. And you calculate the deflection according to Exhibit 7; is that correct?

A. (DR. SOLER) That would be the formula I would use to determine whether the number that I came up with was a rational number to use. But I would have to determine the spring constant from the method I just described.

Q. And what total horizontal stiffness value did you use in your analysis?

A. (DR. SOLER) I'd really have to consult one of my outputs to refresh that number. I've got it here, so let's see if it's -- oh. In Section 5.3 of this report it says refer to an earlier report to get that spring constant. So I guess the report that I

really need to look at to give you that number is --

Q. Is that the '97 report?

A. (DR. SOLER) Yeah, probably 97-1631.

MS. NAKAHARA: I need to do a better search, but I don't believe we ever got the '97 report.

MR. GAUKLER: Okay.

MS. NAKAHARA: We have two other earlier versions of this report, none of the '97 that I could find. That was one of my problems looking at some of the references.

DR. SOLER: So I can't really give you that answer.

MS. NAKAHARA: So to the extent we don't have that report, we would like a copy of it.

MR. GAUKLER: Dr. Khan had a reference to it.

Q. (BY MS. NAKAHARA) Will you explain how you calculated, in general, the horizontal stiffness value?

A. (DR. SOLER) Generally speaking, what we usually do is -- again, the phenomena we're trying to simulate is what's called in the literature a stick-slip phenomena, meaning that nothing happens until you slip, and therefore something suddenly happens after that and you jump up to the value U times whatever the download is. So our simulation of that is

with a spring that has a very stiff spring constant so that it behaves like a linear spring up until the force predicted in that spring is greater than the coefficient of friction times the normal force in the contact spring that goes along with it.

Now, the number you choose is again based on physical principles. Again, if I take the object, whatever it is, and I assume a spring constant that is trying to simulate really something very stiff so that I don't get any deflection without sliding, I want a number that's big enough so that if I put on a load that's less than mu times W, I don't predict some unreasonably large number like quarter of an inch, half an inch. I want to predict something like .00 something, just for this simple problem. That way I know when I do the dynamics problem that what I'm going to see is the slipping behavior and not some elastic motion. So again, it's chosen mainly to get a result that you would expect from your physical intuition and physical observing the problem.

Q. Do you believe that the total contact stiffness for an unanchored cask is constant or varies during the entire duration of the earthquake motion?

A. (DR. SOLER) First let me ask, is that -- the term "total contact stiffness" means what to you?

CONDENSED 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.	)
	) TELEPHONE DEPOSITION OF:
(Private Fuel Storage Facility)	) **KRISHNA P. SINGH** and **ALAN I. SOLER**
	) (Utah Contention L, Part B)

VOLUME I

Thursday, November 15, 2001 - 2:19 p.m.

Location: Office of the Attorney General
160 East 300 South, 5th Floor
Salt Lake City, Utah

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

State's
Exhibit 121

CONDENSED 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.	)
	) TELEPHONE DEPOSITION OF:
(Private Fuel Storage Facility)	) **KRISHNA P. SINGH** and
	) **ALAN I. SOLER**
	) (Utah Contention L, Part B)

VOLUME II

Friday, November 16, 2001 - 11:26 a.m.

Location: Office of the Attorney General
160 East 300 South, 5th Floor
Salt Lake City, Utah

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

1 **process, Bob Youngs. So I'd say there was only one**
2 **person I talked to.**
3 Q. Thank you. And who at Stone and Webster
4 have you had discussions with?
5 **A. (DR. SOLER) Oh, let's see. John Donnell,**
6 **Jerry Cooper, Stan Macy, Paul Trudeau up in the Boston**
7 **office, and a couple of people in the Cherry Hill**
8 **office, Mr. Ebbeson. I don't believe I've missed**
9 **anybody, but there have been so many people over the**
10 **years, I might have. But those are the ones that come**
11 **to mind. Oh, and wait a minute. I guess I've had a**
12 **conversation with Dr. Wen Tseng having to do with the**
13 **input that I gave him for the pad analysis.**
14 Q. Okay, thank you. Who is your current
15 employer?
16 **A. (DR. SOLER) Holtec International.**
17 Q. And what is your position?
18 **A. (DR. SOLER) Executive vice president and**
19 **vice president of engineering.**
20 Q. And what are your duties?
21 **A. (DR. SOLER) Oversee the entire engineering**
22 **staff in general, and specifically have direct charge**
23 **of the people doing structural and seismic analysis.**
24 Q. And now if you'll look at your resume.
25 **A. (DR. SOLER) Okay.**

1 Q. If you'll review your resume, take a moment
2 to review your resume. Could you tell me if it's
3 current?
4 **A. (DR. SOLER) I would say -- let me just take**
5 **a look at the last item on it. I would say it's a**
6 **couple of years out of date. There's no date on here**
7 **saying when it was last updated, but I don't recall**
8 **updating it in the near past.**
9 Q. Is there any experience or publications that
10 is particularly relevant to Utah Contention L, Part B
11 that's not on your resume?
12 **A. (DR. SOLER) Yes. There is a publication,**
13 **actually it was a presentation that was given at the**
14 **recent Structural Mechanics and Reactor Technology**
15 **Conference in Washington, D.C.**
16 Q. And what was the subject of this
17 presentation?
18 **A. (DR. SOLER) Basically dry storage casks and**
19 **their behavior during seismic events.**
20 MS. NAKAHARA: And Paul, I'd request a copy,
21 to the extent he has one, of his presentation.
22 MR. GAUKLER: Okay.
23 Q. (BY MS. NAKAHARA) Dr. Soler, if you'll look
24 on page 2 of your resume in the section entitled Dry
25 Spent Fuel Storage Technology. And you have from 1992

1 to present you're the lead analyst in mechanical/
2 seismic/structural analysis, and I presume that was
3 your description in describing your duties?
4 **A. (DR. SOLER) That's correct.**
5 Q. What does METCON stand for?
6 **A. (DR. SOLER) That's just an acronym for**
7 **metal/concrete construction.**
8 Q. And have you conducted site-specific seismic
9 analyses that estimate the probability of cask failure
10 for sites other than PFS?
11 MR. GAUKLER: Objection. What do you mean
12 by cask failure?
13 Q. Let me rephrase. Have you conducted
14 site-specific cask stability analysis from seismic
15 ground motion for facilities other than the PFS
16 facility?
17 **A. (DR. SOLER) Yes, I have.**
18 Q. For which sites, or for which --
19 **A. (DR. SOLER) For Diablo Canyon, some scoping**
20 **work for Humboldt Bay, some work for Enertgy Northwest,**
21 **and for -- let's see. Did we do -- I believe that we**
22 **did some for Dresden and for the Tennessee Valley**
23 **Authority.**
24 Q. For the analysis at Diablo Canyon, is this
25 for the anchored HI-STORM 100S cask?

1 **A. (DR. SOLER) Yes, it is.**
2 Q. Do you recall the ground motion at Diablo
3 Canyon?
4 **A. (DR. SOLER) In what way?**
5 Q. The ground motion that you estimated, or you
6 performed your cask stability analysis.
7 **A. (DR. SOLER) I recall specifically the**
8 **general level of the zero period accelerations, if**
9 **that's what you're asking me.**
10 Q. Yes.
11 **A. (DR. SOLER) There are a number of**
12 **excitations provided to us.**
13 Q. Approximately when was the ground motion of
14 the zero period acceleration?
15 **A. (DR. SOLER) Between .9 and 1.**
16 Q. For what earthquake magnitude is a zero
17 period acceleration at Diablo Canyon?
18 **A. (DR. SOLER) I think that's what I just gave**
19 **you. I mean, it's not a -- what do you mean by**
20 **earthquake magnitude?**
21 Q. Local Richter.
22 **A. (DR. SOLER) I don't know the answer to that**
23 **one.**
24 Q. And approximately what time period did you
25 prepare the Diablo Canyon analysis?

1 **A. (DR. SOLER) Over the period September 2000**
2 **to essentially September 2001.**
3 MS. NAKAHARA: Can we get a copy of this
4 analysis?
5 MR. GAUKLER: I'll take it under advisement.
6 Q. (BY MS. NAKAHARA) The scoping for Humboldt
7 Bay, approximately what period did you do the scoping?
8 **A. (DR. SOLER) I can't honestly recall the**
9 **dates without leaving the room, I guess.**
10 Q. That's fine. Was this for a HI-STORM 100
11 cask or a 100S cask?
12 **A. (DR. SOLER) It was not.**
13 Q. The Entergy Northwest, what type of cask
14 system did you analyze there?
15 **A. (DR. SOLER) That was a 100S.**
16 Q. And where is the Entergy Northwest facility
17 located?
18 **A. (DR. SOLER) Richland, Washington.**
19 Q. And what were the zero period accelerations
20 that you looked at in that analysis?
21 **A. (DR. SOLER) Outside the building, I**
22 **believe -- I'd just better not say, because I'm**
23 **guessing. But it was not as large as what I quoted you**
24 **for Diablo Canyon.**
25 MS. NAKAHARA: We'd like a copy of that

1 analysis also.
2 MR. GAUKLER: I'll take it under advisement.
3 Q. (BY MS. NAKAHARA) And for the Dresden
4 facility, what type of zero period accelerations did
5 you use there?
6 **A. (DR. SOLER) They were very low, about 0.2.**
7 Q. Okay. And the Tennessee Valley facility,
8 what type of zero period accelerations did you use?
9 **A. (DR. SOLER) In the neighborhood of .5 to**
10 **.6.**
11 Q. And what type of cask system is at the
12 Tennessee Valley facility?
13 **A. (DR. SOLER) A 100S.**
14 MS. NAKAHARA: And we'd like a copy of that
15 analysis also.
16 MR. GAUKLER: I'll take it under advisement.
17 Q. (BY MS. NAKAHARA) If you'll look at your
18 resume and turn to page 6. And listed as publication
19 47 -- oh, strike that. I asked Dr. Singh about this
20 one.
21 If you'll look at page 7. Item publication
22 50 entitled Foundation Stresses Under Support of
23 Freestanding Equipment Subjected to External Loads.
24 Will you generally describe this publication, the
25 subject of this -- not the subject. What's in this

1 publication?
2 **A. (DR. SOLER) Well, it had nothing to do with**
3 **dry storage casks. It dealt with heat exchanger**
4 **foundations, and the external loads were a combination**
5 **of piping and seismic loads. But it was dealing with**
6 **determining the stresses in the foundation of a heat**
7 **exchanger.**
8 Q. Would the principles applied to heat
9 exchanger foundation be similar to that of a dry cask
10 storage system?
11 **A. (DR. SOLER) Not really. The shape was**
12 **specific to heat exchangers, for the most part. I**
13 **mean, if you look at the date, 1985, that was well**
14 **before dry storage entered my thinking.**
15 MS. NAKAHARA: I asked for this document
16 earlier, Mr. Gaukler. I'd still request that we get a
17 copy of this.
18 MR. GAUKLER: Okay.
19 Q. (BY MS. NAKAHARA) And then Dr. Soler, if
20 you'll look at publication No. 57 on page 7 entitled
21 Some Results from Simultaneous Seismic Simulations of
22 All Racks in a Fuel Pool. Will you describe this
23 publication in general terms?
24 **A. (DR. SOLER) That was an analysis of a**
25 **series of spent fuel racks considered immersed in water**

1 **in a fuel pool and subject to a hypothetical seismic**
2 **excitation. It was not specific to any plan.**
3 MS. NAKAHARA: Okay. And we'd request this
4 document also.
5 MR. GAUKLER: Okay.
6 Q. (BY MS. NAKAHARA) And then if you'll look
7 at publication 59, Seismic Response Characteristics of
8 HI-STAR 100 Cask System on Storage Pads. Dr. Soler,
9 will you generally describe what's in this publication?
10 **A. (DR. SOLER) This was basically an early**
11 **work that we did for Pacific Gas and Electric to simply**
12 **evaluate our initial attempts at anchoring casks, and**
13 **at that time we were looking at the HI-STAR 100 cask**
14 **system. It was a paper that was written.**
15 MS. NAKAHARA: Okay. And we still request a
16 copy of this document.
17 MR. GAUKLER: Okay.
18 Q. (BY MS. NAKAHARA) Dr. Soler, are you
19 familiar with NRC's Rulemaking Plan SECY 98-126, which
20 was published approximately in -- obviously 1998? And
21 I apologize for not having a copy for you to look at.
22 **A. (DR. SOLER) The answer is no.**
23 Q. Are you familiar with Exhibit 12 of the
24 deposition exhibits? If Paul would get you a copy of
25 that.

1 describing your cask stability analysis, you mentioned
2 that you used a lump mass model of the system.
3 **A. (DR. SOLER) Yes.**
4 Q. Have you, for the PFS case or any previous
5 case in which you used the same model for a HI-STORM
6 100 cask, have you calibrated that model with any test
7 data?
8 **A. (DR. SOLER) Only against classical**
9 **solutions. I would not say -- certain aspects of the**
10 **model have been calibrated against test data in the wet**
11 **storage arena, but in general the program has been**
12 **validated by comparing against other solutions which**
13 **have the same characteristics.**
14 Q. And do I recall correctly that you compared
15 your model solutions to ANSYS, or am I not recalling
16 correctly?
17 **A. (DR. SOLER) I believe at one stage of the**
18 **development of the algorithm a portion of the model for**
19 **a specific job was compared against a similar model**
20 **from ANSYS, although the comparison was not made on**
21 **racks or casks, it was made on a problem that was**
22 **developed simply for the purposes of making the**
23 **comparison.**
24 Q. To the extent you recall, what other models
25 were compared to the lump mass model used in the PFS

1 analysis?
2 MR. GAUKLER: Objection, vague and ambiguous
3 question.
4 Q. Dr. Soler, do you understand the question?
5 You indicated that you compared other models to the
6 lump mass model you referred to earlier in the PFS cask
7 stability analysis.
8 **A. (DR. SOLER) No, no. What I stated was that**
9 **the entire, the computer codes that we used for PFS has**
10 **been used previously in dry storage submittals in the**
11 **recent past, and in the near and distant past it's been**
12 **used in wet storage applications. And in the course of**
13 **applications before the NRC in wet storage, we did some**
14 **comparisons of the predictions of our program with the**
15 **predictions of other programs the course of validating**
16 **our code. It was not specifically comparing a wet**
17 **storage analysis by our code with a wet storage**
18 **analysis by another code.**
19 Q. Okay.
20 **A. (DR. SOLER) Simply pick a problem that had**
21 **a well-known solution or had been done by another**
22 **analyst using another code and compare it with what we**
23 **would get using our code.**
24 Q. Okay, thank you for the clarification. Did
25 the other codes that you compared in the wet storage

1 arena, did those codes have any actual test data in
2 which they were calibrated, to the extent you know?
3 **A. (DR. SOLER) I would not know one way or the**
4 **other, although at least in one case where a simple**
5 **code was used as part of a thesis of the university, I**
6 **believe that there was some testing of the results of**
7 **that code against an experimental model.**
8 Q. Do you have any, does Holtec have any test
9 data which shows the HI-STORM 100's ability to
10 withstand ground motion?
11 **A. (DR. SOLER) No.**
12 Q. And is it correct that you also -- this
13 would encompass any bench scale test data?
14 **A. (DR. SOLER) The answer would be still no.**
15 MS. NAKAHARA: Okay, thank you.
16 We've been going probably less than 30
17 minutes, but if you're willing to take a perhaps a
18 longer break, a 30-minute break, I think I can make
19 this go a little faster.
20 MR. GAUKLER: Okay.
21 MS. NAKAHARA: Actually, how about if I call
22 you back at four o'clock your time? Is that convenient
23 for you, Mr. O'Neill?
24 MR. O'NEILL: That's fine.
25 MS. NAKAHARA: Thank you. I suspect I have

1 less than an hour of questions.
2 MR. O'NEILL: Dr. Soler, or all together?
3 MS. NAKAHARA: All together.
4 (Lunch recess from 1:27 to 2:06 p.m.)
5 Q. (BY MS. NAKAHARA) We are reconvening the
6 deposition of Dr. Alan Soler and Dr. Krishna Singh, and
7 Dr. Singh has now joined us. Is that correct?
8 **A. (DR. SINGH) That is correct. This is Chris**
9 **Singh.**
10 MS. NAKAHARA: And I think Paul, Mr. Gaukler
11 wanted to go on the record attributing a paragraph in
12 the declaration to Mr. Singh, and I'll let Paul do
13 that.
14 MR. GAUKLER: I want to go on the record to
15 say that he will be testifying to paragraph 33 that's
16 in the declaration, talking about the degradation of
17 concrete due to heat transfer in a tipover condition.
18 And I want to clarify one other point. I
19 had thought that paragraph No. 22, which we didn't have
20 anybody to identify, would be Alan Soler separately.
21 It will be Alan Soler and Chris Singh, paragraph 22.
22 Q. (BY MS. NAKAHARA) Dr. Soler, are you
23 familiar with the location of faults near the proposed
24 PFS site as described in Geomatrix's seismic hazard
25 assessment ?

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1 A. (DR. SOLER) **No, other than the limited**
2 **reading I gave it four years ago, five years ago.**
3 Q. Are you aware that a major fault capable of
4 generating a 6.5 magnitude earthquake could impact the
5 PFS site?
6 A. (DR. SOLER) **That sounds like a two-part**
7 **question to me. Am I aware?**
8 Q. Yes. Are you aware that a major fault
9 capable of generating a 6.5 magnitude earthquake could
10 impact the PFS site?
11 A. (DR. SOLER) **I am not aware of it.**
12 Q. Are you familiar with the term "near fault
13 effects" for earthquakes?
14 A. (DR. SOLER) **Not really.**
15 Q. Then is it fair to say that you did not
16 consider near fault effects in your cask stability
17 analysis for the HI-STORM 100 at the PFS site?
18 MR. GAUKLER: Objection. There is no basis
19 for him to say whether he knew whether he did or
20 whether he didn't.
21 Q. (BY MS. NAKAHARA) Did you --
22 MR. GAUKLER: He's already described that he
23 got input through Geomatrix.
24 Q. (BY MS. NAKAHARA) Did you consider near
25 fault effects, to the extent you know, in the cask

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1 stability analysis for the HI-STORM 100 at the PFS site
2 for a 2,000-year return period?
3 MR. GAUKLER: Same objection on 2,000-year.
4 A. (DR. SOLER) **I considered the earthquakes**
5 **that were given to me. The basis for those earthquake**
6 **time histories I'm not familiar with.**
7 Q. And will you clarify, did you calculate the
8 design-basis ground motion yourself or rely on
9 Geomatrix? You told me what you received, but I guess
10 I don't quite understand.
11 A. (DR. SOLER) **The original deterministic**
12 **earthquake, we received the response spectra and we**
13 **computed the time history from that response spectra.**
14 **For all of the other earthquakes we received the time**
15 **history directly.**
16 Q. Directly from Geomatrix?
17 A. (DR. SOLER) **Correct.**
18 Q. Okay, thank you. In your opinion, would
19 earthquake waves arriving at an angle to the HI-STORM
20 100 cask, would it would it cause additional rocking
21 and torsional motion than if it approached
22 perpendicularly?
23 MR. GAUKLER: Object to the form of the
24 question. You can answer if you can.
25 A. (DR. SOLER) **I'm not an expert in that area.**

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1 **That was my answer.**
2 Q. No, I'm sorry. I'm skipping over questions
3 that were related to that. I'm sorry. I should have
4 told you that.
5 In your analysis HI-2012640 -- I'm sorry, I
6 don't have a title. Do you know which report I'm
7 referring to?
8 A. (DR. SOLER) **Yes, I've got it in front of**
9 **me. Multi-cask Response at PFS ISFSI from 2,000-year**
10 **Seismic Event, Rev 2.**
11 Q. Thank you. What is the maximum weight of a
12 single HI-STORM cask loaded with fuel assemblies used
13 in that report?
14 A. (DR. SOLER) **360,000 pounds.**
15 Q. And did you consider a minimum weight in
16 that report?
17 A. (DR. SOLER) **No, we considered one weight.**
18 Q. With respect to your cask stability analysis
19 for a 10,000-year return period, Holtec report No.
20 HI-2012780, did you use the same maximum weight that
21 you used in the previous report for a single HI-STORM
22 cask loaded with fuel assemblies?
23 A. (DR. SOLER) **Yes.**
24 Q. Back to the 2,000-year report, HI-2012640,
25 did you use values for alpha damping co-efficients in

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1 code MR2V181.EXE for dynamic simulation?
2 A. (DR. SOLER) **No.**
3 Q. Did you use values of beta damping for the
4 same code for dynamic simulation?
5 A. (DR. SOLER) **Yes.**
6 Q. What beta value did you use?
7 A. (DR. SOLER) **That number appropriate to 5**
8 **percent damping.**
9 Q. And with respect to the 2,000-year cask
10 stability analysis HI-2012640, what mathematical model
11 of a single cask was used?
12 A. (DR. SOLER) **A lump mass mathematical model.**
13 Q. And I'm sorry for not recognizing that
14 earlier in your earlier statement.
15 In the 2,000-year cask stability analysis
16 HI-2012640, did you consider any pad-to-pad interaction
17 forces in the cask stability analysis?
18 A. (DR. SOLER) **No.**
19 Q. Do you believe they would impact the cask
20 stability?
21 A. (DR. SOLER) **No.**
22 Q. And why not?
23 A. (DR. SOLER) **Because we took a conservative**
24 **approach that there was nothing surrounding the pad we**
25 **were looking at. If there was any additional**

ANALYTICAL STUDY OF HI-STORM 100 CASK SYSTEM FOR SLIDING AND TIP-OVER POTENTIAL DURING HIGH-LEVEL SEISMIC EVENT

Technical Report No. 01141-TR-001

Revision 0

Prepared for:

Office of The Attorney General, State of Utah

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Report Record

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VERIFICATION: Verification is performed in accordance with EOP 3.4 as indicated below

NA Design review as documented on the following sheet (* Cursory review only. MB 12/11/01)

NA Alternate calculation as documented in attachment ____ or ____

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Mohsin Khan

UT-48129

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1.0 INTRODUCTION

The purpose of this study is to perform sliding and tip-over nonlinear time history analysis of HI-STORM 100 casks. The cask system is planned to sit vertically unanchored on a concrete slab in a 2 by 4 array as described in References 1, 2 and 3. The concrete slab rests on ground soil. This study will use three ground motion time histories for 2000-year return period of the region as provided in Reference 4. The nonlinear dynamic analyses will simulate the cask behavior due to specified input motions applied at the cask base. It will demonstrate the sensitivity of HI-STORM 100 cask response (sliding and rocking) movements due to various input data selected. The results of this study will be compared with the results presented in Reference 1.

2.0 INPUT DATA

The following structural design and analyses information is based on Reference 1, 2 and 3.

Overpack Weight = 267,664 lbs.
Radial Concrete Weight = 163,673 lbs.
Maximum Cask Weight = 360,000 lbs.
MPC Weight (including fuel) = 88,857 lbs.

Length of the Cask = 231.25 inches
Diameter of the Bottom Plate = 132.50 inches
Inside Diameter of the Cask Shell = 72.50 inches
Outside Diameter of the Cask Shells = 132.50 inches

MPC Height = 190.5 inches
MPC Diameter = 68.375 inches
MPC Bottom Plate Thickness = 2.5 inches
MPC Top Plate Thickness = 9.5 inches

Total Vertical Stiffness Between the Pad and Cask = 4.541×10^{8} lbs./inch
Total Horizontal Stiffness Between the Pad and Cask = 4.541×10^{11} lbs./inch

Lowest Coefficient of Friction = 0.2
Highest Coefficient of Friction = 0.8

Two horizontal time histories and one vertical time histories are provided in Reference 4. The amplitudes of these time histories are in terms of gravity (i.e., g) and are at a constant time interval of 0.005 seconds. The time history duration is 30 seconds.

3.0 METHODOLOGY FOR FINITE ELEMENT ANALYSIS

Figure 1 shows a rigid mass than can slide and uplift. Various forces and local stiffnesses needed to solve this problem are also shown. The block moves when inertial forces acting on this mass exceed the frictional force μW. Local contact stiffnesses in the horizontal and

vertical direction (K_H and K_V) are needed in mathematical simulation just before any sliding occurs at any instant of time. There are several structural analyses codes, which can solve this nonlinear analysis problem. SAP2000 and ANSYS structural analysis codes described in References 5 and 6 are used to perform nonlinear time history analyses. The methodologies and the formulation of the finite elements and nonlinear solution methods are described in these references. Three mathematical models considered in this study, are described in section 4.0.

4.0 CASK MODEL STUDIES

Three mathematical cask models with varying degree of complexity are considered in this study as follows:

- The purpose of the first model is to obtain horizontal sliding displacements for the HI-STORM100 cask without any vertical excitation. For this case, two industry standard structural analysis codes, ANSYS and SAP2000, are used to compare the maximum sliding displacements values and the nodal displacement time history traces for benchmarking purposes.
- The second model includes the effect of vertical excitation without any rocking effects due to cask height.
- The third model represents a three-dimensional cask with vertical and horizontal rigid beam finite elements. The mass density of the vertical beam elements are adjusted to obtain approximately the weight of a fully loaded HI-STORM100 cask.

For the last two models, several parametric analyses are performed to show the effect of change of contact stiffnesses and coefficient of friction values on the HI-STORM100 cask motion. The effect of use of various structural damping values (ALFA, BETA, or Modal) is also studied in the third model.

4.1 Rigid Mass Sliding Model

A HI-STORM 100 cask weighing 360 kips is modeled by a single rigid beam element that can slide horizontally. Figure 2 shows the SAP2000 model. The height of beam element is chosen small enough to avoid any rocking effects. The bottom of this element is connected to a nonlinear element that accommodates sliding effects. At the base of the nonlinear springs, horizontal time histories corresponding to 2000-year return period were applied. For checking the adequacy of these input time histories, corresponding response spectra were generated and were compared with those spectra in Reference 4.

The results of nonlinear sliding displacements are shown in Figures 3 and 4. Figure 5 provides the sliding displacements results obtained from ANSYS Computer Code (Reference 6). Table 1 provides the absolute maximum relative displacement values obtained by the two computer codes. This table shows good agreements between sliding displacement values obtained by SAP2000 and ANSYS computer codes.

Table 1

Absolute Maximum Relative Displacements in Horizontal Directions for Sliding Cask

		Stiffness for Non-Linear Elements		Relative Cask Displacements		
Computer Code	Coefficient of Friction μ	Vertical Stiffness K_V (lbs./in.)	Horizontal Stiffness K_H (lbs./in.)	Horizontal Displacement D_x (in.)	Horizontal Displacement D_y (in.)	Vertical Dispalcement D_z (in.)
SAP2000	0.2	0	1×10^5	9.371	5.599	NA
ANSYS	0.2	0	1×10^5	9.371	5.599	NA

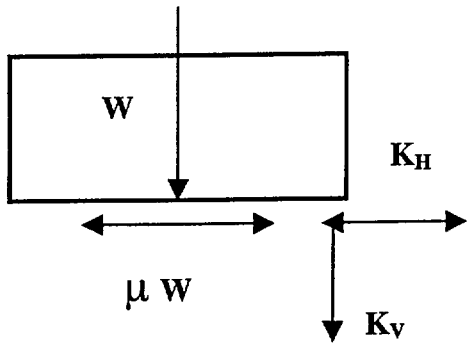

Figure 1
Cask Model for Sliding and Uplift

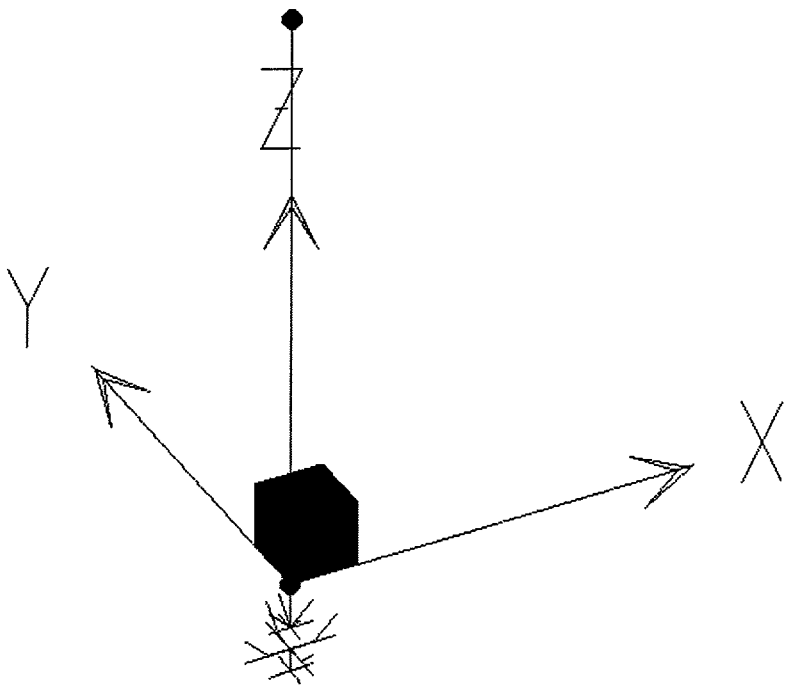

Figure 2
Cask Math Model for Sliding and Uplift

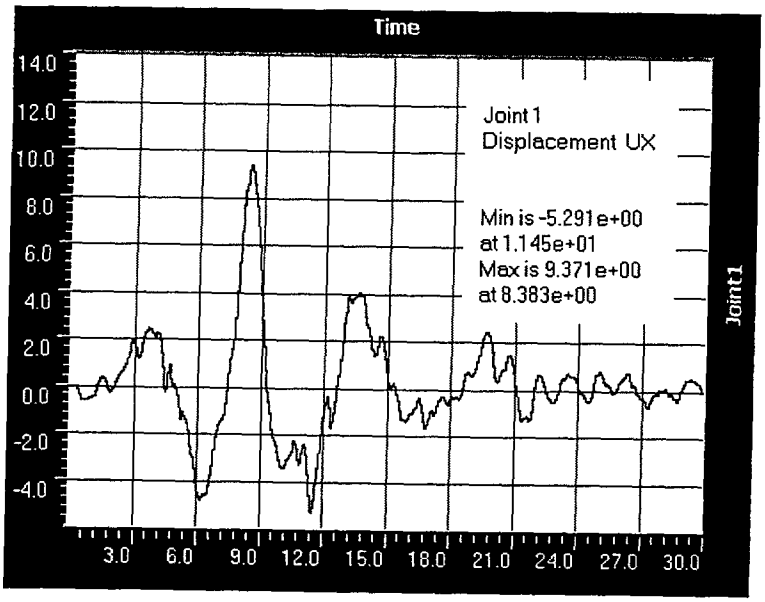

Figure 3
Sliding Cask – Relative Displacement Time History for Horizontal X-Direction

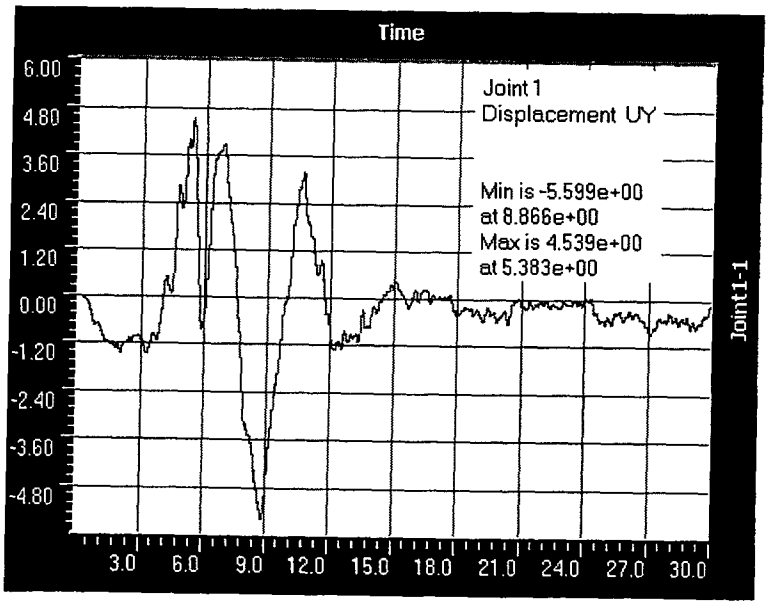

Figure 4
Sliding Cask – Relative Displacement Time History for Horizontal Y-Direction

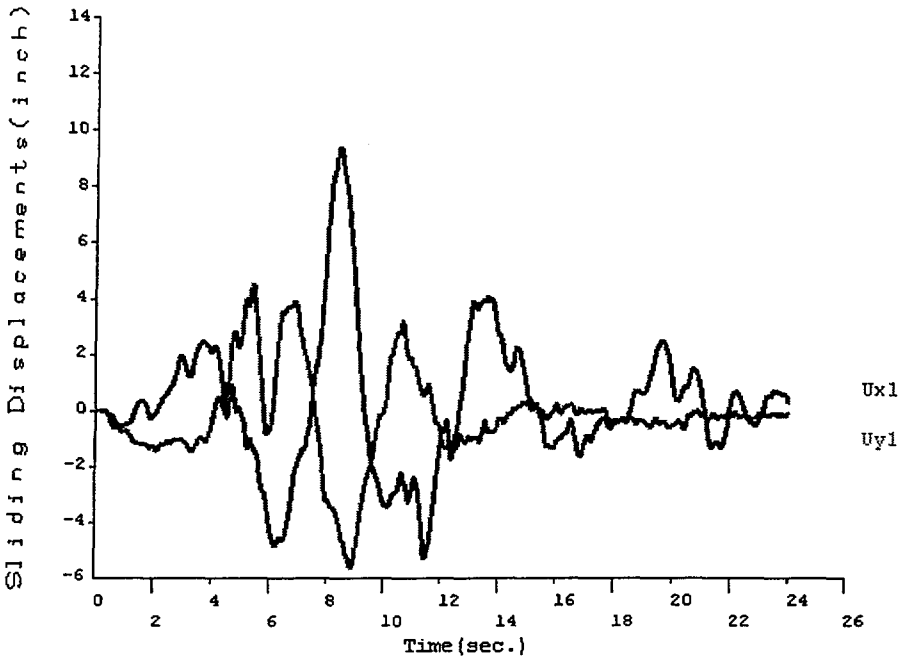

Figure 5
Sliding Cask – Relative Displacement Time History for Horizontal X and Y Directions (ANSYS)

4.2 Sliding and Uplift Model Without Consideration of Rotational Effects

A HI-STORM 100 cask weighing 360 kips is modeled by a small single rigid beam element that can slide and uplift. Figure 2 shows the beam model. The height of this element is chosen small enough to avoid any rocking effects. The bottom of this element is connected to a nonlinear element that can slide and provides compression only nonlinear effect. The rotational effects are ignored. Three time histories corresponding to 2000-year return period were applied at the base of the nonlinear spring.

Starting contact stiffnesses in the horizontal and vertical directions are used as input in performing analyses. These stiffnesses are effective before any sliding or during compression condition. In general, high contact stiffness values are used during numerical simulation of these types of analyses. The real problem with using high stiffnesses is that it artificially treats the solution to be a linear for a significant duration of the input motions. The artificially high stiffnesses also absorb significant amount of energy before sliding actually occurs. The "instantaneous" structural frequencies (before sliding) could also alter the initial sliding velocities, which would affect the final structural response during high level ground input motions. Table 2 summarizes the results of this study. A significant variation in cask displacements are found by selecting different combination of three input parameters; coefficient of friction μ, and local contact stiffnesses in the horizontal and vertical direction (K_H and K_V).

Table 2 shows that as the vertical contact stiffness between the cask and the ISFSI pad increases, the sliding displacements reduce significantly. This effect shows that the cask is approaching a simulated anchored condition to the ISFSI pad and there is no dynamic amplification in the vertical direction. Similarly, as the horizontal contact stiffness increases, the horizontal sliding displacement also reduces. However, the change in vertical stiffness has a greater impact in changing the cask displacements than the horizontal contact stiffness. When vertical displacement is large, small variation in mass also influences the horizontal response siginificantly.

The results presented in Table 2 show that the sliding and uplift displacements are not unique and tend to reduce significantly with the increase in contact stiffness values. Reference 3 also gives a range of displacement solutions, which are not unique. The solution given in Reference 3 would change significantly with lower contact stiffnesses and with lower damping values. The convergent sliding displacements should be obtained by selecting a range of contact stiffnesses. A contact stiffness value of 10^8 lbs/in is considered too high for an unanchored cask that can slide and rock. Ideally, these stiffnesses should be small values for an unanchored cask.

Table 2

Absolute Maximum Relative Displacements for Sliding Cask
Associated with Sliding and Uplift Model Without Consideration of Rocking Effects
Due to Cask Height

		Stiffness for Non-Linear Elements		Relative Cask Displacements		
Study Run Number	Coefficient of Friction μ	Vertical Stiffness K_V (lbs./in.)	Horizontal Stiffness K_H (lbs./in.)	Horizontal Displacement D_x (in.)	Horizontal Displacement D_y (in.)	Vertical Dispalcement D_z (in.)
1	0.2	1×10^6	1×10^5	9.16	12.25	2.10
2	0.8	1×10^6	1×10^5	42.74	31.35	1.75
3	0.2	10×10^6	1×10^5	19.25	7.69	1.93
4	0.8	10×10^6	1×10^5	12.70	14.03	1.94
5	0.2	100×10^6	1×10^5	6.38	3.86	0.006
6	0.8	100×10^6	1×10^5	4.74	3.05	0.006
7	0.2	100×10^6	1×10^6	1.91	1.29	0.006
8	0.8	100×10^6	1×10^6	3.80	1.38	0.006
9	0.2	454×10^6 (**)	1×10^5	6.39	3.86	0.001
10	0.8	454×10^6 (**)	1×10^5	4.83	3.06	0.001
11	0.8	454×10^6 (**)	454×10^6	0.057	0.071	0.001

** Stiffness values correspond to those values given in Reference 1.

4.3 Three-Dimensional Sliding and Uplift/Tip-Over Model

A HI-STORM 100 cask weighing approximately 360 kips is modeled by 72 beam elements. Figure 3 shows a three-dimensional (3D) beam model. The cask can slide, uplift and rock or tip-over under the specified seismic input motions. The base diameter of this model is 132 inches and the cask height is 231 inches. The model is graphically generated using the GUI feature of SAP2000. The bottom 8 nodes are connected to nonlinear elements. Three time histories corresponding to 2000-year return period were applied at the base of the nonlinear springs.

Table 3 summarizes the results of this study. A significant variation in cask displacements found by selecting different combination of three input parameters; coefficient of friction μ, and local contact stiffnesses in the horizontal and vertical direction (K_H and K_V). The cask displacements are comparable to those obtained in Table 2. In this case, solutions are also obtained at 0.01%, 1% and 5% structural damping. In reality, the structural damping (ALFA, BETA or Modal) must be small or insignificant value. Cask sliding and uplift displacements should be independent of structural damping for cask elements, since cask is considered as a rigid body. Only friction should be primary energy dissipation mechanism. From the results presented in Table 3, it is shown that the use of higher damping will significantly lower the nonlinear structural response of the cask system.

Table 3

Absolute Maximum Relative Displacements for Sliding Cask Associated with 3-D Sliding and Tip-Over Model

		Stiffness for Non-Linear Elements		Relative Cask Displacements		
Study Run Number	Coefficient of Friction μ (% Damping)	Vertical Stiffness K_V (lbs./in.)	Horizontal Stiffness K_H (lbs./in.)	Horizontal Displacement D_x (in.)	Horizontal Displacement D_y (in.)	Vertical Dispalcement D_z (in.)
1	0.8 (0.01%)	1×10^6	1×10^5	372.76	229.65	27.24
2	0.2 (1%)	1×10^6	1×10^5	48.69	48.86	2.56
3	0.8 (1%)	1×10^6	1×10^5	306.00	120.45	18.01
4	0.8 (1%)	10×10^6	1×10^5	60.21	46.85	8.59
5	0.2 (5%)	1×10^6	1×10^5	4.38	9.08	2.28
6	0.8 (5%)	1×10^6	1×10^5	21.48	56.85	5.87
7	0.2 (5%)	10×10^6	1×10^5	6.00	4.71	0.15
8	0.8 (5%)	10×10^6	1×10^5	15.11	10.42	0.94
9	0.8 (5%)	100×10^6	1×10^5	27.63	58.27	2.56

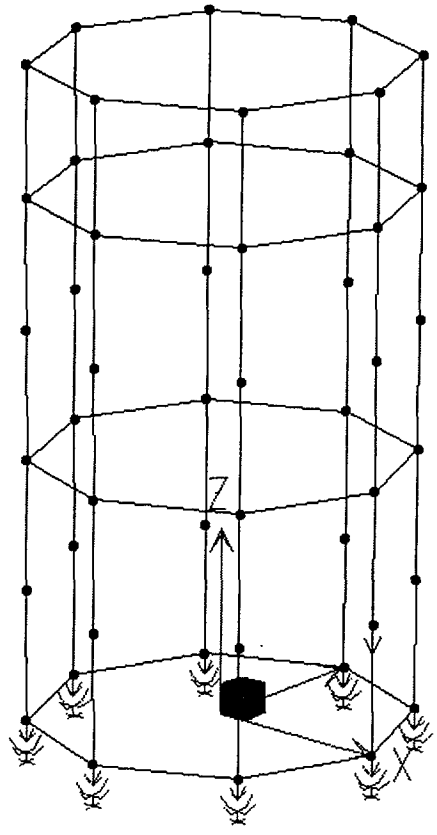

Figure 6
Cask 3-D Math Model for Sliding, Uplift and Rocking

5.0 CASK OVERTURNING/TIPPING

In this section, a calculation is performed to determine the minimum requirement for vertical uplift of the cask body and the kinetic energy (or velocity) required to tip a cask under high level horizontal and vertical earthquake motions. As shown in Figure 7, the cask will tip over when the center of gravity (CG) of the cask body with height (H) and radius (R) moves over one edge.

For HI-STORM 100 Cask:

Height of the Cask (H) = 231.25 inches
Diameter of the Bottom Plate (2R) = 132.50 inches
Maximum Cask Weight = 360,000 lbs.
Cask Velocity = V_M
Horizontal Cask Velocity in X direction = V_X
Horizontal Cask Velocity in Y direction = V_Y
Vertical Cask Velocity in Z direction = V_V
Gravity = 386.4 in./sec.^{2}

$\alpha = \tan^{-1}(H/2R) = \tan^{-1}(231.25/132.50) = 60.2°$

$\beta = 90° - 60.2° = 29.8°$

The value of tipover angle β is conservatively calculated due to selection of cask CG equals to half the cask height.

Maximum Cask Uplift Height = 2 R (sin 29.8°) = 65.85 inches

Change in the C.G Height at Tipping = $\{R^2 + (0.5\,H)^2\}^{1/2} - 0.5\,H = 133.26 - 115.63$
= 17.63 in.

Potential Energy (PE) Required for Cask Overturning:

PE = Cask Weight * Change in the C.G height at tipping
PE = (360,000 lbs.) (17.63 in.) = 6,346,800 lbs.-inch

Kinetic Energy (KE) Required for Cask Overturning:

KE = (1/2) * Cask Mass * (Cask Velocity)^2
KE = (1/2) [360,000/386.4] $(V_M)^2$

Considering all three direction of earthquake motions by using SRSS rule for directional response combination , the (Cask Velocity)^2 is:

$(V_M)^2 = (V_X)^2 + (V_Y)^2 + (V_V)^2$

Table 3 summarizes the spectral accelerations and velocities at 5% damping from 0.4 to 2.0 Hz frequency range for a 2000-year return period seismic event. This is an important frequency range and it is judged that the instantaneous rocking and sliding frequencies lie in this range for an unanchored cask. From Table 4, the average horizontal velocity in this frequency range for both the fault-normal and fault-parallel direction is approximately equal and the average vertical velocity is approximately half of the average horizontal velocities.

Therefore,

$V_X \cong V_Y$ and $V_V \cong 0.5 * V_X$

Substituting these values,

$$(V_M)^2 = (V_X)^2 + (V_X)^2 + (0.5*V_X)^2 \quad = 2.25 * (V_X)^2$$

For Possible Cask Overturning/Tipping:

KE > PE

$$(1/2)\ [360{,}000/386.4]\ (V_M)^2 > 6{,}346{,}800 \text{ lbs.-inch;}$$

$$(1/2)\ [360{,}000/386.4] * 2.25 * (V_X)^2 > 6{,}346{,}800 \text{ lbs.-inch;} \Rightarrow V_X > 77.82 \text{ in/sec.}$$

For some cases summarized in Table 3, the cask velocities obtained from the velocity traces during an earthquake are much higher than the calculated value of V_X. For example, Figure 8 shows a velocity trace for Study Run Number 1 in Table 3. In this case the maximum cask velocity is 271 in/sec.. Therefore, it is possible for HI STORM 100 cask to overturn (i.e., tip over) during a high-level seismic event.

Table 4

Spectral Accelerations and Velocities at 5% Damping From 2000-Year Return Period Seismic Input Time Histories

		Fault-Normal		Fault-Parallel		Vertical	
Period (sec)	Frequency (Hz)	Accelerations (g)	Velocity inch/sec.	Accelerations (g)	Velocity inch/sec.	Accelerations (g)	Velocity inch/sec.
2.500	0.4	0.2020	31.05	0.14228	21.87	0.09851	15.15
2.000	0.5	0.2500	30.75	0.20892	25.70	0.12739	15.67
1.667	0.6	0.3084	31.61	0.28255	28.96	0.14389	14.75
1.429	0.7	0.3604	31.66	0.3794	33.33	0.18618	16.36
1.250	0.8	0.4175	32.09	0.44719	34.38	0.20408	15.69
1.111	0.9	0.4485	30.65	0.50923	34.80	0.23842	16.29
1.000	1.0	0.5851	35.98	0.56703	34.87	0.26506	16.30
0.909	1.1	0.5838	32.64	0.63656	35.59	0.28059	15.69
0.833	1.2	0.6480	33.21	0.71335	36.56	0.33617	17.23
0.769	1.3	0.7524	35.59	0.77482	36.65	0.30952	14.64
0.714	1.4	0.7858	34.52	0.85834	37.70	0.38997	17.13
0.667	1.5	0.7393	30.31	0.94301	38.66	0.39032	16.00
0.625	1.6	0.8890	34.17	0.92933	35.72	0.42693	16.41
0.556	1.8	0.8856	30.26	1.12603	38.47	0.47324	16.17
0.500	2.0	1.0783	33.15	1.17603	36.16	0.56712	17.44
Average Velocity			32.51		33.96		16.06

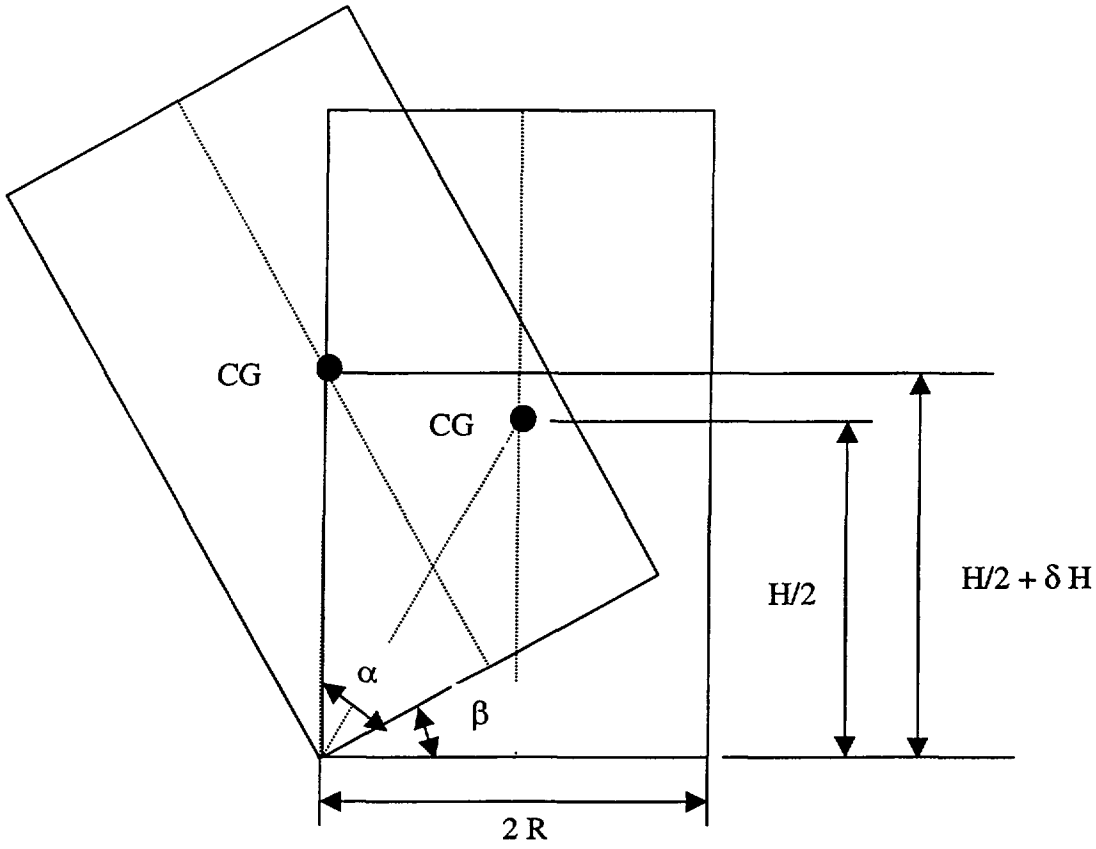

Figure 7
Cask Overturning/Tipping

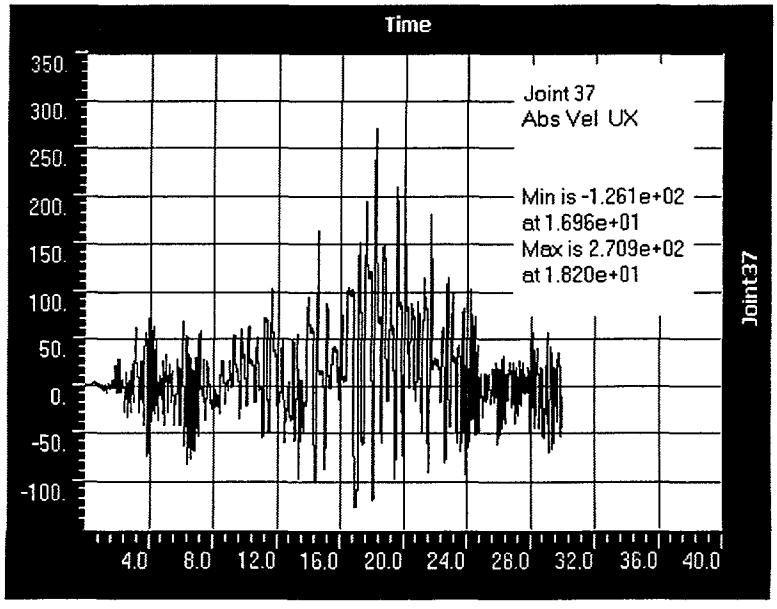

Figure 8
Cask Velocity Trace For Study Run Number 1

6.0 CONCLUSIONS

In this study, evaluation of sliding and tip-over potential of HI-STORM 100 cask under seismic input motion has been performed. The HI-STORM 100 cask movements have been calculated in various ways, from a simple rigid body sliding to a detailed three-dimensional beam model. It has been demonstrated that a wide range of sliding and uplift displacements can be obtained based on the selected input values of the local contact stiffnesses and the coefficient of friction. These local contact stiffnesses are effective before the sliding or uplift begins. In general, high stiffness values are used for solving sliding and tip over type problems. The real issue with this approach is that if these stiffnesses are selected too high, then the program artificially assumes the problem to be linear for a significant duration of the input motion. Using high stiffnesses could absorb some amount of energy before sliding actually occurs. Also, with high stiffnesses the "instantaneous" structural frequencies (before sliding) could also alter the sliding velocities, which would affect the final structural response during high level ground input motion.

Key insights and observations from our study are summarized below.

- The input time histories used for these evaluations were obtained from reference 4 and are based on a 2000-year return period seismic event. Reference 4 time histories did not consider the amplification due to soil structural interaction at the top of ISFSI pad. The soil structural interaction effect would amplify the time history response on top of the ISFSI pad. Therefore, the vertical input motion at the base of the cask would be much higher than those used in this study.

- This study shows that he sliding and rocking displacements are significantly affected by the local stiffness values used as input in the mathematical model simulation. However, the sliding displacements should be independent of the local contact stiffness values used.

- This study shows that using initial high local contact vertical stiffness in Reference 1, 2, 3 significantly minimizes the vertical displacements. This approach also helps in significantly reducing the horizontal sliding displacements.

- For an unconstrained cask that acts as a rigid body, only frictional forces should provide energy dissipation mechanism during sliding and rocking.

- This study shows that the sliding and rocking displacements are significantly affected by using higher damping values during numerical simulation. Reference 1 used a high beta damping value. Use of high structural damping (ALFA, BETA, or Modal) must be avoided since the cask acts as a rigid body. A very small damping value should be used for numerical stability purposes.

- From the numerical study, it is shown that the sliding results presented in References 1, 2, and 3 are unconservative and are not unique. Due to wide variation in cask response obtained based on input data used, it is recommended that the sliding displacements presented in References 1, 2 and 3 be based on mathematical models that have been benchmarked with actual or prototype models based on shake table test data.

- Since the vertical response alters the cask dynamic reactions significantly, the computed cask reactions during cask uplift/drop and impact due to vertical amplifications would be much higher than those used in the pad design. See Reference 1.
- It is possible that for high-level ground input motion, the HI STORM 100 cask could overturn (i.e., tip over).

7.0 REFERENCES

1. Excerpt from Holtec Report No. HI-971631, *Multi-Cask Seismic Response at PFS ISFSI for Private Fuel Storage L.L.C.*, Rev. 0, prepared by Holtec International, May 1997.
2. Excerpt from Holtec Report No. HI-2012653, *PFSF Site-Specific HI-STORM Drop/TipOver Analyses*, Rev 1, prepared by Holtec International, May 2001.
3. Excerpt from Holtec Report No. HI-2012640, *Multi Cask Response at PFS ISFSI From 2000-YR Seismic Event*, Rev. 2, prepared by Holtec International.
4. *Development of Design Ground Motions for the Private Fuel Storage Facility*, Revision 1, prepared by Geomatrix Consultants, Inc., March 2001.
5. SAP2000 User's Manual, Version 7.4, Computers and Structures, Inc.
6. ANSYS Version 5.7 Computer Code, ANSYS Inc.

STATE OF UTAH'S PREFACE TO TESTIMONY OF WALTER J. ARABASZ ON CONTENTION UTAH L/QQ - Seismic Exemption

I. Background

A. PFS/Staff have presented a "moving target" as to the seismic design basis ground motions:

1. In original 1997 SAR, PFS uses a deterministic approach.
2. April 1999 Seismic Exemption, PFS requested to calculate DBE using PSHA and 1,000-year recurrence interval (mean return period, "MPR").
3. Staff did not find 1,000-year MRP acceptable but accepted 2,000-year MRP.
   a. 2,000-year MRP accepted by Staff first in PSER, Dec. 1999 then FSER (Sept. 2000).
4. March 2001 PFS changed site-response modeling because of significant changes in 2,000-year MRP ground motions - Peak horiz accelerations increased from 0.528 g to 0.711g.
5. November 2001 PFS documented for the first time its own case for justifying DBE with 2,000-year MRP.
6. Staff continues to rely on same rationale in SSER (Dec. 2001) as it did in the FSER.

II. Unified Contention L/QQ, Section E.

A. Subsection 1. SECY-98-126 Rulemaking Plan provides useful guidance in selection DBE.

1. In the new rulemaking (SECY-01-0178) Commission will solicit comments on range of probability of exceedance levels from 5.0E-4 through 1.-0E-04 and it has directed Staff to undertake further analysis to support specific proposal.
2. Key contested issue is the validity of PFS's claim that it has met the Commission's requirement to show that the 2000-year design standard is sufficiently protective of public safety and property.
3. Sufficient protection depends on **both** the probability of occurrence of the seismic event and the level of conservatism incorporated into design of SSCs.
4. Whether PFS has demonstrated adequate conservatism in design of its SSCs is deferred to other testimony (Dynamic Analysis, Cask Stability and Lack of Design Conservatism).

B. Radiation exposure, part of sub-section 1 and sub-section 2 is deferred to testimony of Dr. Marvin Resnikoff.

C. Subsection 3. The issue of using median and mean probability of exceeding SSE is based on the Staff's faulty reasoning in its use of Reg. Guide 1.165.

D. Subsection 4. Staff's reliance on DOE-STD 1020-94.

1. DOE-STD 1020-94 available to Staff when it drafted SECY-98-126.
2. Inconsistency between Staff's acceptance of 2,000-year MRP in DOE-STD 1020-94 for PFS's exemption request and 10,000-year MRP in SECY-98-126.
3. Staff ignores that DOE-STD 1020-94 design approach is fundamentally coupled to a target seismic performance goal of 1×10^{-4}.
4. Determination of the mean annual exceedance probability of a DBE for the proposed PFS facility, and whether it ensures sufficient protection, cannot be made independent of an evaluation of design conservatism (or non-conservatism).

E. Subsection 5. Staff's grant of exemption to INEEL (TMI-2) ISFSI.

1. Circumstances specific to DOE site do not present a compelling precedent for PFS.
   a. Design basis ground motions of 0.36g for existing higher risk facility (ICCP) at the TMI-2 ISFSI host site was a consideration of the return period for the INEEL ISFSI.
   b. For the INEEL ISFSI there was a site specific radiological risk analysis and finding that there was a lack of credible mechanism to cause a failure.
2. Design value at INEEL was higher than 2000-year return period mean ground motion

from PSHA - it was on the order of 3,000-4,000 years.

3. SECY-98-126, issued two months after INEEL exemption granted, allowed only 1,000-year or 10,000-year return periods, depending on risk.
4. Sequence of events suggests that INEEL does not serve as a precedent for other Part 72 seismic exemptions.

F. Subsection 6. Inadequate conservatism - higher standard for some structures in Utah and Staff reliance on 20 year license term instead of 40 year operational life.

1. Defer conservatism to engineering and dynamic analyses expert testimony.
2. Staff's comparison of PSHA's conducted in Utah are irrelevant.
3. Staff's comparison that the probability of exceedance of DE at an ISFSI (20 year operation) is the same as for pre-closure facility at Yucca Mountain (100 years operation) means that an ISFSI with a 40 year operational life would have a DEB with a 3,980-year MRP.

G. Seismic hazards in areas of public safety take into account not just annual frequencies of occurrence but the exposure time.

1. Risk-acceptance decision specific to the PFS site is not the same as societal global safety strategy.

III. Conclusion:

A. Key contested issue: whether the 2,000-year design standard is sufficiently protective of public safety and property.

B. Critical part of PFS's claim of sufficient protection is deferred to the State's engineering and dynamic analysis experts who challenge PFS's lack of design conservatism.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of:	)	Docket No. 72-22-ISFSI
PRIVATE FUEL STORAGE, LLC (Independent Spent Fuel Storage Installation)	)	ASLBP No. 97-732-02-ISFSI
	)	April 1, 2002

STATE OF UTAH TESTIMONY OF DR. WALTER J. ARABASZ REGARDING UNIFIED CONTENTION UTAH L/QQ (Seismic Exemption)

Q. 1: Please state your name, affiliation, and qualifications.

A. 1: My name is Dr. Walter J. Arabasz. I am a Research Professor of Geology and Geophysics at the University of Utah in Salt Lake City, Utah, and also Director of the University of Utah Seismograph Stations. I have more than 30 years' professional experience in scientific research, consulting, occasional teaching, and publishing articles in observational seismology, seismotectonics, and earthquake hazard analysis with a primary focus on Utah and the Intermountain West.

Since 1977 I have routinely provided professional consulting services on earthquake hazard evaluations for dams, nuclear facilities, and other critical structures and facilities. Since the mid-1980s I have been directly involved in methodology development and applications of probabilistic seismic hazard analysis. During the past decade I have had major involvement in assessing vibratory and fault-displacement hazards for the high-level nuclear waste repository at Yucca Mountain, including serving on a Peer Review Group for Early Site Suitability Evaluation, reviewing technical reports, and serving on expert teams for seismic source characterization for probabilistic hazard analyses.

My service on numerous national and state advisory boards and panels has included – relevant to this filing – serving on the National Research Council's Panel on Seismic Hazard Evaluation (1992-96), the Utah Seismic Safety Commission (1994 to present; chair, 1997-2001), and numerous panels and work groups under the National Earthquake Hazards Reduction Program since the early 1980s. My *curriculum vitae* is included as State's Exhibit 123.

Q. 2: What is the purpose of your testimony?

A. 2: The purpose of my testimony is to explain the basis for my professional opinion that, within my areas of expertise, the NRC Staff has presented a flawed rationale for recommending that Private Fuel Storage, LLC ("PFS") be granted an exemption from existing regulations and to elaborate on the rationale that PFS has recently presented to support its exemption request.

Q. 3: Describe, generally, your role in assisting the State in the PFS proceeding.

A. 3: I was designated one of the State's testifying experts with respect to Contention Utah L, Basis 2. This portion of the original geotechnical contention, Utah L, was incorporated into the unified contention Utah L/QQ as section B and has been resolved by stipulation. I have also been designated as a testifying witness for Section E of the unified contention; this portion of the unified contention is a consolidation of Utah L, Part B and it deals with PFS's request to the NRC to be exempted from basing their seismic design on the results of a deterministic seismic hazard analysis; instead PFS requests that it be allowed to base its seismic design on results from a probabilistic seismic hazard analysis for a 2,000-year return period.

My involvement in the PFS proceeding as a technical expert for the State has included review of the Applicant's SAR sections, and updates thereof, relating to its earthquake hazards investigation of the proposed site and relevant reports and other documents prepared by the Applicant or its contractors and submitted to the NRC or produced to the State in discovery; assisting the State in answering and preparing discovery; review of the NRC Staff's preliminary, final and supplemental Safety Evaluation Report ("SER") for the PFS facility[1] as well as the Staff's Position on Utah L (April 28, 2000).

In response to PFS's request to be exempted from 10 CFR § 72.102(f)(1), I assisted the State in preparing late filed contentions to modify basis 2 of Contention Utah L.[2] I was deposed by Private Fuel Storage, LLC ("PFS") on October 31, 2001 and I was present when, shortly thereafter, the State deposed PFS witness, Dr. C. Allin Cornell, on the appropriateness of using probabilistic seismic hazard methodology with a 2,000-year return period. When PFS filed for Summary Disposition of Utah L, Part B (November 9, 2001), I gave my primary attention to PFS's Motion, its Statement of Material Facts, and the attached declaration of Dr. C. Allin Cornell, and I provided a declaration in support of the State's

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[1] Dated December 15, 1999, September 29, 2000 and December 21, 2001 respectively.

[2] The State filed modification requests on January 26, 2000 and November 9, 2000.

December 7, 2001 Response and Opposition thereto.

Q. 4: Please describe the evolution of the seismic design basis ground motions at the PFS site?

A. 4: I first became involved in providing technical expertise to the State of Utah regarding seismic hazards at the PFS facility in August 1998. Since then, considerations by both the Applicant and the NRC Staff regarding the seismic design basis ground motions – or, for simplified reference, the design basis earthquake ("DBE") – for the PFS facility have continually evolved, providing a "moving target" for critical evaluation. Some of the noteworthy stages in this process include:

1. PFS's submission of its Safety Analysis Report in 1997 in which a "deterministic" approach was used for establishing the DBE aimed at meeting requirements of 10 CFR 72.102(f)(1).

2. PFS's Request for Exemption to CFR 72.102(f)(1) (April 2, 1999) in which PFS requested to calculate the DBE using a probabilistic seismic hazard analysis ("PSHA") and a 1,000-year recurrence interval.

3. The Staff's review of the PFS's request and finding that use of a 1,000-year return-period value was not acceptable – but that use of a PSHA with a 2,000-year return-period value could be acceptable for reasons provided by the Staff (Staff's Preliminary SER ("PSER") (December 15, 1999) at 2-44 to 2-45.

4. The Staff's finding the PSHA with a 2,000-year return period acceptable (Final SER, "FSER", September 29, 2000, at 2-41 to 2- 42);

5. PFS's changes in site-response modeling for the PFS site in March 2001, which resulted in significant changes to the 2,000-year return-period ground motions, including an increase in the peak horizontal acceleration from 0.528 g to 0.711g (*see* SER Supplement No. 2, "SSER", December 21, 2001) at 7 and 21-23).

6. PFS's Motion for Summary Disposition of Utah L Part B (November 9, 2001) in which PFS has presented, for the first time in a documented way, its own case for justifying a DBE with a 2,000-year mean return period ("MRP").

7. The Staff's continued reliance on the same rationale in its SSER as in the FSER to find acceptable a PSHA with a 2,000-year return period – despite many concerns raised by the State regarding non-conservatism in the engineering design of the PFS facility.

Q. 5: Please describe the framework of your testimony.

A. 5: I will frame my testimony as follows. First, I will briefly revisit the original issue of a deterministic seismic hazard analysis ("DSHA"). Then I will address those issues, within my scope of expertise and testimony, associated with unified Contention Utah L/QQ, Section E. In my testimony I will address issues that arose directly from arguments put forward by the Staff to justify a seismic exemption for the PFS facility (allowing a probabilistic DBE with a 2,000-year MRP) as well as new issues, relevant to my area of expertise, raised in PFS's Summary Disposition Motion. I might add that PFS's Motion for Summary Disposition provides the latest rationale offered by PFS for its seismic exemption request.

Q. 6: Do you have any comments about the Deterministic Seismic Hazard Analysis for the PFS site?

A. 6: In previous submissions to the NRC, I stated that PFS had not conducted a fully deterministic seismic hazard analysis ("DSHA") as required by 10 CFR § 72.102(f)(1) and, by reference, 10 CFR 100 Appendix A. The NRC Staff has acknowledged that the DSHA performed by Geomatrix Consultants, Inc. for the PFS facility and reported in the 1997 SAR and the updated DSHA reported in April 1999 "did not meet the deterministic requirements in 10 CFR 100 Appendix A."[3]

A later updated DSHA by Geomatrix Consultants, Inc. reported in April 2001 follows the same methodology as earlier and presumably would also not meet the deterministic requirements of 10 CFR 100 Appendix A.

The relevance of a valid DSHA, other than being required by current NRC regulations, is that it establishes a benchmark to which results of any probabilistic seismic hazard analysis can correctly be compared to evaluate the conservatism of the PSHA results, such as earlier done for the NRC Staff by Stamatakos et al.[4]

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[3] NRC Staff's Objections and Responses to the "State of Utah's Sixth Set of Discovery Requests Directed to the NRC Staff (Utah Contention L)" (February 14, 2000), Response to Requests for Admissions 1 and 2 at 7-8.

[4] *See* Stamatakos, Chen, McCann & Chowdhury, *Seismic Ground Motion and Faulting Hazard at Private Fuel Storage Facility in the Skull Valley Indian Reservation, Tooele County, Utah – Final Report* (September 1999) at 2-46.

Q. 7: Please describe your concerns about Subsection E.1 of the Unified Contention Utah L/QQ.

A. 7: Unified Contention L/QQ, Sub-section E.1 states:

> The requested exemption fails to conform to the SECY-98-126 (June 4, 1998) rulemaking plan scheme, i.e., only 1000-year and 10,000-year return periods are specified for design earthquakes for safety-important systems, structures, and components (SSCs) – SSC Category 1 and SSC Category 2, respectively – and any failure of an SSC that exceeds the radiological requirements of 10 C.F.R. § 72.104(a) must be designed for SSC Category 2, without any explanation regarding PFS SSC compliance with section 72.104(a).

The scope of my testimony with respect to subsection 1 excludes radiological dose consequences. Subsection 2, which also deals with radiological dose limits, is similarly outside the scope of my testimony.

The State has challenged the NRC Staff's proposal to grant an exemption request to PFS that would allow use of a DBE with a 2,000-year return period; the State argued, in part, that the NRC Rulemaking Plan set forth in SECY-98-126 (June 4, 1998) provides only two alternatives for design basis ground motions: a 1,000-year return period or a 10,000-year return period.[5] The Staff has rejected the use of a 1,000-year return period. FSER at 2-41. The Commission has instructed that the State "may not rely solely on the rulemaking plan [SECY-98-126] to prove its contention." CLI-01-12, 53 NRC 416 (June 14, 2001), slip op. at 16. At the same time, the Commission instructed that "PFS is not bound by the rulemaking plan, but it does have the burden to show that the 2000-year design standard is sufficiently protective of public safety and property." Id.

In its Motion for Summary Disposition PFS argued, in part, that non-compliance of a 2,000-year return period with SECY-98-126 is now mooted because the Staff has recommended a Modified Rulemaking Plan in which use of a DBE with a 2,000-year MRP is proposed for dry-cask ISFSIs. Whether the latter indeed moots the issue is questionable in light of the Commission's recent issuance of Staff Requirements Memorandum[6] relating to SECY-01-0178 (September 26, 2001), wherein the Commission writes:

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[5] State of Utah's Request for Admission of Late-filed Modification to Basis 2 of Utah Contention L (November 9, 2000) ("Request for Modification of Utah L") at 6-7.

[6] Staff Requirements Memorandum to William D. Travers dated November 19, 2001, included as State's Exhibit 124.

> Central to this rulemaking is the determination of the mean annual exceedance probability of an earthquake at a proposed ISFSI. The proposed rule should solicit comment on a range of probability of exceedance levels from 5.0E-04 through 1.0E-04. Staff should undertake further analysis to support a specific proposal.

Q. 8: What do you consider to be the key issue in subsection 1?

A. 8: The key contested issue linked to subsection 1 is the validity of PFS's claim that it has met the Commission's requirement to show that "the 2000-year design standard is sufficiently protective of public safety and property."[7] PFS's claim fundamentally rests on the proposition that sufficient protection "depends on both the probability of occurrence of the seismic event (often expressed as the mean annual probability of exceedence or "MAPE" of a given earthquake level) and the level of conservatism incorporated in the design procedures and criteria."[8] I agree with the proposition – but the latter critical part of PFS's claim of sufficient protection is challenged by the State's engineering and dynamic analyses experts, who dispute PFS assertions that it has demonstrated adequate conservatism in design of SSCs at the PFS facility. Here, and ultimately at the end of my testimony, I defer to these experts for more complete discussion of their disputes, which go the heart of "appropriately conservative" and "sufficiently protective" design of the PFS facility. *See* Testimony of Dr. Steven F. Bartlett and Dr. Farhang Ostadan (Dynamic analysis); Dr. Ostadan and Dr. Mohsin R. Khan (Cask stability), and Dr. Ostadan and Dr. Bartlett (Lack of design conservatism) (hereafter "Engineering and Dynamic Analyses Expert Testimony").

Q. 9: Please describe your concerns about Subsection E.3 of the Unified Contention Utah L/QQ.

A. 9: Subsection E.3 of the unified Contention Utah L/QQ, states:

> The staff's reliance on the reduced radiological hazard of stand-alone ISFSIs as compared to commercial power reactors as justification for granting the PFS exemption is based on incorrect factual and technical assumptions about the PFS facility's mean annual probability of exceeding a safe shutdown earthquake (SSE), and the relationship between the median and mean probabilities for exceeding an SSE for central and eastern United States commercial power reactors and the median and mean

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[7] PFS's Motion for Summary Disposition at 10.

[8] Id. at 6.

probabilities for exceeding an SSE for the PFS facility.

In its Request for Modification of Utah L, the State evaluated the rationale put forward by the Staff in its September 2000 SER to justify a DBE with a 2,000-year return period for the PFS facility and characterized the Staff's reasons as *ad hoc* and either flawed or not compelling.[9] Subsection 3 concerns a series of three statements made by the Staff leading to the conclusion: "On the basis of the foregoing, the mean annual probability of exceedance for the PFS Facility may be less than [sic] 10^{-4} per year." FSER at 2-42. The Staff's flawed reasoning, as presented, was to posit that a design ground motion (for an SSE) at the PFS site which had a median reference probability of exceedance of 10^{-5} as defined in Regulatory Guide 1.165 would be the same as a design ground motion with a mean annual probability of exceedance of 10^{-4}.

Q. 10: Do you believe there has been a misperception about the issue the State has raised in Sub-section 3?

A. 10: Yes I do.

In support of PFS' Motion for Summary Disposition, Dr. Cornell challenges Subsection 3 – formerly Utah L, Part B, Basis 3 – on various grounds and concludes that "the argument raised by the State in Basis 3 is inconsequential and irrelevant to the issue whether a 2,000-year earthquake should be used at the PFSF." Declaration of C. Allin Cornell ("Cornell Dec.") at ¶40. What remains relevant is the benchmark for an SSE at the PFS site if the DBE for an ISFSI is to be compared to that benchmark, as was done by the Staff in its September 2000 SER. Absent a determination by the Staff along the lines of Dr. Cornell's beliefs of what the Staff "today would both select and prefer" (Cornell Dec. ¶35), or "can reasonably be expected to revert to" (id. ¶37), or "would likely conclude" (id. ¶38), or "would today not only accept but prefer" (id. ¶39), the State relied on guidance in Regulatory Guide 1.165 and on corresponding commentary by the Staff. Murphy et al., *Revision of Seismic and Geologic Siting Criteria*, Transactions of the 14th International Conference on Structural Mechanics in Reactor Technology (August 17-22, 1997), 1-12, included as State's Exhibit 125.

Dr. Cornell states that "The provision in Regulatory Guide 1.165 that a median value of 10^{-5} could be used is only the result of historical circumstances . . . [involving] a significant discrepancy in the assessment of the mean estimates between the two major CEUS seismic hazard studies then available . . . [which has] since been resolved . . ." (Cornell Dec. ¶36). This assertion is at odds with the following commentary by the Staff in 1997:

> It should be noted that this RP [Reference Probability of 1E-5/yr] is

[9] Request for Modification of Utah L at 7.

> calibrated with the past design bases, it is not derived directly from any quantitative risk or safety goals. In fact, one of the reasons for using the median hazard curve in the regulatory guide approach is that the controlling earthquakes resulting from the de-aggregation of the median hazard curve are very similar to those used in the past licensing from the deterministic procedures.

Murphy et al. (1997) op. cit. at 7.

A similar commentary by the Department of Energy notes the following:

> In developing Regulatory Guide 1.165, NRC staff considered whether to define the reference probability as a mean or median value. The mean value has the advantage of better reflecting the uncertainty in the seismic hazard evaluation (i.e., it is sensitive to the range of interpretations of seismic source zone configurations, earthquake magnitude recurrence relationships, and ground motion attenuation relationships). However, precisely because the median is less sensitive to uncertainties, it provides a more stable regulatory benchmark than does the mean. Another consideration leading to the staff's preference for the median was the finding that, when median hazard curves were disaggregated, the magnitudes and distances of the controlling earthquakes tended to be more sharply defined and to agree better with the safe shutdown earthquakes of the selected plants than when mean hazard curves were disaggregated (Bernreuter et al. 1996).

DOE Topical Report YMP/TR-003-NP, 1997) at §3.1.2.1; *see* Exh. 3 to Cornell's Dec. in PFS's Motion for Summary Disposition at pages 2-3 of 7.

From the above discussion, it is not the State's argument that a median estimate should be used "in lieu of the mean estimate for the design of nuclear power plants, and similarly for ISFSIs . . ." PFS's Statement of Material Facts on Which No Genuine Dispute Exists at ¶19. Rather, the argument rests with the Staff's guidance in Regulatory Guide 1.165. Therein the procedure is specified for determining the reference probability, the annual probability of exceeding the SSE, at future nuclear power plants: "This reference probability [median annual exceedance probability of 1.0E-05] is also to be used in conjunction with sites not in the Central and Eastern United States (CEUS) . . . However, the final SSE at a higher reference probability may be more appropriate and acceptable . . . for some sites . . . Reference B.4 includes a procedure to determine an alternative reference probability on the risk-based considerations; its application will also be reviewed on a case-by-case basis." Regulatory Guide 1.165 at 12.

Q. 11: Please describe your concerns about Subsection E.4 of the Unified Contention Utah L/QQ.

A. 11: Subsection E.4 of the unified contention Utah L/QQ, states:

> In supporting the grant of the exemption based on 2000-year return period, the staff relies upon United States Department of Energy (DOE) standard, DOE-STD-1020-94, and specifically the category-3 facility SSC performance standard that has such a return period, notwithstanding the fact the staff categorically did not adopt the four-tiered DOE category scheme as part of the Part 72 rulemaking plan.

The Staff's reliance on DOE-STD-1020-94 in its December 1999 PSER, its September 2000 FSER and its December 21, 2000 SSER to justify a DBE with a 2,000-year return period for the PFS facility suffers from two circumstances. First, DOE-STD-1020-94 was fully available to, and was referenced by, the Staff when it drafted its 1998 Rulemaking Plan (SECY-98-126). Yet the Staff chose in its 1998 Rulemaking Plan not to propose the use of a 2,000-year return period for ISFSIs. Second, the Staff cited the 2,000-year return period (mean annual probability of exceedance of 5×10^{-4}) for Performance Category-3 ("PC3") SSCs without acknowledging that in the design approach of DOE-STD-1020-94, the MAPE for PC3 is fundamentally coupled to a target seismic performance goal of 1×10^{-4} (the annual probability of exceedance of acceptable behavior limits). DOE-STD-1020-94 at B-7 to B-8.

PFS's Motion for Summary Disposition is replete with acknowledgments that, just as in the overall design approach of DOE-STD-1020-94, there should be a coupling of the hazard exceedance probability and a level of conservatism in design procedures that together ensure a desired performance goal. For example:

> [T]he risk of failure of a facility or structure depends on both the probability of occurrence of the seismic event (often expressed as the mean annual probability of exceedence or "MAPE" of a given earthquake level) and the level of conservatism incorporated in the design procedures and criteria. Cornell Dec. ¶13.

PFS's Motion for Summary Disposition at 6.

> As discussed above, the level of safety achieved depends on both the earthquake threat definition and the design procedures and criteria utilized to protect against that threat; thus, looking only at the earthquake return period is incorrect.

Id. at 15.

> Two factors are relevant to determining the likelihood of seismic failure of a facility or structure due to an earthquake event. These are (1) the seismic design basis earthquake ("DBE") for the facility or structure and (2) the conservatisms embodied in the codes and standards applicable to its seismic design. Cornell Dec. ¶¶18-19; see also Arabasz Dep. at 41-42, 81-84, 115-117.

PFS's Statement of Material Facts on Which No Genuine Dispute Exists, ¶12.

> While the risk-graded approach is implemented in somewhat different ways in the various fields of seismic design, the standards of practice almost invariably utilize a DBE defined at some mean annual probability of exceedance and a set of design procedures and acceptance criteria.

Cornell Dec. ¶18.

> Both the MAPE of the DBE and the level of conservatism incorporated in the design procedures and criteria affect the failure probability of seismically-designed facilities and structures. . . . [I]t is important to understand that both the MAPE and the level of conservatism in the design procedures and criteria must be considered when assessing and comparing the safety implications of various seismic design standards.

Cornell Dec. ¶19.

The discovery and deposition process for Contention Utah L, Part B, has led me to the opinion that determination of the mean annual exceedance probability (or equivalent return period) of a DBE for the proposed PFS facility, and whether it ensures sufficient protection, cannot be made independent of an evaluation of conservatism (or non-conservatism) in design procedures.

Q. 12: Do you have any comment on DOE Standard 1020-01?

A. 12: A final point of particular relevance to Subsection 4 is the recent release of Revised DOE Standard 1020-2001 for review and comment. Memorandum from Richard L. Black to Technical Standards Program Managers dated August 22, 2001. For PC3 the revised standard changes the MAPE from 5×10^{-4} (2,000-year return period) to 4×10^{-4} (2,500-year return period) while retaining the same target seismic performance goal of 1×10^{-4} per year for sites not near tectonic plate boundaries. Revised DOE-STD-1020-2001,

Table G-3 at G-6, included as State's Exhibit 126. The new DOE-STD-1020-2001 was released before the Staff issued the SSER yet the Staff makes no mention of it and still relies on the 1994 version.

Q. 13: Please describe your concerns about Subsection E.5 of the Unified Contention Utah L/QQ.

A. 13: Subsection E.5 of the unified contention Utah L/QQ, states:

> In supporting the grant of the exemption based on 2000-year return period, the staff relies upon the 1998 exemption granted to DOE for the Idaho National Engineering and Environmental Laboratory (INEEL) ISFSI for the Three Mile Island, Unit 2 (TMI-2) facility fuel, which was discussed in SECY-98-071 (Apr. 8, 1998), even though that grant was based on circumstances not present with the PFS ISFSI, including (a) existing INEEL design standards for a higher risk facility at the ISFSI host site; and (b) the use of a peak design basis horizontal acceleration of 0.36 g that was higher than the 2000-year return period value of 0.30 g.

In my opinion, circumstances specific to the seismic exemption awarded to DOE for the TMI-2 ISFSI at INEEL (SECY-98-071, April 8, 1998) do not justify using the exemption as a compelling precedent for the PFS exemption request.

The design basis of an existing higher risk facility, namely the Idaho Chemical Processing Plant ("ICPP"), at the host site for the TMI-2 ISFSI was a definite consideration in DOE's proposal of a DBE for the ISFSI.[10] Under existing DOE design standards at INEEL, based on DSHA results from the 1970s, the peak design basis horizontal acceleration for the ICPP was set at 0.36 g, including effects of soil amplification.[11] DOE proposed to use the same acceleration for the DBE for the TMI-2 ISFSI. In an analysis for the NRC, the regulatory problem was stated this way:

> [T]he DOE-proposed design PHA of 0.36 g does not bound the most recent 84th-percentile deterministic value of 0.56 g and 10,000-yr return period probabilistic value of 0.47 g. Therefore, a judgment

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[10] Chen and Chowdhury, *Seismic Ground Motion at Three Mile Island Unit 2 Independent Spent Fuel Storage Installation Site in Idaho National Engineering and Environmental Laboratory – Final Report* (June 1998), excerpts included as State's Exhibit 127, at 4-1.

[11] Id.

> of whether the DOE-design approach is acceptable depends on whether there are regulatory and technical bases to accept an ISFSI-design value that bounds the 50th-percentile deterministic value and the 2,000-yr return period probabilistic value.[12]

Ultimately, DOE was allowed to use a design earthquake with 0.36 g peak horizontal acceleration (together with an appropriate response spectrum) for the TMI-2 ISFSI. SECY-98-071 at 3. What the NRC approved in terms of a design-basis ground motion was a design value higher than the 2,000-year return period mean ground motion from the PSHA. In their analysis for the NRC, Chen and Chowdhury provided information showing that the 0.36 g horizontal design value for the ISFSI soil site lies between the 2,000-year probabilistic value of 0.30 g and the 10,000-year probabilistic value of 0.47 g. Id. at 3-5 (State's Exh. 127). Although the report by Chen and Chowdhury does not contain sufficient information to identify precisely the return period corresponding to 0.36 g on soil, the bounding probabilistic values for 2,000 years (0.30 g) and 10,000 years (0.47 g) suggest that 0.36 g corresponds to a return-period value on the order of three to four thousand years (the precise return period would have to be determined from the original PSHA data). Thus, a 2,000-year return period for the PFS facility would be significantly lower than what was approved for the INEEL ISFSI.

Another factor that significantly influenced the Staff's approval of the TMI-2 ISFSI exemption was a site-specific radiological risk analysis coupled with "the lack of a credible mechanism to cause a failure." SECY-98-071 at 3.

On April 8, 1998, the NRC informed the DOE, "Since the rulemaking to revise the Part 72 seismic requirement for ISFSIs is unlikely to be completed before issuance of the TMI-2 ISFSI license, the staff intends to grant the exemption as requested if the Environmental Assessment (EA) is favorable." SECY-98-071 at 3. Two months later in June 1998, the Part 72 Rulemaking Plan (SECY-98-126) was released with allowance only for design basis ground motions with mean annual probabilities of exceedance corresponding to return periods of 1,000 years or 10,000 years, depending on risk. This sequence of events, in my opinion, does not support PFS's assertion that "there is no doubt that at the time the INEEL exemption was approved, the NRC Staff and the Commission expected (and intended) that it would serve as a precedent towards the granting of similar exemptions in the future." PFS's Motion for Summary Disposition at 14.

Q. 14: Please describe your concerns about Subsection E.6 of the Unified Contention Utah L/QQ.

A. 14: Subsection E.6 of the unified contention Utah L/QQ, states:

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[12] Id. at 4-2.

> Because (a) design levels for new Utah building construction and highway bridges are more stringent; and (b) the PFS return period is based on the twenty-year initial licensing period rather than the proposed thirty- to forty-year operating period, the 2000-year return period for the PFS facility does not ensure an adequate level of conservatism.

PFS's witness, Dr. Cornell, addresses the relative comparison of a DBE with a 2,000-year mean return period proposed for the PFS facility with the higher return period value of approximately 2,500 years required by the International Building Code 2000. Cornell Dec. ¶46. He states:

> One should not draw the erroneous conclusion, however, that this difference in the definition of the DBE implies a lower probability of failure for SSCs designed to IBC-2000 versus those, such as the PFSF, designed to the 2,000-year MRP and the NRC's SRP design procedures and criteria.

Id. Granting that "the safety achieved depends on *both* the DBE MRP and the design procedures and criteria utilized" (id.), the contested issue once again becomes the conservatism (or non-conservatism) in design of SSCs at the PFS facility. As in Answer No. 8 above, I defer the latter issue to the State's engineering and dynamic analyses experts (including implications for the analogous situation of comparing a 2,000-year MRP DBE for the PFS facility with a 2,500-year MRP DBE for new highway bridges in Utah). *See* Engineering and Dynamic Analyses Expert Testimony.

I might add that the Staff's comparison between probabilistic ground motions used for the design of new Interstate 15 highway bridges in the Salt Lake Valley and those proposed for use at the PFS site is partially erroneous and, in any case, irrelevant due to the many differences between the two sites. *See* SSER at 18.

Part (b) of Subsection 6 (the significance of a 20-year initial licensing period versus a 30- to 40-year total operational period) concerns a metric the Staff put forward for justifying the adequacy of a 2,000-year return period for seismic design of the PFS facility, namely, a 99-percent probability that the DBE not be exceeded in the 20-year licensing period of the facility. The Staff wrote:

> Considering the radiological safety aspects of a dry spent fuel storage facility, conservative peak ground motion values that have a 99 percent likelihood of not being exceeded in the 20-year licensing period of the facility are considered adequate for its seismic design. This exceedance probability corresponds to a return period of 2,000 years.

PSER at 2-45. The Staff again relies on this same metric in its recent Modified Rulemaking Plan as one basis to justify the proposed mean annual probability of 5×10^{-4} (return period of 2,000 years) for a DBE for dry-cask ISFSIs. Attachment to SECY-01-0178 at 7. Therein, the Staff argues:

> The total probability of exceedance for a design earthquake at an ISFSI facility with an operational period of 20 years (20 years x 5.0E-04 = 1.0E-02) is the same as the total probability of exceedance for an earthquake event at the proposed pre-closure facility at Yucca Mountain with an operational period of 100 years (100 years x 1.0E-04 = 1.0E-02).

Id. Using this metric, a facility with an operational life of 40 years would have to have a DBE with a mean return period of 3,980 years. State of Utah's Objections and Responses to Staff's First Set of Formal Discovery Requests to State of Utah (November 5, 2001), Answer to Interrogatory No. 1 at 8-10.

PFS's witness, Dr. Cornell, attacks Subsection 6(b) of Utah L (now E.6(b) of Unified Contention Utah L/QQ) stating:

> This contention is unfounded because in virtually all areas of public safety hazards are measured as annual probabilities (or frequencies) of occurrence, regardless of the length of the activity in question, the exposure time, the estimated facility life, or the licensing duration [Ref. 12 (Paté-Cornell paper)].

Cornell Dec. ¶49. In my deposition, I deferred to probability experts, including Dr. Cornell, when asked, "Do you have an opinion as to whether risks should be expressed on an annual basis or the total life of a facility?" Arabasz Dep. at 51-52. However, I beg to differ with Dr. Cornell's statement above and will elaborate.

Q. 15: Please elaborate on how considerations of seismic hazards in areas of public safety commonly take into account the exposure time and not just the annual probabilities (or frequencies) of occurrence.

A. 15: One of the well-established standards for portraying ground-shaking hazard in the United States is the suite of national seismic hazard maps published by the U.S. Geological Survey. "The hazard maps depict probabilistic ground motions and spectral response with 10%, 5%, and 2% probabilities of exceedance (PE) in 50 years." *National Seismic-Hazard Maps: Documentation June 1996, USGS Open-File Report 96-532* at 1. These maps provide reference ground motions for the International Building Code 2000. Dr. Cornell and I were co-members of a Review Panel for the USGS national maps in 1996.

Another well-established standard linked to building codes is the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 1997 Edition (FEMA 303)* ("Provisions"). The Commentary to the Provisions states:

> In past editions of the *Provisions*, seismic hazards around the nation were defined at a uniform 10 percent probability of exceedance in 50 years . . . . While this approach provided for a uniform likelihood throughout the nation that the design ground motion would not be exceeded, it did not provide for a uniform margin of failure for structures designed to that ground motion. . . . The approach adopted in these *Provisions* is intended to provide for a uniform margin against collapse at the design ground motion. . . . For most regions of the nation, the maximum considered earthquake ground motion is defined with a uniform likelihood of exceedance of 2 percent in 50 years (return period of about 2500 years.)

Provisions, Part 2— Commentary at 37.

The National Research Council's Panel on Seismic Hazard Analysis noted the following:

> [A]TC-3 (Applied Technology Council, 1978) has suggested the design seismic hazard level should have a 10 percent probability of exceedance in 50 years, which corresponds to an annual exceedance probability of about 2×10^{-3}. . . . The proposed Department of Defense tri-services seismic design provisions (Joint Departments of Army and Air Force, USA, 1985) suggests [sic] for category II facilities a dual level for the design seismic hazard. Such facilities should remain essentially elastic for seismic hazard with about a 50 percent probability of exceedance in 50 years or about a 1×10^{-2} annual exceedance probability and should not fail for a seismic hazard that has about a 10 percent probability of exceedance in 100 years . . ."

Panel on Seismic Hazard Analysis, *Probabilistic Seismic Hazard Analysis*, National Academy Press, Washington, D.C. (1988) at 31-32.

Procedures for estimating the probability of exceeding some level of ground motion during an exposure period of interest are commonly given for design guidance. For example, DOE-STD-1020-94 includes such a procedure at A-1, and Leon Reiter in his text, *Earthquake Hazard Analysis*, similarly includes such a procedure, including a graph from NUREG/CR-1582, 2 (1980), for relating return period, period of interest and desired

probabilities of exceedance during the period of interest. L. Reiter, *Earthquake Hazard Analysis*, Columbia University Press (1990) at 185.

The cited paper by Paté-Cornell does not convincingly establish as a norm for public safety that "hazards are measured as annual probabilities (or frequencies) of occurrence, regardless of the length of the activity in question, the exposure time, the estimated facility life, or the licensing duration." Cornell Dec. ¶49. First, in the context of noting that "current PRA [probabilistic risk analysis] methodology tends to focus on the technical causes of system failure" (while ignoring human and organizational factors), Paté-Cornell writes: "Classical technical PRA's tend to focus on the probability that an extreme value of the loads to which a system may be exposed (during a given year <u>or lifetime</u>) exceeds its capacity." Paté-Cornell paper at 148, footnote 4, underlining added. Second, while hardly a commentary on "virtually all areas of public safety," the paper reviews five precedents as examples of safety targets: (a) nuclear power plants in the U.S., (b) cancer risks in the U.S., (c) offshore oil and gas industry in Norway, (d) fatality accident rate in the U.K., and (e) the Dutch government standards. Significantly, cases (b) and (d) involve risk measured per individual or worker <u>lifetime</u>. In case (c) the Norwegian Petroleum Directorate temporarily adopted a severe-accident criterion in terms of an annual probability of major initiators of platform failure but "recently backed away from their severe-accident criterion . . . because this criterion was leading to a 'numbers game' that seemed to be distracting both the industry and the regulators from fundamental safety issues. . ." <u>Id</u>. at 150. Third, after discussing issues that have emerged in recent years in safety debate, Paté-Cornell <u>proposes</u> an approach to a global safety strategy, of which one element (of six) is that "it should be ensured that the *annual probability of catastrophic failure* (the severe accident criterion) is less than a specified threshold, e.g., 10^{-4} per year." <u>Id</u>. At 151. Fourth, the cited paper includes discussion of "time horizon" as a relevant risk factor, albeit in the context of shorter lifetime of aging facilities versus new ones.

Dr. Cornell attempts to bolster his argument by noting that "risk acceptance guidelines promulgated by the NRC" (for nuclear power plants) are in terms of annual risk for Core Damage Frequency and Large Early Release Frequency. Nevertheless, within a context of evolving regulatory guidance for ISFSIs, the Staff itself uses the metric of total probability of exceedance during a 20-year operational period to justify a DBE with a 2,000-year mean return period for dry-cask ISFSIs. Attachment to SECY-01-0178 at 7, included as State's Exhibit 128.

Finally, Dr. Cornell explains the reasons for focusing on annual risks in making safety decisions, in part, because "any facility providing a needed service will, at the end of its operating life, most likely be replaced by some other facility used for the same purposes with its own, similar risks." Cornell Dec. ¶49. While consideration of risk involving where spent fuel is now stored or may eventually be stored in the future at Yucca Mountain may be relevant for a societal global safety strategy (such as described in the Paté-Cornell paper), the issue at hand is a risk-acceptance decision specific to the PFS site.

Q. 16: Do you have anything further to add?

A. 16: In my testimony I have attempted to systematically address each of the subsections, within my scope of expertise and testimony, associated with unified contention Utah L/QQ, Section E. In my opinion, the key contested issue is the validity of PFS's claim that it has met the Commission's requirement to show that "the 2000-year design standard is sufficiently protective of public safety and property" as called for by the Commission in CLI-01-12. PFS's claim fundamentally rests on the proposition that sufficient protection "depends on both the probability of occurrence of the seismic event (often expressed as the mean annual probability of exceedence or "MAPE" of a given earthquake level) and the level of conservatism incorporated in the design procedures and criteria."[13] I agree with the proposition – but the latter critical part of PFS's claim of sufficient protection is challenged by the State's engineering and dynamic analyses experts, who dispute PFS assertions that it has demonstrated adequate conservatism in design of SSCs at the PFS facility. I defer to these experts for more complete discussion of their disputes, which go the heart of "appropriately conservative" and "sufficiently protective" design of the PFS facility. *See* Engineering and Dynamic Analyses Expert Testimony.

Q. 17: Does this conclude your testimony?

A. 17: Yes.

[13] PFS's Motion for Summary Disposition at 6.

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Current Professional Activities: Utah Seismic Safety Commission—Member (since 1994), past Chair (1994-2001), and Member, Geoscience Standing Committee (since 1994); Regional Coordinator, Advanced National Seismic System, Intermountain West Region (since 2000); Member, Infrastructure Protection Subcommittee, Utah Olympic Public Safety Command (since 1998).

General Statement of Experience: Dr. Walter J. Arabasz has more than 30 years of professional experience in research, project management, consulting, and occasional teaching in observational seismology, tectonics, and earthquake hazard evaluation. He is the author or co-author of 37 published papers, 77 published abstracts and numerous technical reports. His present responsibilities at the University of Utah include seismological research and extensive project management—chiefly relating to the operation and modernization of a 160-station regional/urban seismic network covering Utah and neighboring parts of the Intermountain area.

He has been affiliated, since its inception, with the U.S. National Earthquake Hazards Reduction Program—variously as a Principal Investigator of funded research, as a participant in scores of workshops and conferences, and as a member of peer review panels. He has served on numerous national and state advisory and policy-making committees, including the Committee on Seismology of the National Research Council (1989-1994), the Board of Directors of the Seismological Society of America (1994-1997), the Council of the National Seismic System (1993-1999), a Senior Advisory Group to the U.S. Geological Survey for a 1999 report to Congress on an Assessment of Seismic Monitoring in the United States (1998-1999), and the Utah Seismic Safety Commission (1994-present). Since 1977 he has routinely provided professional consulting services on earthquake hazard evaluations for dams, nuclear facilities, and other critical structures and facilities for engineering firms, the International Atomic Energy Agency, the Department of Energy, the Soil Conservation Service, the Bureau of Reclamation, the Electric Power Research Institute, and the State of Utah.

PUBLICATIONS AND SELECTED REPORTS SINCE 1990

Arabasz, W. J., editor, 1990, A guide to reducing losses from future earthquakes in Utah—"Consensus document": Utah Geological and Mineral Survey Miscellaneous Publication 91-1, 30 pp.

Arabasz, W. J., editor, 1990, Earthquake instrumentation for Utah, Report and Recommendations of the Utah Policy Panel on Earthquake Instrumentation, Utah Geological and Mineral Survey Open-File Report 168, 167 pp.

Arabasz, W. J., and Atwood, G., 1990, Seismic instrumentation—A five-element, $3M state initiative for Utah's earthquake program [abs.]: Seismological Research Letters, v. 61, p. 37.

Johnston, A. C., Arabasz, W. J., Bollinger, G. A., Filson, J. R., Herrmann, R. B., Jones, L., and Kanamori, H., 1990, Assessing the Nation's earthquakes—The health and future of regional seismograph networks: Washington, D. C., National Academy Press, 67 pp.

Pechmann, J. C., Nava, S. J., and Arabasz, W. J., 1990, Left-lateral shear beneath the NW Colorado Plateau: The 1988 San Rafael Swell and 1989 So. Wasatch Plateau earthquakes [abs.]: Seismological Research Letters, v. 61, p. 44.

McGuire, R. K., and Arabasz, W. J. 1990, An introduction to probabilistic seismic hazard analysis, *in* Ward, S. H., ed., Geotechnical and environmental geophysics, Vol. 1: Review and tutorial: Society of Exploration Geophysicists, Investigations in Geophysics No. 5, pp. 333-353.

Arabasz, W. J., Pechmann, J. C., Nava, S. J., and Brown, E. D., 1991, Information from observational seismology relevant to earthquake engineering in the Wasatch Front area, Utah [abs.]: Earthquake Engineering Research Institute, Abstracts With Program 1991 Annual Meeting, Salt Lake City.

Pechmann, J. C., Nava, S. J., and Arabasz, W. J., 1991, Seismological analyses of four recent moderate (M_L 4.8 to 5.4) earthquakes in Utah, Final Report to the Utah Geological Survey, Salt Lake City, Utah, Contract No. 89-3659, 107 pp.

Smith, R. B., and Arabasz, W. J., 1991, Seismicity of the Intermountain seismic belt, *in* Slemmons, D. B., Engdahl, E. R., Zoback, M. D., Zoback, M. L., and Blackwell, D., eds., Neotectonics of North America: Geological Society of America, Decade Map Volume 1, pp. 185-228.

Arabasz, W. J., 1992, Earthquake basics for dam safety [extended abs.]: Abstracts and Proceedings, Utah Department of Natural Resources, The State of Utah Dam Safety Workshop, March 19, 1992, Salt Lake City, 9 pp.

Arabasz, W. J., 1992, New agenda for Committee on Seismology: Eos (Transactions American Geophysical Union), v. 73, no. 50, p. 541.

Arabasz, W. J., Pechmann, J. C., and Brown, E. D., 1992, Observational seismology and the evaluation of earthquake hazards and risk in the Wasatch front area, Utah, *in* Gori, P. L., and Hays, W. W., eds., Assessment of regional earthquake hazards and risk along the

Wasatch Front, Utah: U. S. Geological Survey Professional Paper 1500-A-J, pp. D1-D36.

Pechmann, J.C., Arabasz, W. J., and Nava, S. J., 1992, The St. George, Utah, earthquake of September 2, 1992: A normal-faulting earthquake with very weak aftershock activity [abs.]: Eos (Transactions American Geophysical Union), v. 73, no. 43, p. 399.

Pechmann, J.C., Nava, S. J., and Arabasz, W. J., 1992, Four moderate (M_L 4.8 to 5.4) earthquakes in Utah, 1987 to 1989 [abs.]: Seismological Research Letters, v. 63, p. 38.

Reasenberg, P. A., and others, including W. J. Arabasz, 1992, Remote seismicity triggered by the M 7.5 Landers, California, earthquake of June 28, 1992 [abs.]: Eos (Transactions American Geophysical Union), v. 73, v. 43 (Supplement), p. 392.

Arabasz, W. J., 1993, Earthquakes in Utah—Quantifying and dealing with the threat [abs.]: Proceedings, Utah Water Users Association, 1993 Annual Workshop, March 9-10, 1993, St. George, Utah.

Hill, D. P., and others, including W. J. Arabasz, 1993, Remote seismicity triggered by the M 7.5 Landers, California, earthquake of June 28, 1992: Science, v. 260, pp. 1617-1623.

Pechmann, J. C., Arabasz, W. J., and Brown, E. D., 1993, Seismotectonics of the 1987-88 Lakeside, Utah, earthquakes: Seismological Research Letters, v. 64, pp. 225-238.

Arabasz, W. J., 1994, Fundamentals of the Wasatch Front's earthquake threat: Proceedings, Seminar 1: Economic Impacts of a Large Earthquake, Earthquake Engineering Research Institute Wasatch Front Seismic Risk Regional Seminar; Salt Lake City, Utah, November 1994, pp. 1-1 to 1-25.

Arabasz, W. J., 1994, Wasatch Front seismicity and expectable strong ground motion: Proceedings, Seminar 2: Earthquake Research and Mitigation, Earthquake Engineering Research Institute Wasatch Front Seismic Risk Regional Seminar; Salt Lake City, Utah, November 1994, pp. 2-1 to 2-40.

Arabasz, W. J., and Hill, S. J., 1994, Aftershock temporal behavior and earthquake clustering in the Utah region [abs.]: Seismological Research Letters, v. 65, p. 32.

Nava, S. J., Arabasz, W. J., and Pechmann, J. C., 1994, The M 5.9 Draney Peak, Idaho (Idaho-Wyoming border) earthquake of February 3, 1994—A preliminary report [abs.]: Seismological Research Letters, v. 65, nos. 3/4, p. 239.

Nava, S. J., Arabasz, W. J., Pechmann, J. C., and Smith, R. B., 1994, Status and developments in data distribution and exchange—University of Utah Regional Seismic Network [abs.]: Eos (Transactions American Geophysical Union), v. 75, no. 44, p. 430.

Pechmann, J. C., Arabasz, W. J., and Nava, S. J., 1994, Refined analysis of the 1992 M_L 5.8 St. George, Utah, earthquake and its aftershocks [abs]: Seismological Research Letters, v. 65, p. 32.

Arabasz, W., and Malone, S., 1995, Council of the National Seismic System, background and update: Seismological Research Letters, v. 66, no. 4, pp. 29-30.

Arabasz, W. J, and Nava, S. J., 1995, Jordanelle Reservoir: Status of test monitoring for reservoir-induced seismicity [abs.]: Seismological Research Letters, v. 66, no. 2, p. 24.

O'Brien, E. H., Nava, S. J., and Arabasz, W. J., 1995, Earthquake education services for Utah schools [abs.]: Seismological Research Letters, v. 66, no. 2, p. 41.

Pechmann, J. C., and Arabasz, W. J., 1995, The problem of the random earthquake in seismic hazard analysis: Wasatch Front region, Utah, *in* Lund, W. R., ed., Environmental and engineering geology of the Wasatch Front region: Utah Geological Association Publication 24, pp. 77-93.

Pechmann, J. C., Walter, W. R., Nava, S. J., and Arabasz, W. J., 1995, The February 3, 1995, M_L 5.1 seismic event in the trona mining district of southwestern Wyoming: Seismological Research Letters, v. 66, no. 3, pp. 25-34 [minor correction, including revision of magnitude to M_L 5.2, added in v. 66, no. 4, p. 28].

Arabasz, W. J., and Hill, S. J., 1996, Applying Reasenberg's cluster-analysis algorithm to regional earthquake catalogs outside California [abs.]: Seismological Research Letters, v. 67, no. 2, p. 30.

Arabasz, W. J., and Wyss, M., 1996, Significant precursory seismic quiescence in the extensional Wasatch Front region, Utah [abs.]: Eos (Transactions American Geophysical Union), v. 77, no. 46, p. F455.

Wyss, W., Wiemer, S., and Arabasz, W. J., 1996, Seismic quiescence occurs as a precursor after all [abs.]: Eos (Transactions American Geophysical Union), v. 77, no. 22 (Supplement), p. W92.

Arabasz, W. J., Nava, S. J., and Phelps, W. T., 1997, Mining seismicity in the Wasatch Plateau and Book Cliffs coal mining districts, Utah, USA, *in* Gibowicz, S. J., and Lasocki, S., eds., Rockbursts and Seismicity in mines: Rotterdam, A.A. Balkema, pp. 111-116.

Pechmann, J. C., Brumbaugh, D. A., Nava, S. J., Skelton T. G., Fivas, G. P., and Arabasz, W. J., 1997, The 1994 Draney Peak, ID, earthquake and its aftershocks [abs.]: Eos (Transactions American Geophysical Union), v. 78, no. 46 (Supplement), p. F480.

Arabasz, W. J., 1998, Do regional seismic networks in the U.S. have a future?: Seismological Research Letters, v. 69 no. 6, pp. 513-515.

Arabasz. W. J., 1998, Utah's earthquake information needs—New technologies and solutions: Fault Line Forum (published by Utah Geological Survey), v. 14, no. 12, pp. 1-4.

Arabasz, W. J., and Pechmann, J. C., 1998, Earthquake data-base issues for seismic hazard analysis in the Utah region [abs.]: Proceedings Volume, Basin and Range Province Seismic-Hazards Summit, Utah Geological Survey Misc. Publication 98-2, p. 31.

Arabasz, W. J., and Wyss, M., 1999, Spatially-varying b-values in coal-mining seismicity in Utah: Inferred influence of heterogeneity and stress differences on local mean magnitude [abs.]: Eos (Transactions American Geophysical Union), v. 80, no. 46, p. F651.

Benz, H., Filson, J., Arabasz, W., and Gee, L.,1999, An assessment of seismic monitoring in the United States: Requirement for an Advanced National Seismic System: U.S. Geological Survey Circular 1188, 55 pp.

Pechmann, J. C., Nava, S. J., and Arabasz, W. J., 1999, Installation and calibration of five new broadband digital telemetry stations in Utah [abs.]: Seismological Research Letters, v. 70, no. 2, p. 244.

Arabasz, W. J., and McCarter, M. K., 2000. Mine seismicity and the interface between mining engineers and seismologists [abs.]: Seismological Research Letters, v. 71, no. 2, p. 220.

Arabasz, W. J., and Nava, S. J., 2000, New urban earthquake monitoring in Utah: Fault Line Forum (published by Utah Division of Comprehensive Emergency Management), v. 16, no. 1, p. 1,3.

Arabasz, W. J., Nava, S. J., Cluff, D. C., Drobeck, D. L., Moeinvaziri, A. , and Pechmann, J. C., 2000, New real-time urban strong-motion network in Utah (Phase I) [abs.]: Eos (Transactions Ameican Geophysical Union), v. 81, no. 48, Fall Meet. Suppl., S71A-08, p. F864.

Pechmann, J. C., Nava, S. J., Bernier, J. C., and Arabasz, W. J., 2000, A critical analysis of systematic time-dependent coda-magnitude errors in the University of Utah earthquake catalog, 1981-1999 [abs.]: Eos (Transactions American Geophysical Union), v. 81, no. 48, Fall Meet. Suppl., S71C-10, p. F869.

Arabasz, W. J., and Pechmann, J. C., 2001, Seismic characterization of coal-mining seismicity in Utah for CTBT monitoring, Technical Report UCRL-CR-143772, prepared for Lawrence Livermore National Laboratory, Livermore, California, LLNL Research Agreement No. B344836, 120 pp.

Arabasz, W. J., Pechmann, J. C., and McCarter, M. K., 2001, Mechanisms and ground-truth observations for the largest (M_L = 3.0-4.2) coal-mining-induced seismic events in Utah, 1978-2000 [abs.]: Seismological Research Letters, v. 72, no. 2, p. 286.

Pankow, K. L., Pechmann, J. C., Nava, S. J., and Arabasz, W. J., 2001, Implementing ShakeMap in Utah before the 2000 Winter Olympics [abs.]: Seismological Research Letters, v. 72, no. 2, p. 232.

Pechmann, J. C., Bernier, J. C., Nava, S. J., Terra, F. M., and Arabasz, W. J., 2001, Correction of systematic time-dependent coda-magnitude errors in the Utah and Yellowstone National Park Region earthquake catalogs, 1981-2001 [abs.]: Eos (Transactions American Geophyical Union), v. 81, no. 49, Fall Meet. Suppl. (in press).

Youngs, R. R., and others, including W. J. Arabasz, 2001, Probabilistic fault displacement hazard analysis (PFDHA): Earthquake Spectra (submitted).

PROFESSIONAL CONSULTING

1. Rollins, Brown and Gunnell, Inc., Provo, Utah (1977). Site-response analysis for proposed hospital in Logan, Utah.

2. International Atomic Energy Agency, Vienna, Austria (Short-Term Expert, Chile, 1979). Seismotectonic considerations in northern Chile, 21°-27° S, with respect to the siting of a nuclear power plant.

3. Weidlinger Associates, Menlo Park, California (1980). Geological and geophysical information relevant to site-dependent ground motions at Wing V (Wyoming-Nebraska-Colorado).

4. EG&G Idaho, Inc. (Department of Energy), Idaho Falls, Idaho (1980-81). Preparation and presentation of proposal for seismic risk zone revision in southeast Idaho to International Conference of Building Officials.

5. EG&G Idaho Inc. (Department of Energy), Idaho Falls, Idaho (1982). Document review: *Site Investigation at Idaho National Engineering Laboratory, LMFBR Large Developmental Plant (LDP), Conceptual Design Study-Phase III.*

6. Lindvall, Richter and Associates, Los Angeles, California (1981-82). Seismic safety investigation of eight Soil Conservation Service dams in southwestern Utah.

7. U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado (1982-83). Review and analysis of geologic, seismotectonic, and design data for the proposed Jordanelle Dam, Bonneville Unit, Central Utah Project, Utah. (Consultant review by W. J. Arabasz, R. H. Jahns, and R. B. Peck.)

8. EG&G Idaho, Inc. (Department of Energy), Idaho Falls, Idaho (1983). Member, Geotechnical Advisory Panel to assist EG&G Idaho, Inc. and DOE regarding programmatic efforts toward site characterization of the Idaho National Engineering Laboratory for the proposed siting of a New Production Reactor Facility.

9. Electric Power Research Institute, Palo Alto, California (1984). Participant, "Data Needs Workshop; regarding data management plan and tectonic evaluation for earthquake hazards in the eastern U.S.; participant and editor of *Proceedings of a Seminar on Defining Tectonic Mechanisms Causing Earthquakes in the Eastern United States*.

10. Dames & Moore, Golden, Colorado/Electric Power Research Institute (EPRI), Palo Alto, California (1984-85). Member of "Seismic Hazard Methodology Team," EPRI Seismic Hazards Research Program, for evaluation of earthquake hazards in the eastern United States for the siting of nuclear generating facilities. (Participation in seven formal workshops, two academic seminars, and three series of interactive meetings with six teams of tectonic evaluation contractors in the central and eastern U.S.).

11. Electric Power Research Institute, Palo Alto, California (1985-87). Participation in scientific review, technical description, and comparative evaluation of EPRI seismic hazard methodology for the central and eastern United States.

12. U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado (1986-87). Review and analysis of geologic, seismotectonic, and design data for the proposed Jordanelle Dam, Bonneville Unit, Central Utah Project. (Consultant review by R. B. Peck, W. J. Arabasz, G. S. Tarbox, and D. D. Campbell.)

13. U.S. Bureau of Reclamation, Engineering and Research Center,Denver, Colorado (1988). Review and evaluation of seismotectonic conclusions and details of final embankment dam design for Jordanelle Dam, Bonneville Unit, Central Utah Project. (Consultant review by R. B. Peck, W. J. Arabasz, and T. G. McCusker.)

14. Dames & Moore, Los Angeles, California (1989). Member of advisory panel for project on seismic code decisions under risk, sponsored by the National Science Foundation.

15. Lawrence Livermore National Laboratory, Livermore, California (1990-91). Member of Seismicity and Tectonic Expert Group, New Production Reactors Project, Idaho National Engineering Laboratory Site.

16. U.S. Bureau of Reclamation-Engineering and Research Center, Denver, Colorado, and Regional Office, Salt Lake City, Utah (1990-92). Review and evaluation of foundation conditions, ongoing geologic mapping procedures, and seismic-safety aspects of the Jordanelle Dam, Bonneville Unit, Central Utah Project. (Consultant review by W. J. Arabasz, R. B. Peck, and D. D. Campbell.)

17. Science Applications International Corporation, Las Vegas, Nevada (1991). Member of Peer Review Group for Early Site Suitability Evaluation of the Potential Repository Site at Yucca Mountain, Nevada.

18. Geomatrix Consultants, San Francisco, California (1991-92). Member of expert panel, Electric Power Research Institute High Level Waste (EPRI-HLW) project to assess earthquake and tectonic issues for the proposed high-level nuclear waste repository at Yucca Mountain, Nevada.

19. Risk Engineering, Inc., Golden, Colorado (1992-94). Investigator for Seismology as part of a Seismic Hazard Study for Systematic Evaluation Program, Rocky Flats Plant, conducted for EG&G Rocky Flats, Inc. and sponsored by the U.S. Department of Energy.

20. Woodward-Clyde Federal Consultants, Las Vegas, Nevada (1993-94). Technical reviewer for (1) *Topical Report: Methodology to Assess Seismic Hazards at Yucca Mountain* and (2) *Seismic Design Inputs for the Exploratory Studies Facility at Yucca Mountain.*

21. U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado (1994). Review of design, construction, and operation of Jordanelle Dam and Reservoir, Bonneville Unit, Central Utah Project, Utah. (Consultant review by W. J. Arabasz, D. D. Campbell, and R. B. Peck.)

22. Jack R. Benjamin & Associates, Inc., Mountain View, California (1994). Technical reviewer for *Probabilistic Seismic Hazard Assessment for the U.S. Army Chemical Demilitarization Facility, Tooele, Utah.*

23. TRW Environmental Safety Systems, Inc., Vienna, Virginia (1995). Member of expert team for seismic source characterization for a probabilistic seismic hazard assessment of a high-level nuclear waste repository at Yucca Mountain, Nevada.

24. Rutherford & Chekene, San Francisco, California (1995). Technical review and consulting advice on seismicity and ground-motion considerations for design of a manufacturing plant at Lehi, Utah, for Micron Technology, Inc.

25. TRW Environmental Safety Systems, Inc., Vienna, Virginia (1995). Organizer and chair of plenary session of FOCUS 95—Methods of Seismic Hazard Evaluation (a topical meeting co-sponsored by the American Nuclear Society and the Geological Society of America, September 18-20, 1995, Las Vegas, Nevada).

26. William Lettis & Associates, Inc., Walnut Creek, California (1995). Technical review and consulting advice on seismic source characterization for the stability evaluation of Lake Almanor and ButtValley Dams, California.

27. Parsons Brinckerhoff, Salt Lake City, Utah (1995-96). Member of Seismic Advisory Committee to Utah Department of Transportation for seismic hazard analysis of the I-15 interstate highway corridor (consulting undertaken under a University of Utah contract).

28. TRW Environmental Safety Systems, Inc., Vienna, Virginia (1996-98). Member of expert team for seismic source characterization for a probabilistic seismic hazard assessment of a high-level nuclear waste repository at Yucca Mountain, Nevada.

29. Utah Department of Environmental Quality (1998-2002). Seismicity and earthquake expert for evaluation of a proposed high-level radioactive waste storage facility in Skull Valley, Tooele County, Utah (consulting undertaken under a University of Utah contract).

30. Los Alamos National Laboratory, Los Alamos, New Mexico (2000-2001). Member of Laboratory Seismic Review Committee, Nuclear Materials and Stockpile Management Program, an advisory group on the Laboratory's seismic risks and hazards and related technical and operational activities.

31. U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado (2001). Review of seismic hazard and design of Deer Creek Dam and Reservoir, Utah. (Consultant review by W. J. Arabasz, J. Mitchell, and R. B. Peck).

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

OFFICE OF THE
SECRETARY

November 19, 2001

MEMORANDUM TO: William D. Travers
Executive Director for Operations

FROM: Annette L. Vietti-Cook, Secretary

SUBJECT: STAFF REQUIREMENTS - SECY-01-0178 - MODIFIED RULEMAKING PLAN: 10 CFR PART 72 -- "GEOLOGICAL AND SEISMOLOGICAL CHARACTERISTICS FOR SITING AND DESIGN OF DRY CASK INDEPENDENT SPENT FUEL STORAGE INSTALLATIONS"

This is to advise you that the Commission has not objected to the staff's plan to revise the approved rulemaking plan for the geological and seismological characteristics for the siting and design of dry cask independent spent fuel storage installations (10 CFR Part 72), subject to the comments provided below.

Central to this rulemaking is the determination of the mean annual exceedance probability of an earthquake at a proposed ISFSI. The proposed rule should solicit comment on a range of probability of exceedance levels from 5.0E-04 through 1.0E-04. Staff should undertake further analysis to support a specific proposal.

The proposed rule should be submitted to the Commission for review prior to publication.
(EDO) (SECY Suspense: 3/22/02)

cc: Chairman Meserve
Commissioner Dicus
Commissioner Diaz
Commissioner McGaffigan
Commissioner Merrifield
OGC
CFO
OCA
OIG
OPA
Office Directors, Regions, ACRS, ACNW, ASLBP (via E-Mail)
PDR

State's
Exhibit 124

14th International Conference on
Structural Mechanics in Reactor Technology
Lyon, France, August 17-22, 1997

PAPER NO.717

ATTORNEY GENERAL
OCT 30 2000
ENVIRONMENT

Title: Revision of Seismic and Geologic Siting Criteria

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Exhibit 125

ABSTRACT

In September of 1990, the U.S. Nuclear Regulatory Commission (NRC) began the formal process to revise the seismic and geologic siting criteria for nuclear power plants. This process is now complete and the revised criteria have been promulgated. During the course of the revision, two draft versions were issued for public comments and extensive interactions took place between the NRC and the industry. This paper describes the new siting criteria and associated regulatory guidance.

1.0 INTRODUCTION

The U.S. seismic siting regulation, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," Appendix A to 10 CFR Part 100, became effective in December, 1973 (Ref. 1). Although it has been a relatively successful licensing tool for over two decades, significant difficulties have been encountered in applying it. For example, there have been substantial advances in the geosciences, and because of the inherent inflexibility of a regulation, it has been difficult or impossible to accommodate these changes or to modify the criteria. Furthermore, Appendix A is based on deterministic seismic hazard concepts, and the large uncertainties intrinsic to geosciences such as seismic sources and ground motions, are not quantitatively taken into account.

Attempts to revise the regulation were started as early as 1979. Ref.2 document the major issues involved and provided strong justification to revise the regulation. However, it was not until 1990 that official revision of Appendix A began.

During the period 1990 through 1995 the proposed regulations and draft guidance documents were prepared, and the new methodologies they invoked were developed and tested. From the onset, it was decided that Appendix A to 10 CFR Part 100 would b

retained and continue to apply to operating plants that received their licenses prior to publication of the new regulation.

In developing the new regulation, it was decided to separate siting from design. Therefore, the engineering portions of Appendix A were transferred to 10 CFR Part 50 in a new Appendix S (Ref.3).

The geosciences portion of Appendix A was condensed to general requirements only. The prescriptive elements were placed in Regulatory Guide 1.165 (Ref.4). The new regulations, Section 100.23 to Part 100 (Ref.5), Geologic and Seismic Siting Factors, and Appendix S to Part 50 were published in December 1996 with the effective date of January 10, 1997. The regulatory guides were published in March 1997. The new regulations and guides are applicable to future plants.

2.0 REVISED SEISMIC AND GEOLOGIC SITING AND EARTHQUAKE ENGINEERING CRITERIA

2.1 Siting Criteria

The revised seismic and geologic siting regulation, 10 CFR 100.23, requires that geological, seismological, and engineering characteristics of a site and its environs be investigated in sufficient scope and detail to permit an evaluation of the proposed site, to provide sufficient information to determine the Safe Shutdown Earthquake Ground Motion (SSE), to permit engineering solutions to geologic and seismic effects, to assess the potential for surface deformations, and to establish the design bases for seismically induced floods and water waves, and other design conditions at the proposed site.

Selected portions of the rule are reproduced below. With respect to the intent of the regulation, the rule states the following:

§ 100.23 Geologic and seismic siting factors.

This section sets forth the principal geologic and seismic considerations that guide the Commission in its evaluation of the suitability of a proposed site and adequacy of the design bases established in consideration of the geologic and seismic characteristics of the proposed site, such that, there is a reasonable assurance that a nuclear power plant can be constructed and operated at the proposed site without undue risk to the health and safety of the public.

With respect to the specific requirements, the rule contains the following elements:

*(c) **Geological, seismological, and engineering characteristics**. The geological, seismological, and engineering characteristics of a site and its environs must be investigated in sufficient scope and detail to permit an adequate evaluation of the proposed site, to provide sufficient information to support evaluations performed to arrive at estimates of the Safe Shutdown Earthquake Ground Motion, and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site. The size of the region to be investigated and the type of data pertinent to the investigations must be determined based on the nature of the region surrounding the proposed site. Data on the vibratory ground motion, tectonic surface deformation, nontectonic deformation, earthquake recurrence rates, fault geometry and slip rates, site foundation material, and seismically induced floods and water waves must be obtained by reviewing pertinent literature and carrying out field investigations. However, each applicant shall investigate all geologic and seismic factors (for example, volcanic activity) that may affect the design and operation of the proposed*

nuclear power plant irrespective of whether such factors are explicitly included in this section.

(d) Geologic and seismic siting factors. *The geologic and seismic siting factors considered for design must include a determination of the Safe Shutdown Earthquake Ground Motion for the site, the potential for surface tectonic and nontectonic deformations, the design bases for seismically induced floods and water waves, and other design conditions as stated in paragraph (d)(4) of this section.*

(1) Determination of the Safe Shutdown Earthquake Ground Motion. ***The Safe Shutdown Earthquake Ground Motion for the site is characterized by both horizontal and vertical free-field ground motion response spectra at the free ground surface. The Safe Shutdown Earthquake Ground Motion for the site is determined considering the results of the investigations required by paragraph (c) of this section. Uncertainties are inherent in such estimates. These uncertainties must be addressed through an appropriate analysis, such as a probabilistic seismic hazard analysis or suitable sensitivity analyses. [Underline added for emphasis]. Paragraph IV(a)(1) of Appendix S to Part 50 of this chapter defines the minimum Safe Shutdown Earthquake Ground Motion for design.***

(2) Determination of the potential for surface tectonic and nontectonic deformations. Sufficient geological, seismological, and geophysical data must be provided to clearly establish whether there is a potential for surface deformation.

(3) Determination of design bases for seismically induced floods and water waves. The size of seismically induced floods and water waves that could affect a site from either locally or distantly generated seismic activity must be determined.

(4) Determination of siting factors for other design conditions. Siting factors for other design conditions that must be evaluated include soil and rock stability, liquefaction potential, natural and artificial slope stability, cooling water supply, and remote safety-related structure siting. Each applicant shall evaluate all siting factors and potential causes of failure, such as, the physical properties of the materials underlying the site, ground disruption, and the effects of vibratory ground motion that may affect the design and operation of the proposed nuclear power plant.

A comparison of 10CFR 100.23 with Appendix A to Part 100 will clearly show that the new rule contains only the basic requirements and all of the prescriptive procedures are now removed. The requirements for adequate site characterization are basically the same as those in Appendix A, therefore, the geological and seismological investigations are equally important. One of the most significant changes is underlined above. The rule now recognizes that there are uncertainties in estimating the design basis ground motion and they should be specifically addressed in that determination. A probabilistic hazard analysis is permitted to address these uncertainties.

2.2 Earthquake Engineering Criteria

Another important change is with respect to the Operating Basis Earthquake (OBE) in the new Appendix S to Part 50. The existing regulation in Appendix A to Part 100 states that the maximum vibratory ground motion of the OBE be at least one half the maximum vibratory ground motion of the Safe Shutdown Earthquake ground motion. In some cases, for instance piping, this requirement for the OBE made it possible for the OBE to have more design significance than the SSE. A decoupling of the OBE and SSE has been suggested in several documents (e.g., Ref.2). Appendix S allows the value of the OBE to be set at (i) one-third or less of the SSE, where OBE requirements are satisfied without an explicit response or design analyses being performed, or (ii) a value greater than one-third of the SSE.

where analysis and design are required. There are two issues the applicant should consider in selecting the value of the OBE: first, plant shutdown is required if vibratory ground motion exceeding that of the OBE occurs, and second, the amount of analyses associated with the OBE. An applicant may determine that at one-third of the SSE level, the probability of exceeding the OBE vibratory ground motion is too high, and the cost associated with plant shutdown for inspections and testing of equipment and structures prior to restarting the plant is unacceptable. Therefore, the applicant may voluntarily select an OBE value at some higher fraction of the SSE to avoid plant shutdowns. However, if an applicant selects an OBE value at a fraction of the SSE higher than one-third, a suitable analysis shall be performed to demonstrate that the requirements associated with the OBE are satisfied.

As stated, it is determined that if an OBE of one-third or less of the SSE is used, the requirements of the OBE can be satisfied without the applicant performing any explicit response analyses. In this case, the OBE serves the function of an inspection and shutdown earthquake. Three regulatory guides, RG 1.12, RG 1.166, and RG 1.167 were prepared to describe the methodologies acceptable to the staff that should be employed to satisfy the Appendix S requirements pertaining to plant shutdown and restart due to a seismic event.

3.0 GUIDANCE ON DETERMINING SAFE SHUTDOWN EARTHQUAKE GROUND MOTION

Regulatory Guide 1.165 has been developed to provide general guidance on procedures acceptable to the NRC staff for conducting geological, seismological, and geophysical investigations; identifying and characterizing seismic sources; conducting probabilistic seismic hazard analyses; and determining the Safe Shutdown Earthquake Ground Motion (SSE) for satisfying the requirements of 10 CFR 100.23.

The following is an abbreviated discussion of the step-wise procedure outlined in the guide to determine the SSE at a site. This procedure is schematically illustrated after describing the steps.

1. Regional and site geological, seismological, and geophysical investigations should be performed.
2. For central and eastern US (CEUS) sites (sites east of the Rocky Mountains), the Lawrence Livermore National Laboratory (LLNL) (Ref.6) or the Electrical Power Research Institute (EPRI) (Ref.7) Probabilistic Seismic Hazard Analysis (PSHA) should be performed using original or updated sources (based on the investigations performed in Step 1). For sites in other parts of the country, a site-specific PSHA should be accomplished. The ground motion estimates should be made for rock conditions in the free-field or by assuming hypothetical rock conditions for a non-rock site to develop the seismic hazard information base.
3. Using the reference probability (RP) of 1E-5 per year (rational for this value is described in Section 4.0) determine the 5% of the critically damped median spectral ground motion levels for the average of 5 and 10 Hz ($S_{5,10}$) and for the average of 1 and 2.5 Hz ($S_{1,2.5}$).
4. The median probabilistic hazard characterization should be deaggregated to determine the controlling earthquakes[1] magnitudes and distances.

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1. Given a reference probability (expressed as an annual probability of exceeding a ground motion level), the total seismic hazard can be de-aggregated to obtain contributions from different magnitude and distance events. The earthquakes which contribute most to this hazard are then called controlling earthquakes. This concept is schematically illustrated later.

After completing the PSHA and determining the controlling earthquakes, the following procedure should be used to determine the SSE.

5. With the controlling earthquakes determined as described above and using the procedures in Standard Review Plan (SRP) Section 2.5.2 (Ref.8), develop 5% of critical damping response spectral shapes for the actual or assumed rock conditions. The same controlling earthquakes are also used to derive vertical response spectral shapes.
6. Use S_{5-10} to scale the response spectrum shape corresponding to the controlling earthquake.
7. For nonrock sites, perform a site-specific soil amplification analysis considering uncertainties in site-specific geotechnical properties and parameters to determine response spectra at the free ground surface in the free-field for the actual site conditions.
8. Compare the smooth SSE spectrum or spectra used in design (e.g., 0.3g, broad-band spectra used in advanced light-water reactor designs) with the spectrum or spectra determined in Step 2 for rock sites or determined in Step 3 for nonrock sites to assess the adequacy of the SSE spectrum or spectra.

   When site-specific design response spectra are needed, to obtain an adequate design SSE based on the site-specific response spectrum or spectra, develop a smooth spectrum or spectra or use a standard broad band shape that envelopes the spectra of Step 6 or Step 7.

The concept of the methodology to estimate controlling earthquakes, outlined in steps 2 through 4, is illustrated in Figures 1,2, and 3. Figure 1 shows the total median seismic hazard curve in terms of 5 and 10 Hz spectral values. This figure also shows the ground motion levels at the reference probability, S_5 and S_{10}. The S_{5-10} is obtained by averaging S_5 and S_{10}. Figure 2 shows median seismic hazard curves for a set of magnitude and distance intervals defined in Table 1. Figure 3 shows graphically the contributions of magnitude and distance intervals to the ground motion level, S_{5-10}. In this figure, the major contributing earthquakes are nearby and of moderate size. Thus, in concept, this defines the notion of a controlling earthquake. Mathematically, the controlling earthquake are determined using the following equations:

$$M_c = \frac{\sum_m \sum_d m\overline{H_{md}}}{\sum_m \sum_d \overline{H_{md}}} \qquad Log(D_c) = \frac{\sum_m \sum_d Log(d)\overline{H_{md}}}{\sum_m \sum_d \overline{H_{md}}}$$

where M_c and D_c are magnitude and distance values of the controlling event. H_{md} is the average seismic hazard values of 5 and 10 Hz for each magnitude and distance interval (See Figure 2) estimated at ground motion levels for the reference probability.
Table 2 shows estimates of controlling earthquakes for several CEUS sites for the ground motion level corresponding to the reference probability of 1E-5/yr using LLNL median hazard results.

Once the controlling earthquake is determined, site specific spectral shape is derived using Ref.8. Figure 4 through 6 illustrate how the site specific spectral shape is used to develop SSE spectra or show adequacy of the previously selected SSE spectra (Steps 6 through 8). For engineering purposes, it is essential that the design ground motion response

spectra be a broad-band smooth response spectra with adequate energy in the frequencies of interest. In the past, it was general practice to select standard broad-band spectra, such as the spectra in Regulatory Guide 1.60 (Ref.9), and scale them by a peak ground motion parameter (usually peak ground acceleration (PGA)), which is derived based on the size of the controlling earthquake. In the past licensing review these spectra were checked against site-specific spectral estimates to be sure that the SSE design spectra adequately enveloped the site-specific spectra. These past practices to define the SSE are still valid and, based on this consideration, the following three possible situations are depicted in Figures 4 through 6.

Figure 4 depicts a situation in which a site is to be used for a certified design with an established SSE (for instance, an Advanced Light Water Reactor with 0.3g PGA SSE). In this example, the certified design SSE spectrum compares favorably with the site-specific response spectra determined in Step 6 or 7.

Figure 5 depicts a situation in which a standard broad-band shape is selected and its amplitude is scaled so that the design SSE envelopes the site-specific spectra.

Figure 6 depicts a situation in which a specific smooth shape for the design SSE spectrum is developed to envelope the site-specific spectra. In this case, it is particularly important to be sure that the SSE contains adequate energy in the frequency range of engineering interest and is sufficiently broad-band.

In the regulatory guide the probabilistic approach has been chosen for several reasons. The probabilistic methods have been used in the licensing of several plants when issues of different interpretations have arisen by applying the deterministic procedures of Appendix A to Part 100. Two major probabilistic studies and databases exist for CEUS which facilitate uniform and reproducible implementation of the probabilistic methods. The probabilistic approach explicitly considers the likelihood of an event or recurrence period. The need to perform PSHAs is important in characterizing seismic sources in the CEUS due to relatively low seismicity and the inability to associate earthquakes with specific tectonic structures. However, experience in performing seismic hazard evaluations in active plate-margins regions in the western United States has also identified uncertainties associated with the characterization of seismic sources. Sources of uncertainties include fault geometry, rupture segmentation, rupture extent, seismic activity rate, ground motion, and earthquake occurrence modeling. As in the case for sites in the CEUS, alternative hypotheses and parameters must be considered to account for these uncertainties. Thus, the probabilistic approach is deemed as one acceptable approach in addressing the uncertainties in determining the SSE. However, as indicated in the rule and the regulatory guide, alternate approaches can be used to address uncertainties. Decision on how to best approach the issue of uncertainty depends on the knowledge of tectonics, seismic sources, historical records, and feasibility to do exploratory work.

4.0 REFERENCE PROBABILITY (RP)

One of the key parameters in implementing a probabilistic method is the reference probability (RP). In Reference 4 the RP of 1E-5/yr has been defined considering the design basis of certain recently licensed plants in CEUS. The RP is the annual probability level such that 50% of a set of currently operating plants has an annual median probability of exceeding the SSE below this level. The RP is determined for the annual probability of exceeding the average of the 5 and 10 Hz SSE response spectrum ordinates associated with 5% of critical damping.

The RP was calculated using the LLNL methodology and results, but is also considered applicable for the EPRI study. The selected plants represent relatively recent

designs that used Regulatory Guide 1.60 or similar spectra as their design bases. The use of these plants should ensure an adequate level of conservatism in determining a SSE consistent with recent licensing decisions.

The following procedure was used to determine the RP and should be used in the future if general revisions to PSHA methods or data bases result in significant changes in hazard predictions for the selected plant sites.

1. Using LLNL methodology the median seismic hazard results were calculated for the selected sites for spectral responses at 5 and 10 Hz.

2. The composite annual probability of exceeding the plant SSE values for spectral responses at 5 and 10 Hz was calculated using median hazard estimates of Step 1. The composite annual probability is determined by:

   Composite probability = 1/2(a1) + 1/2(a2)

   where a1 and a2 represent median annual probabilities of exceeding SSE spectral ordinates at 5 and 10 Hz, respectively. The procedure is illustrated in Figure 7.

3. Figure 8 illustrates the distribution of median probabilities of exceeding the SSEs for selected 35 plants. The reference probability is simply the median probability of this distribution.

It should be noted that this RP is calibrated with the past design bases, it is not derived directly from any quantitative risk or safety goals. In fact, one of the reasons for using the median hazard curve in the regulatory guide approach is that the controlling earthquakes resulting from the de-aggregation of the median hazard curve are very similar to those used in the past licensing from the deterministic procedures. Table 3 shows a comparison of controlling earthquakes derived from the regulatory guide approach with those used in the past design. The regulatory guide recognizes that the final SSE at a higher RP may be more appropriate and acceptable for some sites considering the slope characteristics of the site hazard curves, the overall uncertainty in calculations (i.e., differences between mean and median hazard estimates), and the knowledge of the seismic sources that contribute to the hazard. The guide references a procedure, Ref.10, to determine an alternative reference probability on the risk-based considerations

A mean risk goal, such as seismically induced mean core damage frequency (cdf), is computed by a convolution of mean hazard with mean fragility, where fragility is the conditional probability of cdf given a particular hazard level. The relationship can be expressed as follows:

$$\text{mean goal} = \int \text{mean hazard} \bullet \text{mean fragility}$$

This relationship can be visualized from the schematic representation in Figure 9. The risk computations take into the account the entire hazard curve while the design basis is established based on the exceedance probability at one ground motion level. Clearly, the convolution of hazard curve 1 in Figure 9 with the fragility will produce higher mean cdf than

the convolution of hazard curve 2. The slope of the hazard curve is an important parameter computing a mean risk goal.

For the CEUS sites, as shown in Figure 10, the median reference probability of 1E-5/yr corresponds to the mean probability of 1E-4. That is, the use of either mean of 1E-4 or median of 1E-5 will result in roughly the same ground motion level. Figure 11 shows situations where the risk implications may be quite different based on the slope of the overall hazard curve and the nature of uncertainty. Figure 11 shows a hypothetical CEUS and western US (WUS) situation. Since the mean and median are shown closer for the WUS site, use of the median reference probability from the CEUS site would imply that WUS plants should be designed to a more stringent risk criteria. In other words, if the risk-based considerations are to be applied, it may be more appropriate to use a different, and perhaps mean-based, reference probability. For the hypothetical situation shown in Figure 11, while the use of mean RP of 1E-4 would result into the same ground motion level, S_{CEUS}, as using the median RP of 1E-5 for the CEUS site, there will be a substantial difference in the ground motion levels S_{W1} and S_{W2} resulting from the use of median RP of 1E-5 versus the use of mean RP of 1E-4.

ACKNOWLEDGMENT

Many people have participated in the revision of rule and preparation of the regulatory guides described in this paper. Don Bernreuter, Auguste Boissonnade, and Jean Savy of LLNL were primary NRC contractors conducting technical studies. An expert panel consisting of Dr. Robert Budnitz, Prof. C. Allin Cornell, Dr. Kevin Coppersmith, Mr. James Devine, Dr. Walter Hays, Dr. Robert Kennedy, and Dr. Paul Pomeroy provided critical reviews. Many of the concepts emerged during the interactions with the panel. There were also intensive interactions with industry groups. The seismic ad-hoc group established by the Nuclear Energy Institute provided many comments and conducted technical studies which were also used in the development of the regulation and guidance. The NRC staff members, Mr. Goutam Bagchi, Dr. Phyllis Sobel, and Dr. Abou-Bakr Ibrahim were also integral members of the team which was charged to revise the rule. Finally, the comments provided by many individuals and institutions during the two public review periods help clarify many issues and have shaped the final rule.

DISCLAIMER

The views expressed in this paper are those of the authors and should not be construed to reflect the official U.S.NRC position.

REFERENCES

1. Code of Federal Regulations, Title 10, Energy (1997), Appendix A, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," to 10 CFR Part 100, "Reactor Siting Criteria."
2. USNRC, "Identification of Issues Pertaining to Seismic and Geologic Siting Regulation, Policy, and Practice for Nuclear Power Plants," SECY-79-300, April 27, 1979.

3. Code of Federal Regulations, Title 10, Energy (1997), Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," to 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities."
4. USNRC, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Ground Motion," Regulatory Guide 1.165, March, 1997.
5. Code of Federal Regulations, Title 10, Energy (1997), Section 100.23, "Geologic and Seismic Siting Factors," of 10 CFR Part 100, "Reactor Siting Criteria."
6. P. Sobel, "Revised Livermore Seismic Hazard Estimates for Sixty-Nine Nuclear Power Plant Sites East of the Rocky Mountains," NUREG-1488, USNRC, April, 1994.
7. Electric Power Research Institute, "Probabilistic Seismic Hazard Evaluations at Nuclear Power Plant Sites in the Central and Eastern United States," NP-4726, All Volumes, 1989-1991.
8. USNRC, "Vibratory Ground Motion," Section 2.5.2, Revision 3, of "Standard Review Plan," NUREG-0800, March 1997.
9. USNRC, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Regulatory Guide 1.60.
10. Attachment to Letter from D. J. Modeen, Nuclear Energy Institute, to A.J. Murphy, USNRC, Subject: Seismic Siting Decision Process, May 25, 1994.

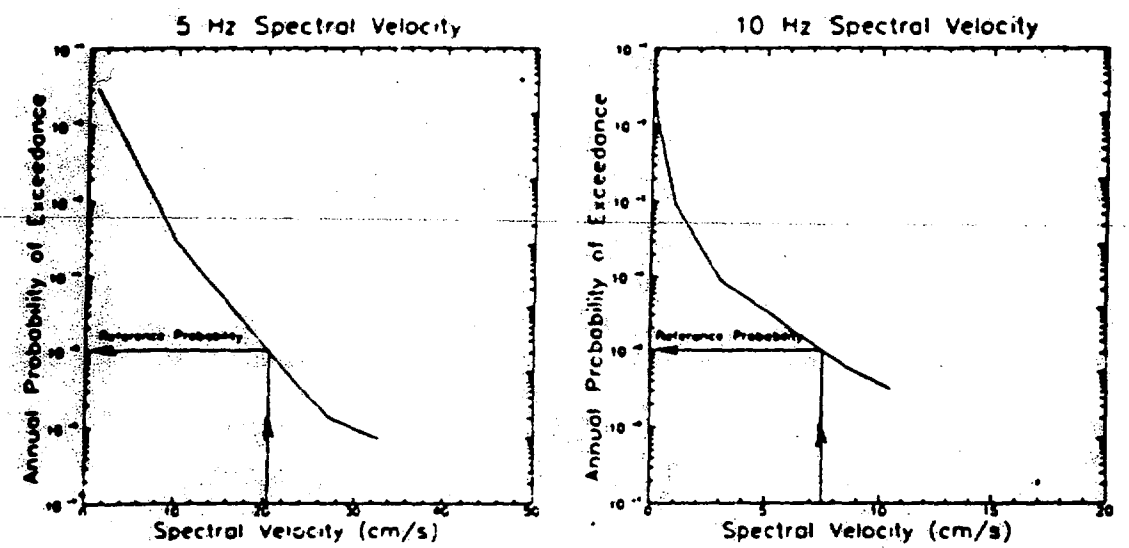

Fig.1 Total median seismic hazard for a site

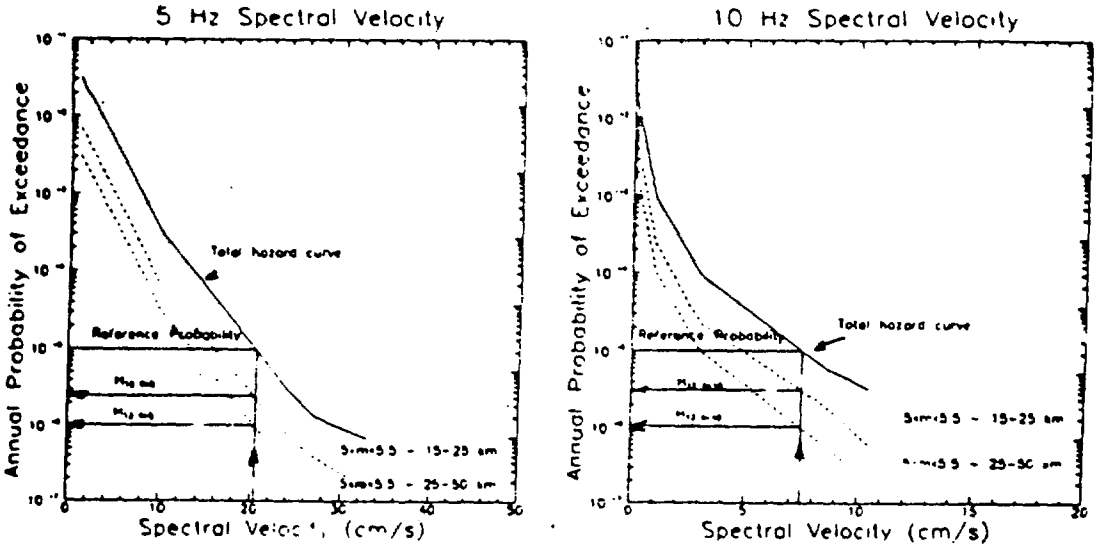

Fig.2 Deaggregated median seismic hazard for a site

Table 1
Contribution of Magnitude-Distance Intervals to Total Hazard

Distance Range of Bin (km)	Magnitude Range of Bin				
	5 - 5.5	5.5 - 6	6 - 6.5	6.5 - 7	> 7
0-15	0.417	0.097	0.000	0.000	0.000
15-25	0.220	0.079	0.000	0.000	0.000
25-50	0.080	0.042	0.000	0.000	0.000
50-100	0.004	0.014	0.001	0.000	0.000
100-200	0.000	0.008	0.031	0.000	0.000
200-300	0.000	0.001	0.004	0.000	0.000
300	0.000	0.000	0.000	0.000	0.002

Table 2
Estimates of Controlling Earthquakes

Site No.	Controlling Earthquake (Case 1)	
	Magnitude	Distance (km)
1	5.7	23
2	5.8	18
3	5.8	18
4	5.5	19
5	5.7	19
6	5.6	18
7	5.5	20
8	5.5	21

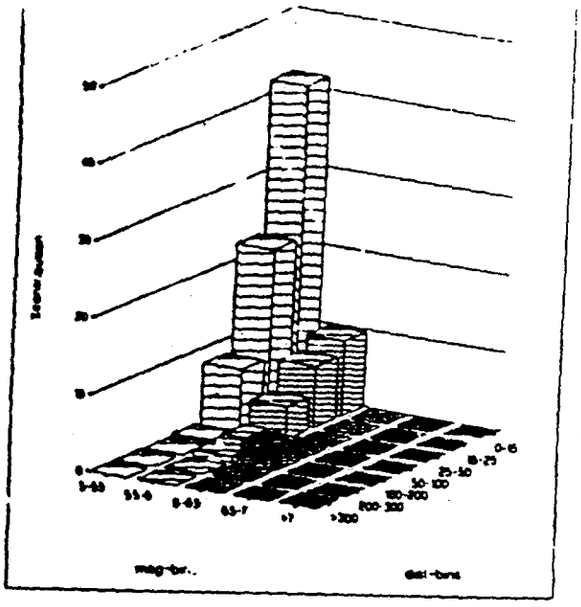

Fig 3 Contribution of magnitude-distance intervals for the average of 5 and 10 Hz spectral accelerations

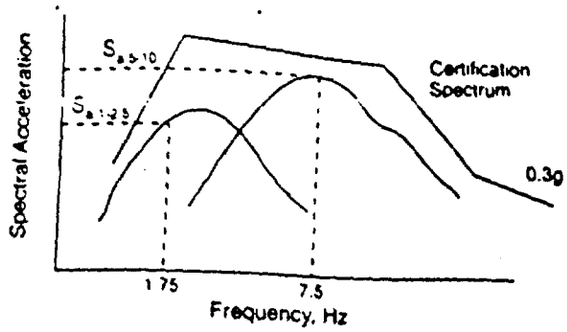

Figure 4 Use of SSE Spectrum of a Certified Design

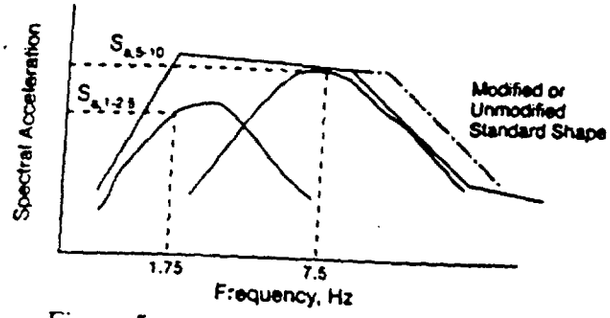

Figure 5 Use of a Standard Shape for SSE

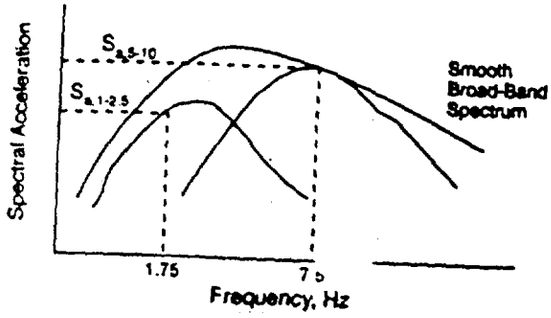

Figure 6 Development of a Site-Specific SSE Spectrum

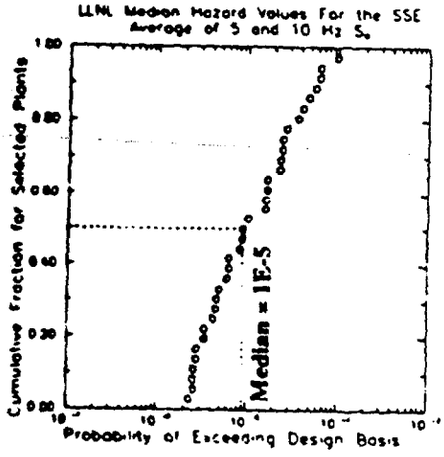

Fig.8 Distribution of the probability of exceeding the SSE at 5-10 Hz

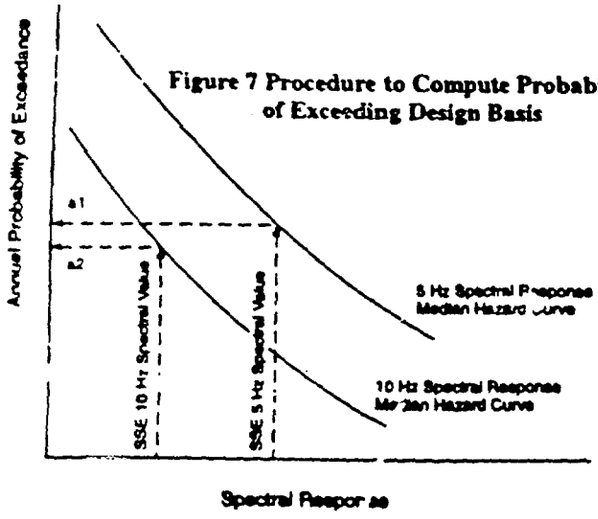

Table 3
Comparison between Controlling Earthquakes
and Past Seismic Design Criteria

Site No	Controlling Earthquakes		Past Seismic Design	
	Magnitude	Distance (km)	Magnitude	Distance (km)
1	5.4	18	5.0	15
2	5.6 7.2	24 275	5.8 7	15 250
3	5.5	14	5.3	15
4	5.6	14	5.3	15
5	5.7	14	5.7	15
6	5.5	16	5.3	15
7	5.3 7.3	18 340	4.8 7.3	15 370
8	5.7	14	6	15
9	5.6	14	5.8	15

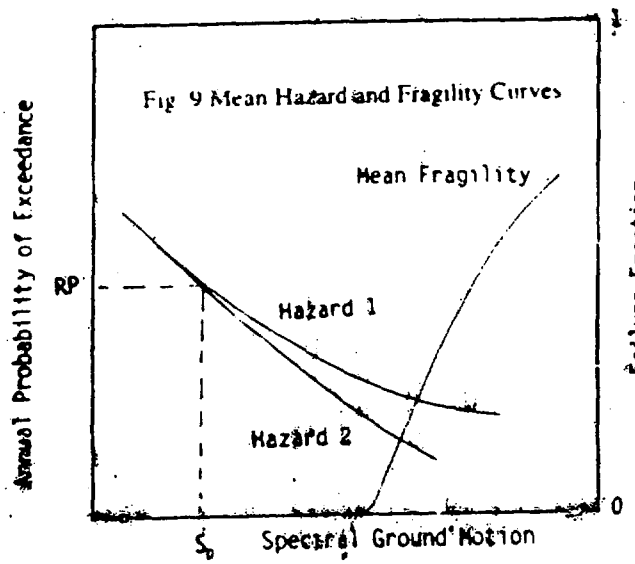

Fig 9 Mean Hazard and Fragility Curves

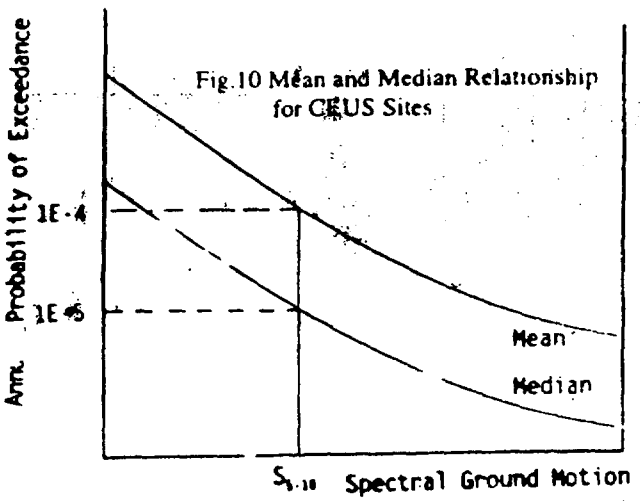

Fig.10 Mean and Median Relationship for CEUS Sites

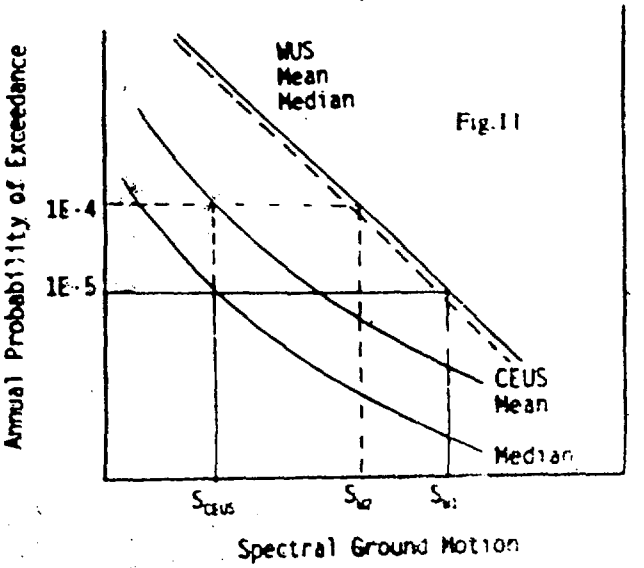

Fig.11

DOEF 1325.8
(8-89)
EFG (07-90)

United States Government **Department of Energy**

memorandum

DATE: August 22, 2001

REPLY TO ATTN OF: Office of Nuclear and Facility Safety Policy:HChander:301-903-6681

SUBJECT: REVISED DOE STANDARD 1020-2001, NATURAL PHENOMENA HAZARDS DESIGN AND EVALUATION CRITERIA FOR DEPARTMENT OF ENERGY FACILITIES, PROJECT NUMBER NPHZ - 0001

TO: Technical Standards Program Managers

The subject technical standard has been revised to conform to latest industry codes/standards and is released for your review and comment. The technical standard can be found at the Technical Standards Program Web Site at http://tis.eh.doe.gov/techstds/. After comments have been resolved, the document will be approved as a DOE standard and listed in the DOE standards Index, DOE-TSL-1.

Please review the document and provide your comments to the preparer, Dr. Harish Chander, EH-53, by the comment due date (45 day coordination period) listed for this project at the above Web Site. Your comments must be designated as either essential or suggested and proposed resolutions to those comments provided. Essential comments are those which, if not addressed, would make the document technically unacceptable to your organization and must be supported by detailed rationale. All comments must be in the form of word-for-word changes to the draft document.

Responses from DOE Area Offices, Laboratories, and M&O Contractors should be returned through the appropriate DOE management or organization channels in sufficient time to permit consolidation by the Operations Office and subsequent transmittal to the preparer before the due date. Comments received after that date will be held for the next revision, unless it is possible to address them without affecting the timely approval of the document. Please contact Dr. Chander if you have any questions on this DOE Technical Standards project. Dr. Chander can be reached at 301-903-6681. Please e-mail questions to Dr. Chander at harish.chander@eh.doe.gov.

Richard L. Black

Richard L.&lack, Director
Office of Nuclear and
Facility Safety Policy

State's
Exhibit 126

UT-47027

Table C-3 Seismic Performance Goals & Specified Seismic Hazard Probabilities

Performance Category	Target Seismic Performance Goal, P_F	Seismic Hazard Exceedance Probability, P_H	Risk Reduction Ratio, R_R
1	$1x10^{-3}$**	$4x10^{-4}$*	
2	$5x10^{-4}$**	$4x10^{-4}$*	
3	$1x10^{-4}$	$4x10^{-4}$* ($1x10^{-3}$)[1]	4 (10)[1]
4	$1x10^{-5}$	$1x10^{-4}$ ($2x10^{-4}$)[1]	10 (20)[1]

* The seismic exceedence probability is based on USGS maps generated in 1997 (and included in IBC 2000) for 2% exceedence probabiity in 50 years.

** The design methodology of IBC 2000 for Seismic Use Groups I and III achieves approximately performance goals of PC-1 & PC-2 respectively though it does not meet the relationship shown in equation C-1 for the seismic provisions.

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

Different structures, systems, or components may have different specified performance goal probabilities, P_F. It is required that for each structure, system, or component, either: (1) the performance goal category; or (2) the hazard probability (P_H) or the DBE together with the appropriate R_R factor will be specified in a design specification or implementation document that invokes these criteria. As shown in Table C-3, the recommended hazard exceedance probabilities and performance goal exceedance probabilities are different. These differences indicate that conservatism must be introduced in the seismic behavior evaluation approach to achieve the required risk reduction ratio, R_R. In earthquake evaluation, there are many places where conservatism can be introduced, including:

1. Maximum design/evaluation ground acceleration and velocity.
2. Response spectra amplification.
3. Damping.
4. Analysis methods.
5. Specification of material strengths.
6. Estimation of structural capacity.
7. Load or scale factors.
8. Importance factors/multipliers.
9. Limits on inelastic behavior.
10. Soil-structure interaction (except for frequency shifting due to SSI).
11. Effective peak ground motion.
12. Effects of a large foundation or foundation embedment.

For the earthquake evaluation criteria in this standard, conservatism is intentionally introduced and controlled by specifying (1) hazard exceedance probabilities, (2) load or scale

CNWRA 98-007

SEISMIC GROUND MOTION AT THREE MILE ISLAND UNIT 2 INDEPENDENT SPENT FUEL STORAGE INSTALLATION SITE IN IDAHO NATIONAL ENGINEERING AND ENVIRONMENTAL LABORATORY—FINAL REPORT

Prepared for

Nuclear Regulatory Commission
Contract NRC-02-97-009

Prepared by

Rui Chen
Asadul H. Chowdhury

Center for Nuclear Waste Regulatory Analyses
San Antonio, Texas

June 1998

NRC-00551

State's
Exhibit 127

It is interesting to note that the DOE's current 0.36-g horizontal design value for the proposed TMI-2 ISFSI soil site bounds the 2,000-yr return period probabilistic event (0.30 g, table 3-2). However, it does not bound the 10,000-yr return period event (0.47 g, table 3-2).

3.5 DEVELOPMENT OF DESIGN BASIS EARTHQUAKE PARAMETERS

To comply with DOE Standards 1020-94 and 1023-94 and to be consistent with NRC regulations, WCFS (1996b) developed DBE ground motion parameters for the proposed ISFSI site based on the WCFS (1996a) PSHA. These parameters were developed according to procedures described in DOE Standards 1023-94 and are in the form of acceleration response spectra and time histories as reviewed in more detail in section 2.5 and illustrated in figures 2-19 through 2-21 and tables 2-6.

NRC-00617

4 DISCUSSION AND RECOMMENDATION

4.1 DISCUSSION

Seismic issues that are important to siting the proposed TMI-2 storage facility include identification of potential seismic sources, source characteristics, and associated uncertainties; deterministic and probabilistic seismic hazard assessment using state-of-the-art knowledge and techniques, including ground motion attenuation predictions and spectral analyses; and development of design basis parameters in compliance with applicable regulations and regulatory guidance. These issues have been discussed in detail in chapters 2 and 3.

In the TMI-2 SAR, the DOE-ID (1996b) has proposed a seismic design horizontal acceleration of 0.36 g. This is based on seismic design criteria contained within the INEEL AE standards (U.S. Department of Energy, 1992) that provide technical direction and guidance to architects and engineers in the development of designs for construction-type work performed for DOE-ID at INEEL. The PHAs for rock in the AE standards are based on deterministic studies conducted in the 1970s (Woodward-Lungren and Associates, 1971; Woodward-Clyde Consultants, 1975; Allied Chemical Corporation, 1975; Agbabian Associates, 1977) and supported by the results of a further deterministic analysis conducted by WCC (1990). In the AE standards related to a reactor or similar higher risk facility, the peak design basis horizontal acceleration for the ICPP is 0.36 g, including effects of soil amplification. This corresponds to the 84th percentile of the 1970s DSHA studies. This acceleration was used as the design basis SSE for the chemical processing plant, and it is intended to serve as the design basis SSE for the proposed TMI-2 ISFSI at the ICPP.

However, DOE-ID continued DSHA to develop site-specific seismic design criteria for the proposed TMI-2 ISFSI site. This analysis was based in part on the results of the 1990 deterministic evaluation for INEEL (Woodward-Clyde Consultants, 1990) and recent fault-trenching studies conducted along the Lemhi and Lost River faults (Woodward-Clyde Consultants, 1992b; 1995). The Lemhi fault is the closest Basin and Range normal fault to the proposed TMI-2 ISFSI site and controls the deterministic seismic hazard. The resulting 50th- and 84th-percentile deterministic values of PHAs at the proposed TMI-2 ISFSI site are 0.34 g and 0.56 g, respectively.

Since the 1980s, the DOE-ID also conducted PSHA for INEEL. TERA Corporation (1984) performed a PSHA for the ANL—West Facility. The site-specific seismic hazard curves developed by TERA Corporation have been used by LLNL (Coats and Murrary, 1984) to calculate peak horizontal ground surface accelerations for the INEEL for various return periods. The PHAs for INEEL are 0.14, 0.21, and 0.24 g for return periods of 1,000, 5,000, and 10,000 yr. However, recently the DOE-ID has completed another probabilistic seismic hazard evaluation for facility areas of INEEL, including the ICPP (Woodward-Clyde Federal Services, 1996a). The methodology used in the new probabilistic study provides for explicit inclusion of the range of seismologic and tectonic interpretations including seismic source characterization and ground motion estimation consistent with approaches contained in NRC Guide 1.165 (Draft was DG-1032, Nuclear Regulatory Commission, 1997a). Based on this study, the PHAs for the proposed TMI-2 ISFSI site are 0.23, 0.30, and 0.47 g for return periods of 1,000, 2,000, and 10,000 yr, respectively.

In the light of new deterministic and probabilistic hazard assessment data, the DOE-ID has proposed to the NRC (U.S. Department of Energy-Idaho Operations Office, 1996b) for NRC acceptance

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of the DOE-ID SAR a DE value of 0.36 g that will envelope the 50th-percentile deterministic value of 0.34 g and 2,000-yr return period probabilistic value of 0.30 g.

The literature survey conducted herein indicates that the majority of the pertinent literature was produced by the DOE and its contractors or subcontractors. It is important to recognize that the DOE has sufficiently identified, utilized, and referenced previous as well as the state-of-the-art knowledge and information that exist in the literature in its site characterization efforts. Seismotectonic characteristics that are significant for seismic hazard evaluation at INEEL have been analyzed and potential seismic sources have been identified. Various studies, especially the recent probabilistic and deterministic seismic hazard analyses conducted by a DOE subcontractor, the WCFS (1996a,b), have also taken sufficient considerations of uncertainties associated with seismic source characteristics using the state-of-the-art investigation and analysis techniques. This study by WCFS (1996a,b) also included sophisticated sensitivity analyses that isolated the contributions to the total ground motion hazard produced by various potential seismic sources and the evaluated relative importance of various uncertainties associated with characterization of those seismic sources. Those analyses are consistent with recommendations in Appendix A of 10 CFR Part 100. In summary, the DOE seismic hazard analysis approach for the proposed TMI-2 ISFSI appears to be technically sound, and resultant ground motion values represent the best estimates. However, the DOE-proposed design PHA of 0.36 g does not bound the most recent 84th-percentile deterministic value of 0.56 g and 10,000-yr return period probabilistic value of 0.47 g. Therefore, a judgment of whether the DOE-design approach is acceptable depends on whether there are regulatory and technical bases to accept an ISFSI-design value that bounds the 50th-percentile deterministic value and the 2,000-yr return period probabilistic value.

Section 2.5.2.6 of NUREG-0800 (Nuclear Regulatory Commission, 1997b) provides guidance on assessing the SSE ground motion. Substantial uncertainties are inherent in deriving spectra for seismic ground motions, and the guidance states NRC preference that the 84th-percentile (median plus one standard deviation) response spectra be used for both spectral shape and ground motion amplitude estimates. Although a strict interpretation of 72.102(f)(1) may lead one to conclude that 0.56 g is the requisite DE value for the proposed TMI-2 ISFSI site, there is a regulatory basis for a different design value that may be adequate. In 1980, when 10 CFR Part 72 was first promulgated, ISFSIs were largely envisioned to be spent fuel pools or massive dry-storage structures. Moreover, ISFSIs were expected to be built at existing power plant sites where SSE values are already determined. In the Statements of Consideration (SOC) accompanying the initial rulemaking, the NRC recognized that the design PHA for dry casks and canisters need not be as high as a power reactor SSE: "For ISFSIs which do not involve massive structures, such as dry-storage casks and canisters (*sic*), the required design earthquake will be determined on a case-by-case basis until more experience is gained with licensing these types of units." With over 10 yr of experience licensing dry-cask storage, and robust analyses demonstrating cask behavior in accident scenarios, the NRC staff now have a reasonable technical basis to consider a different design PHA that is adequate for licensing dry storage ISFSIs.

The NRC selected the 84th-percentile DE for power reactors to provide an extra level of conservatism for those higher risk facilities. An operating ISFSI is inherently less hazardous and less vulnerable to earthquake-initiated accidents than is an operating nuclear power reactor (Hossain et al., 1997). Unlike a nuclear power plant, an ISFSI does not have an active nuclear reaction, and hence does not need to meet requirements for active cooling and safe shutdown systems in order to ensure the integrity of the high-pressure reactor coolant boundary and for shutting down the reactor in the event of a very large earthquake. Operations in an ISFSI facility need not be continuous but can be shut down,

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