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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	)	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel	)	
Storage Installation)	)	April 1, 2002

STATE OF UTAH'S PREFILED TESTIMONY ON UNIFIED CONTENTION UTAH L/QQ - GEOTECHNICAL

Enclosed for filing are the following:

1. Key Determinations.
2. Preface to and testimony of Barry Solomon (Geologic Setting)
3. Preface to and testimony of Dr. Steven F. Bartlett (Soils Characterization)
4. Preface to and testimony of Dr. Steven F. Bartlett & Dr. Farhang Ostadan (Dynamic Analysis)
5. Preface to and testimony of Dr. Mohsin Khan and Dr. Farhang Ostadan (Cask Stability).
6. Preface to and testimony of Dr. Walter J. Arabasz (Seismic Exemption)
7. Preface to and testimony of Dr. Farhang Ostadan & Dr. Steven F. Bartlett (Lack of Design Conservatism)
8. Preface to and testimony of Dr. Marvin Resnikoff (Radiation Exposure)
9. List of Exhibits for Unified Contention Utah L/QQ (State's Exhibit 91 to143).

Template= SECY-055

SECY-02

As a precautionary measure, the State is filing its Exhibit 107 and Answer No. 9 to the Cask Stability testimony as proprietary pleadings. By so doing, however, the State makes no claim as to their confidentiality.

DATED this 1st day of April, 2002.

Respectfully submitted,

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CERTIFICATE OF SERVICE

I hereby certify that a copy of STATE OF UTAH'S PREFILED TESTIMONY ON UNIFIED CONTENTION UTAH L/QQ - GEOTECHNICAL was served on the persons listed below by electronic mail (unless otherwise noted) with conforming copies by United States mail first class, this 1st day of April, 2002:

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STATE OF UTAH'S KEY DETERMINATIONS - UNIFIED CONTENTION UTAH L/QQ

I. The standards PFS must meet to obtain a Part 72 license.

A. Burden is on PFS to show it meets all the following regulations prior to license issuance.

1. Site specific soil stability investigations and laboratory analyses to demonstrate adequacy of foundation loading. 10 CFR § 72.102 (c) ("Sites other than bedrock sites must be evaluated for ... other soil instability due to vibratory ground motion") and (d) ("Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading").
2. SSCs designed to withstand the effects of earthquakes 10 CFR § 72.122(b)(2) (SSCs must be designed to withstand the effects of earthquakes without impairing their capability to perform safety functions. The design bases for these SSCs is the most severe reported natural phenomena. The ISFSI should also be designed to prevent massive collapse of building structures or the dropping of heavy objects as a result of building structural failure on the spent fuel or SSCs)
3. Exemption from Part 72 is authorized by law, will not endanger life or property or the common defense and security and is otherwise in the public interest. 10 CFR § 72.7.

II. All of the State's witnesses are well qualified experts

A. Expert witnesses are well qualified based on their education, training and experience.

III. Geologic Setting

A. PFS is located in a seismically active area: the Basin and Range physiographic province, the Intermountain seismic belt, and the Bonneville lake basin.

B. Capable faults are found in the area of the PFS site: Stansbury Fault 6 miles to the east, East Cedar Mountain fault 10 miles to the west; East fault 0.6 miles to the east; and the West fault 1.2 miles west of the PFS site.

C. Earthquakes in the range of magnitude 6.0 to 6.5 can occur in Skull Valley, even where no geologic evidence exists for Quaternary surface faulting.

IV. Characterization of Subsurface Soils (Unified Utah L/QQ - Section C) [Bartlett]

A. The Issue: Should PFS be required to conduct additional sampling and analysis as well as physical property testing for engineering analysis to demonstrate that the soils (and soil-cement), have an adequate margin against potential failure during a seismic event.

B. Findings of Fact for the Board to Make

1. Soil conditions are not adequate for the proposed foundation loading.
2. Sliding, overturning and bearing capacity are the failure modes for the pads and CTB.
3. PFS must meet a factor of safety against sliding of $\geq$1.1.
4. PFS has not met the 1.1. factor of safety against sliding for foundation failure modes.
5. PFS has not accounted for variation of shear strength properties across the pad area.
6. PFS has not taken soil variability into account in selecting design soil properties
7. Upper Lake Bonneville sediments are of critical importance because PFS relies on the shear strength of this layer to provide resistance to sliding.
   a. There has been extreme undersampling of the upper Lake Bonneville sediments and

PFS has not continuously samples/characterized depth of those sediments.

8. PFS's analysis is deficient because it has not conduced soil structure interaction and cyclic triaxial tests and triaxial extension tests.

C. Summary of conclusions:
   1. Based on PFS's design values, the upper Lake Bonneville sediments have inadequate shear strength to resist earthquake loading
   2. PFS has not demonstrated acceptable factors of safety against seismic sliding and bearing capacity failure for the pads or the CTB during a seismic event.

V. PFS's Proposed Use of Soil-Cement (Unified Utah L/QQ, C.3/ D.1.c)[Bartlett/Mitchell]

A. The Issue: Has PFS proven its soil-cement (cement-treated soil) design concept through qualified physical property testing and engineering analyses such that the CTB and storage pads can meet the 1.1. factor of safety against sliding by relying on soil-cement to provide dynamic stability to the CTB and storage pads foundation systems from a design basis earthquake?

B. Findings of Fact for the Board to Make
   1. Unique application of adding cement to soil to provide additional seismic sliding resistance and stability to shallowly embedded foundations from strong ground motions.
   2. No prior precedent for PFS's proposed use of soil-cement concept
   3. No site specific analyses and testing to verify that the design concept will perform as intended
   4. No analysis of the impact to the critical underlying native soils from the impact of construction and placement of cement-treated soil
   5. PFS's proposed post license soil cement program will not prove the design concept and there will be an inadequate and arbitrary basis for a licensing decision.

C. Summary of conclusions: PFS has not shown that use of soil cement will provide an acceptable seismic design for storage of spent nuclear fuel at the Skull Valley site.

VI. Seismic Design and Foundation Stability (Unified Utah L/QQ, D) [Ostadan/Bartlett]

A. The Issue: Do the storage pads, the CTB, their foundations systems, and the storage casks have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake?

B. Findings of Fact for the Board to Make
   1. PFS has a one-of-a kind design that is unprecedented and unproven, which results in an unconservative design, primarily due to the following design features:
      a. 4,000 unanchored casks sitting on shallowly embedded foundations with soil cement added to relatively soft soils to provide resistance to sliding.
      b. "Controlled" and in-phase sliding of the storage casks during a seismic event.
      c. Conflicting requirements: storage pads need to be rigid enough to allow controlled sliding but somewhat flexible for cask tipover; stiffness of cement treated soil is constrained by the cask tipover condition but must be stiff enough to provide resistance to pad sliding.
   2. The storage casks and the CTB and the storage pads, and their foundation systems, are "structure, systems and components important to safety."

3. Design basis ground motions based on a 2,000 year earthquake are 0.711g (horizontal) and 0.695g vertical.
4. Design basis ground motion based on deterministic seismic hazard analysis are 1.15g (horizontal) and 1.17g vertical.
5. Soils at the PFS site are compressibility, deformable and of relatively low strength
   a. No demonstration that soil cement and cement treated soil will provide an "engineering mechanism" to improve poor soil conditions
   b. Soil cement is not sufficiently durable over time to resist dynamic sliding.
      (1) Foundations overlying compressible soils will settle,
      (2) Soil cement may crack because of loadings or environmental conditions.
   c. Insufficient testing of soil strength/durability at DBE levels of strain
6. PFS in its seismic analysis has not demonstrated that its design has an adequate margin of safety.
   a. Adequacy of foundation design is a function of the dynamic forces imparted to them
   b. PFS has underestimated the seismic loads
      (1) Critical part of ICEC calculation for the storage pad is the forces acting on the pad
      (2) Holtec did not give ICEC total dynamic forces acting on the pad
      (3) Holtec calculation sensitive to input parameters (see Altran Report).
      (4) ICEC calculation shows pad foundation acts flexibly under seismic loads.
      (5) Fundamental frequency of pad vibration suggests the pads are flexible.
      (6) Flexible pad = less radiation damping and, thus, underestimation of seismic loads.
      (7) SWEC's use of pga in its structural analysis of the pads in computing dynamic forces is invalid - pga has nothing to do with cask/pad response
         (a) SWEC use of pga for pads is contradicted by SWEC analysis of CTB.
         (b) SWEC should have obtained and used correct acceleration from Holtec report.
   c. PFS's seismic analysis did not analyze pad-to-pad interaction - this results in PFS incorrectly calculating dynamic forces for stability:
      (1) Not realistic to assume a quadrant of pads will slide in unison.
      (2) Wave energy created from simultaneously vibrating pads at the natural frequency of the pads creates a source of energy that PFS's has not analyzed.
   d. During earthquake cycling separating/gapping of soil cement from the pads will occur, most liked along preexisting cracks; this will introduce out-of-phase motion.
      (1) If soil cement does not fail in compression then it may act as a strut and transfer inertial forces from pad to pad.
   e. PFS's reliance on soil cement buttress during a seismic event will be ineffective.
      (1) Separation and cracking of soil cement may occur from out-of-phase motion of CTB foundation mat and soil cement
      (2) Reduction of factor of safety against sliding to < 1.1 because stiff soil cement perimeter around CTB impacts soil spring and damping parameters and kinematic motion of mat foundation.
      (3) Underestimation of seismic loads - no valid determination that foundation mat is rigid which in turn means improper soil damping used in dynamic analysis.
   f. PFS has not considered cold bonding, potential variations in the motion of the pad

and the casks, and the sensitivity of Holtec's nonlinear analysis to input motion

g. PFS does not comply with ASCE 4-98 - PFS has not considered nonvertically propagating waves, accidental torsion and multiple set of time histories

C. Summary of Conclusions: Slight margin for error in PFS's design. PFS used erroneous assumptions and has not demonstrated unique features of its design will perform safety.

VII. Cask Stability and Cask Tipover (Unified Utah L/QQ, D.1.) [Khan/Ostadan]

A. The Issue: Will the free standing HI-STORM 100 casks experience excessive sliding, uplift, collision, or tip over under design basis ground motions at the PFS site?

B. Findings of Fact for the Board to Make

1. PFS has a one-of-a kind design that is unprecedented and unproven with no redundancies, thus, a comprehensive analysis and testing is necessary to determine whether the HI-STORM cask will excessively, slide, uplift, or tip over under the 2,000-year DBE.
2. The Altran independent analysis shows:
   a. Cask displacement varies significantly with the contact stiffness value and damping value used in the cask stability model.
   b. At the ground motions for a 2,000-year DBE, the HI-STORM 100 cask displacements may be significantly higher than estimated by the Holtec cask stability analysis and the HI-STORM 100 cask may tip over.
3. PFS's cask stability analysis performed by Holtec are not comprehensive or adequate to estimate cask behavior.
   a. The mathematical finite element code, Dynamo, used to analyze the cask stability for a 2,000-year DBE results are inconclusive.
      (1) Holtec has not quantified the limitations of Dynamo to handle cask rotation. If the cask rotation exceeds Dynamo's ability, Dynamo will produce erroneous results.
      (2) Dynamo has not been previously used to analyze cask behavior at sites with equivalent or greater ground motion than the 2,000-year DBE at PFS.
      (3) Cask displacement results have not been benchmarked with actual test data or another structural analysis code.
   b. Holtec used a large contact stiffness which may underestimate the actual cask displacement.
   c. Holtec used a large damping value which may also under estimate the actual cask displacement.
4. Contact stiffness in a dynamic analysis cannot be calculated with a simple formula so a range of reasonable contact stiffness values must be modeled.
5. The cask displacement results must be benchmarked against actual test data such as shake table data.

C. Summary of Conclusions: PFS's cask stability analysis is inconclusive and the State's independent analysis suggest the possibility that the casks will slide excessively, uplift, or over turn. This inability to accurately estimate cask behavior does not allow PFS to demonstrate that the casks and storage pads have adequate factors of safety to sustain the dynamic loading.

VIII. Seismic Exemption Request (Utah L/QQ, E) [Arabasz/Bartlett/ Ostadan/ Resnikoff]

A. The Issue: Is there sufficient conservatism built into PFS's ISFSI design such that its ISFSI design and subsequent consequences from a seismic event will not endanger life or property or the common defense and security and it is otherwise in the public interest to allow PFS a substantially lower design standard than mandated by the existing seismic hazard analysis regulations.

B. Findings of Fact for the Board to Make

1. Sufficient protection depends on both the probability of occurrence of the seismic event and the level of conservatism incorporated into the SSC design.
2. The DBE and seismic performance are inextricably linked, thus, in order to establish an appropriate DBE the seismic performance of the SSCs in concert with risk reduction ratios must be comprehensively evaluated.
   a. DOE Standard 1020 provides an acceptable methodology if followed in toto.
   b. Fragility curves for each PFS SSC are needed to determine the seismic performance and conservatism in the selected DBE.
   c. Performance goals are not inherent in ISFSI SRPs and must be determined on a site specific basis.
      (1) ISFSIs are not designed to meet SRPs for nuclear power plants.
      (2) The 1994, Kennedy and Short fragility curves for nuclear power plants did not assess the seismic performance of unanchored dry casks in seismic area with ground motions equivalent to the PFS site.
      (3) PFS's unprecedented, unconventional, one of a kind design and lack of redundancy requires the determination of the seismic performance for the cask, storage pad, and CTB.
3. The 2,000-year DBE is lower than that established by other entities for nuclear facilities and general building code standards.
4. Site specific circumstances in for 2,000-year DBE for INEEL TMI-2 ISFSI is not a clear precedent and has little if any bearing in this case.
5. Occupancy time to calculate the dose at the control boundary is 8,760 hours.
6. Allowable dose at the control boundary is exceeded as a result of a 2,000-year DBE.
7. Because of the potential for cask tipover, cannot rely solely on the non-mechanistic tipover analysis.
8. The initial angular velocity will be exceeded and the 45 g design basis for the canister will be exceeded.
9. Tipover may cause cask flattening, concrete cracking, stretching of the steel, and cask lid displacement which may result in an increase in dose.
10. Tipped over casks with bottoms facing the control boundary will increase the dose.
11. Potential cask uplift may exceed 45 g design basis limit for the canister.

C. Summary of Conclusions: A design basis earthquake for a 2,000-year return interval at the PFS site is not in the public interest, is not founded on a proper technical basis and may result in exceedance in the allowable dose at the control boundary.

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

State of Utah List of Hearing Exhibits - Unified Contention Utah L/QQ

State Exh. No.	Description	Witness	Contention
91	Curriculum Vitae of Barry Solomon	Solomon	L/QQ
92	Curriculum Vitae of Dr. Steven F. Bartlett	Bartlett/soils	L/QQ
93	NUREG-0800, *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants*, title page, page 3.8.5-7	Bartlett/soils	L/QQ
94	PFS Safety Analysis Report, page 2.6-45, Fig. 2.6-19	Bartlett/soils	L/QQ
95	Stone and Webster ("SWEC") G(B)04, Rev. 9, *Stability Analyses of Cask Storage Pads*, July 26, 2001, pages 1-2, 11, 15-17, 23, 32, 59	Bartlett/soils	L/QQ
96	SWEC Calc. No. G(B)13, Rev. 6, *Stability Analyses of Canister Transfer Building*, July 26, 2001, pages 1, 9-10, 23	Bartlett/soils	L/QQ
97	Reg. Guide 1.132, *Site Investigations for Foundations of Nuclear Power Plants*, Rev. 1 (March 1979), pages 1.132-1, -3, -5, -6, -21, -22	Bartlett/soils	L/QQ
98	Panel Deposition Transcript of Dr. Thomas Y. Chang and Dr. Paul Trudeau, November 15, 2000, title page, page 39	Bartlett/soils	L/QQ
99	Excerpt from Panel Deposition Transcript of Dr. Steven F. Bartlett and Dr. Farhang Ostadan (November 16, 2000), title pages 241-243; Figs. 1-8, Graphs prepared by Dr. Bartlett using data from ConeTec, Inc's Cone Penetration Testing Results of Soils at the PFS Facility, G(PO30), Rev. 1 (May 1999)	Bartlett/soils	L/QQ
100	Electric Power Research Institute (1990). "Manual on Estimating Soil Properties for Design," EPRI Report No. EL-6800, Research Project 1493-6 ("EPRI 1990")	Bartlett/soils	L/QQ
101	SWEC Calc. No. G(B)05, Rev. 2, *Document Bases for Geotechnical Parameters Provided in Geotechnical Design Criteria*, June 15, 2000, pages 1, 35	Bartlett/soils	L/QQ

State Exh. No.	Description	Witness	Contention
102	Makdisi, F. I., and Seed, H. B. (1978), "Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformation," American Society of Engineers Journal of Geotechnical Engineering Division, pp. 849 - 867, July 1978	Bartlett/soils	L/QQ
103	Figure 4-6 from EPRI, with additional caption by Dr. Bartlett	Bartlett/soils	L/QQ
104	Saye, S. and Ladd, C. C. (2000). "Design and Performance of the Foundation Stabilization Treatments for the Reconstruction of Interstate 15 in Salt Lake City," URS Corporation Speciality Conference, June 24, 2000	Bartlett/soils	L/QQ
105	Curriculum vitae of Dr. James K. Mitchell	Bartlett & Mitchell /soil cement	L/QQ
106	PFS Safety Analysis Report, pp. 2.5-108 through -121	Bartlett & Mitchell /soil cement	L/QQ
107	Confidential, Claimed Proprietary Engineering Services Scope of Work for Laboratory Testing of Soil-Cement Mixes between Stone and Webster and Applied Geotechnical Engineering Consultants, Inc. ("AGEC"), ESSOW No. 05995.02-G010 (Rev. 0), dated January 21, 2001	Bartlett & Mitchell /soil cement	L/QQ
108	Excerpts from the deposition transcript of Mr. Paul Trudeau (March 6, 2002), title page and pages 18, 33-34, 51-52, 67-68, 71-81, 88-89, 91-92, 96-99, 110	Bartlett & Mitchell /soil cement	L/QQ
109	Excerpts from the deposition transcript of Dr. Anwar Wissa (March 15, 2002), title page and pages 15-34, 42-44	Bartlett & Mitchell /soil cement	L/QQ
110	Curriculum Vitae of Dr. Farhang Ostadan	Barttlett & Ostadan /Dynamic Analysis	L/QQ
111	Various earthquake pictures and explanations	Barttlett & Ostadan /Dynamic Analysis	L/QQ

State Exh. No.	Description	Witness	Contention
112	Excerpts from the deposition transcript of Dr. Farhang Ostadan (March 8, 2002), title page and pages 89-120, Deposition Exhibit 31.	Barttlett & Ostadan /Dynamic Analysis	L/QQ
113	Excerpts from deposition transcript ("Tr.") of Dr. Wen-Shou Tseng (March 12, 2002), title page and pages 69-72.	Barttlett & Ostadan /Dynamic Analysis	L/QQ
114	Excerpts from deposition transcript of Mr. Paul Trudeau (March 6, 2002), title page and pages 37-44.	Barttlett & Ostadan /Dynamic Analysis	L/QQ
115	Excerpts from *Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage (PFS) Facility*, (March 8, 2002), Luk, Vincent K., et al, Sandia National Laboratory, title page and pages 32, 33, 35-37.	Barttlett & Ostadan /Dynamic Analysis	L/QQ
116	Excerpts from Calc. G(B) 04, Rev 9, Stability Analyses of Cask Storage Pads (March 30, 2001), title page and pages 14, 15, 46-51.	Barttlett & Ostadan /Dynamic Analysis	L/QQ
117	Excerpts from Calc. G(B)-13, Rev. 6, Stability of Canister Transfer Building (July 26, 2001), title page and 23.	Barttlett & Ostadan /Dynamic Analysis	L/QQ
118	ASCE 4-98 §1.1, title page, pages 1, 19, 20, 25.	Barttlett & Ostadan /Dynamic Analysis	L/QQ
119	Curriculum vitae of Dr. Mohsin R. Khan.	Khan & Ostadan /Cask Stability	L/QQ
120	Excerpts from Deposition Transcript of Dr. Krishna P. Singh and Dr. Alan I. Soler (March 6, 2002), title page, and pages 13-32, 41-44, 81-84.	Khan & Ostadan /Cask Stability	L/QQ

State Exh. No.	Description	Witness	Contention
121	Excerpts from Deposition Transcript of Dr. Krishna P. Singh and Dr. Alan I. Soler (November 15-16, 2001), title pages, and pages 33-40, 93-100.	Khan & Ostadan /Cask Stability	L/QQ
122	*Analytical Study of HI-STORM 100 Cask System Under High Seismic Condition*, Technical Report No. 01141-TR-000, Revision 0 (Dec. 11, 2001).	Khan & Ostadan /Cask Stability	L/QQ
123	Curriculum Vitae of Dr. Walter J. Arabasz.	Arabasz /Seismic Exemption	L/QQ
124	Staff Requirements Memorandum to William D. Travers dated November 19, 2001.	Arabasz /Seismic Exemption	L/QQ
125	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), pages 1-12.	Arabasz /Seismic Exemption	L/QQ
126	Revised DOE-STD-1020-2001, Table C-3 at C-6.	Arabasz /Seismic Exemption	L/QQ
127	Excerpts of 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), title page, 3-5, 4-1, 4-2.	Arabasz /Seismic Exemption	L/QQ
128	Excerpts to 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 Installation,"* (September 26, 2001), cover page, title page, page7.	Arabasz /Seismic Exemption	L/QQ
129	Excerpts from Declaration of Dr. C. Allin Cornell (Nov. 9, 2001), title page, pages 11-16, 27, Attachment A in its entirety.	Bartlett & Ostadan /Lack of Design Conservatism	L/QQ
130	Excerpts from Deposition Transcript of Dr. C. Allin Cornell (November 1, 2001), title page, page 49.	Bartlett & Ostadan /Lack of Design Conservatism	L/QQ

State Exh. No.	Description	Witness	Contention
131	Excerpts from Deposition Transcript of Dr. Krishna P. Singh and Dr. Alan I. Soler (November 16, 2001), title page, page 63.	Bartlett & Ostadan /Lack of Design Conservatism	L/QQ
132	Excerpts from DOE Standard 1020, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* page 2-24.	Bartlett & Ostadan /Lack of Design Conservatism	L/QQ
133	Excerpts from letter accompanying the Diablo Canyon Independent Spent Fuel Storage Installation License Application dated December 21, 2001, pages 1-4.	Bartlett & Ostadan /Lack of Design Conservatism	L/QQ
134	Curriculum Vitae of Dr. Marvin Resnikoff.	Resnikoff /Dose Exposure	L/QQ
135	Excerpts from the Holtec HI-STORM 100 Cask Certificate of Compliance for Spent Fuel Storage Casks (effective date May 31, 2000), docket number 72-1014, Appendix A 5.0-4; Appendix B, pages 3-8.	Resnikoff /Dose Exposure	L/QQ
136	PFS EIS Commitment Resolution Letter # 13 (September 25, 2001).	Resnikoff /Dose Exposure	L/QQ
137	Excerpts from Deposition Transcript of Everett Lee Redmond II (November 15, 2001), title page, pages 37-40, 45-52, 57-64.	Resnikoff /Dose Exposure	L/QQ
138	Excerpts from HI-STORM 100 Safety Evaluation Report, title page, pages 3-10, 11-5.	Resnikoff /Dose Exposure	L/QQ
139	Excerpts from HI-STORM 100 Topical Safety Analysis Report, HI-951312 (February 4, 2000), cover letter, title page, pages 1.D-4, 3.A-5 to 3.A-7, 3.A-15, Figure 3.A.18, 3.B-5, 11.2-6, 11.2-7.	Resnikoff /Dose Exposure	L/QQ
140	RWMA's Schematic Cross Section of HI-STORM 100 Cask Bottom.	Resnikoff /Dose Exposure	L/QQ

State Exh. No.	Description	Witness	Contention
141	RWMA's Rough Calculations: Dose Esmanating from Bottom of Tipped-Over Cask, pages 1-8.	Resnikoff /Dose Exposure	L/QQ
142	Excerpts of PFS SAR, Table 4.2-2, Rev. 12.	Resnikoff /Dose Exposure	L/QQ
143	RWMA Calculation of Neutron Dose at ElevatedConcrete Temperatures.	Resnikoff /Dose Exposure	L/QQ

STATE OF UTAH'S PREFACE TO TESTIMONY OF BARRY SOLOMON ON CONTENTION UTAH L/QQ - Seismic Setting

I. General Setting

A. PFS site is in the center of Skull Valley, about 24 miles south of the Great Salt Lake and 50 miles southwest of Salt Lake City.

B. For purposes of geological and geotechnical interpretation, Skull Valley lies within 3 regional zones: Basin and Range physiographic province; Intermountain seismic belt; and Bonneville Lake basin.

II. Basin & Range Physiographic Province

A. Extends east-west from Wasatch Range in central Utah to Sierra Nevada and north-south from southern Oregon and Idaho to northern Mexico.

B. Northern part of province has range-bounding faults with significant Quaternary displacement, commonly with active faults scarps on adjacent piedmont slopes.

C. Stansbury fault on the east side of Skull Valley is an active fault.

D. Wasatch fault zone is about 50 miles east of the proposed site – it is one of the longest (230 miles) and most active (up to M7.5) normal-slip faults in the world.

III. Intermountain Seismic Belt (ISB)

A. The ISB is a prominent north-tending zone of mostly shallow earthquakes about 60-120 miles wide; extends 900 miles from southern Nev./northern Ariz. to northwestern Mont.

B. Since 1900, 49 moderate to large earthquakes (M5.5 to 7.5) have been generated in the ISB.

1. Largest historic earthquake in Utah - 1934 Hansel Valley M6.6 earthquake, located at the northern end of the Great Salt Lake.
2. Largest recorded earthquake in the ISB - 1959 Hebgen Lake M7.5 earthquake in Montana near Yellowstone National Park.
3. In Utah, strong ground motions occurred from the 1962 Richmond M5.7 earthquake.

C. Lack of correlation in the ISB between scattered background seismicity and mapped Cenozoic faults.

1. Upper bound of background seismicity appears to be in the range of M6.0 to 6.5.
   a. Earthquakes up to this size can occur anywhere in the ISB, including Skull Valley, even where no geologic evidence exists for Quaternary surface faults.

IV. Bonneville Lake Basin

A. Bonneville lake basin is a geomorphic subbasin mostly occupying northwestern Utah.

1. It consists of a number of topographically closed structural basins that were hydrologically connected during major lacustral episodes.
2. Most recent major lake (about the time of the last major ice age) is Lake Bonneville.

B. Late Pleistocene deposits of Lake Bonneville are a significant component of foundation soils at the proposed PFS facility site.

1. Important datums for estimating age of latest Quaternary fault movement: the variations in lake level and shorelines resulting from major periods of persistent lake levels.
2. Two such shorelines, Stansbury and Provo, are present within the proposed PFS site.
3. Promontory soil – formed on alluvium and eolian deposits prior to the start of the Bonneville lake cycle – is another datum useful for estimating age of late Pleistocene fault movement.

V. Capable Faults found in the area of the proposed PFS site

A. Stansbury Fault located 6 miles east of the site.

B. East Cedar Mountain fault located 10 miles west of the site.

C. Geomatrix 1998 geologic investigation identified two mid-valley faults.

1. The East fault lies 0.5 miles east of the site.
2. The West fault 1.2 miles to the west.

VI. Ground Motions from Geomatrix probabilistic seismic hazard analysis (PSHA)

A. In 1999 Geomatrix calculated peak ground accelerations (pga) from 2,000-year earthquake to be 0.53g (horizontal) and 0.53g (vertical).

B. After further site investigation in 2001, revised pga calculated to be 0.711g (horizontal) and 0.695g (vertical).

1. There is approximately a 35% increase in pga from that computed in 1999.

VII. State of Utah's Remaining concerns are presented in other testimony

A. Dr. Steven F. Bartlett, soils characterization.
B. Dr. Steven F. Bartlett & Dr. James K. Mitchell, soil cement.
C. Dr. Steven F. Bartlett & Dr. Farhang Ostadan, dynamic analysis.
D. Dr. Farhang Ostadan & Dr. Mohsin Khan, cask stability analysis.
E. Dr. Walter Arabasz, seismic exemption.
F. Dr. Farhang Ostadan & Dr. Steven F. Bartlett, lack of design conservatism.
G. Dr. Marvin Resnikoff, radiation exposure.

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	)	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel	)	
Storage Installation)	)	April 1, 2002

STATE OF UTAH TESTIMONY OF BARRY SOLOMON ON UNIFIED CONTENTION UTAH L/QQ - GEOTECHNICAL (Geologic Setting)

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

A. 1: My name is Barry Solomon. I hold a Masters Degree in Geology from San Jose State University. I have twenty-seven years of experience in successfully developing, implementing, and managing various geologic studies. Specifically, I have studied geologic hazards and geologically characterized, screened and selected sites for hazardous, nuclear waste, construction, and mining projects. My studies are used to ensure that these projects comply with government regulations. A copy of my resume and list of publications are included as State's Exhibit 91.

I work for the Utah Geological Survey ("UGS") and have been with the UGS since September 1988. I serve generally two roles with the UGS. One is to conduct regional studies of geologic hazards and the other is to review geotechnical reports that are submitted to local governments by developers. I have mapped the Quaternary geology of Tooele Valley[1] – a valley directly to the east of Skull Valley. I was also involved in a study which used this mapping as the basis to delineate areas that have potential geologic hazards.[2]

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[1] Solomon, B.J., 1996, Surficial geology of the Oquirrh fault zone, Tooele County, Utah, *in* Lund, W.R., editor, The Oquirrh fault zone, Tooele County, Utah - surficial geology and paleoseismicity: Utah Geological Survey Special Study 88, p. 1-17.

[2] Black, B.D., Solomon, B.J., and Harty, K.M., 1999, Geology and geologic hazards of Tooele Valley and the West Desert Hazardous Industry Area, Tooele County, Utah: Utah Geological Survey Special Study 96, 65 p.

I am the co-principal investigator for a study funded by the National Earthquake Hazards Reduction Program ("NEHRP"). In this study I evaluate the potential for geologic hazards caused by a scenario earthquake on the Wasatch Fault Zone. I have also mapped the active West Cache fault zone and seismic hazards in northern Utah under other NEHRP grants and was previously the principal investigator for studies funded by the U.S. Environmental Protection Agency to investigate the relation of geology to the indoor-radon hazard in Utah. Through these two programs and other studies conducted at UGS, I have become very familiar with northern Utah's unique geological landscape.

My past experience includes employment with the Battelle Project Management Division, where I was a Geotechnical Advisor responsible for planning geotechnical surface-based site activities of the salt characterization program for siting of a high-level radioactive waste repository. I have worked for Breckinridge Minerals, Inc., where I directed all phases of the exploration for oil-shale and tar-sand deposits in the United States and Canada, and I was employed by the U. S. Geological Survey where I conducted resource evaluations and stratigraphic studies of minerals considered leaseable by the United States government. I have also worked for Fugro, Inc. as an engineering geologist, conducting regional and site-specific fault investigations and engineering-geologic studies of Quaternary deposits for potential nuclear power-plant sites in Arizona and Puerto Rico. I was responsible for site mapping, logging of core and soil samples, and trenching to evaluate geologic structure.

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

A. 2: The purpose of my testimony is to provide an overview of the geologic setting of the Private Fuel Storage, LLC ("PFS") spent nuclear fuel storage facility that is proposed to be located in Skull Valley, Utah.

Q. 3: What is your familiarity with the PFS project?

A. 3: I reviewed the PFS license application submitted to the NRC in 1997 and was one of the original sponsors of Contention Utah L. I was deposed by PFS on October 18, 2000 with respect to Basis 4 of Utah L (Collapsible Soils). I have reviewed relevant sections of the PFS Safety Analysis Report (SAR) and supporting calculations. I have been following the geotechnical issues in the PFS proceeding but my day-to-day involvement in the past few years has not been as great as it was during the late 1990s.

Q. 4: What is the general setting of the proposed site for the PFS facility?

A. 4: The proposed site for the PFS facility is located near the center of Skull Valley, about 24 miles south of Great Salt Lake and 50 miles southwest of Salt Lake City. The valley lies within three regional zones relevant to the interpretation of geological and geotechnical aspects of site suitability. These zones include the Basin and Range physiographic province, the Intermountain seismic belt, and the Bonneville lake basin.

Q. 5: Describe the Basin and Range physiographic province.

A. 5: The Basin and Range physiographic province extends east-west from the Wasatch Range in central Utah to the Sierra Nevada along the California-Nevada border, and north-south from southern Oregon and Idaho to northern Mexico (Stokes, 1977[3]). The northern part of this province, including Skull Valley, is characterized by asymmetrical mountain ranges separated by intervening valleys, both with north-south axes. This topography was created by late Cenozoic extension, or stretching, of the earth's crust that began about 17 to 14 million years ago and is ongoing (Hintze, 1988[4]). The extension resulted in range-bounding faults with significant Quaternary displacement, commonly with active fault scarps on adjacent piedmont slopes. One such active fault is the Stansbury fault on the east side of Skull Valley. Another is the Wasatch fault zone, one of the longest and most active normal-slip faults in the world. The Wasatch fault zone, about 50 miles east of the proposed site, forms the eastern boundary of the Basin and Range physiographic province in north-central Utah. This fault zone is 230 miles long, lies on the eastern edge of the Salt Lake City metropolitan area, and is capable of generating earthquakes as large as magnitude 7.5 (Machette and others, 1992[5]).

Q. 6: What effect does the Intermountain Seismic Belt have on seismicity in Skull Valley?

A. 6: The Stansbury fault and Skull Valley are along the western edge of the Intermountain seismic belt ("ISB"). This belt is a prominent north-trending zone of mostly shallow (less than 15 miles) earthquakes, about 60 to 120 miles wide, that extends at least 900 miles from southern Nevada and northern Arizona to northwestern Montana (Smith and Arabasz, 1991[6]). Contemporary deformation in the ISB is dominated by the same extension that characterizes the Basin and Range province. This extension has generated 49

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[3] Stokes, W.L., 1977, Subdivisions of the major physiographic provinces in Utah: Utah Geology, v. 4, no. 1, p. 1-17.

[4] Hintze, L.F., 1988, Geologic history of Utah – a field guide to Utah's rocks: Brigham Young University Special Publication 7, 202 p.

[5] Machette, M.N., Personius, S.F., and Nelson, A.R., 1992, Paleoseismology of the Wasatch fault zone – a summary of recent investigations, conclusions, and interpretations, *in* Gori, P.L., and Hays, W.W., editors, Assessment of regional earthquake hazard and risk along the Wasatch Front, Utah: U.S. Geological Survey Professional Paper 1500, p. A1-A71.

[6] 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., editors, Neotectonics of North America: Geological Society of America, Decade Map v. 1, p. 185-228.

moderate to large earthquakes (with magnitudes from 5.5 to 7.5) since 1900. These earthquakes include the largest historical earthquake in Utah, the 1934 magnitude 6.6 Hansel Valley earthquake in a sparsely populated area at the northern end of Great Salt Lake; the largest recorded earthquake in the ISB, the 1959 magnitude 7.5 Hebgen Lake earthquake in Montana near Yellowstone National Park; and the most damaging earthquake in Utah's history, the 1962 magnitude 5.7 Richmond earthquake in northern Utah, the only sizable earthquake in Utah for which strong-motion records currently exist. However, there is a lack of distinct correlation in the ISB between scattered background seismicity and mapped Cenozoic faults which may, in part, be due to uncertain subsurface geologic structure and discordance between surface fault patterns and seismic slip at depth. The upper bound of this background seismicity appears to be in the range of magnitude 6.0 to 6.5, representing the threshold of surface faulting in the ISB (Smith and Arabasz, 1991). Earthquakes up to that size can occur anywhere in the ISB, including Skull Valley, even where no geologic evidence exists for Quaternary surface faulting.

Q. 7: Does the Bonneville Lake Basin influence the Skull Valley soils and geology and if so how?

A. 7: Yes. In addition to being a structural basin within the Basin and Range physiographic province, Skull Valley is a geomorphic subbasin of the Bonneville lake basin (Oviatt and others, 1992[7]). The Bonneville lake basin, occupying northwestern Utah and small parts of adjacent Nevada and Idaho, consists of a number of topographically closed structural basins in the northeastern Basin and Range province that were hydrologically connected during major lacustral episodes. Lake Bonneville, the most recent major lake to have formed in the Bonneville lake basin, was essentially coincident with the last major ice age and persisted from about 30,000 to 10,000 radiocarbon years ago. Great Salt Lake and Utah Lake are two remnants of Lake Bonneville. Great Salt Lake is a saline lake with no outlet and Utah Lake is a freshwater lake that drains into Great Salt Lake through the Jordan River in Salt Lake Valley. Although other Quaternary lakes existed in the basin at various times prior to the Bonneville lake cycle, Lake Bonneville was the deepest and most extensive lake in the series and Late Pleistocene deposits of Lake Bonneville are a significant component of foundation soils for the proposed PFS facility. However, the lake level varied throughout its existence because of climate changes, changes in the relative proportion of inflow to the lake versus evaporative outflow, and the catastrophic failure of the lake threshold in southern Idaho. Variations in lake level are now well documented, and shorelines resulting from major periods of persistent lake levels are important datums for estimating the age of latest Quaternary fault movement. These shorelines include the Stansbury (from about 22,000 to 20,000 radiocarbon years ago), Bonneville (about 15,000

[7] Oviatt, C.G., Currey, D.R., and Sack, Dorothy, 1992, Radiocarbon chronology of Lake Bonneville, eastern Great Basin, USA: Palaeogeography, palaeoclimatology, Palaeoecology, v. 99, p. 225-241.

years ago), Provo (about 14,000 years ago), and Gilbert (from about 11,000 to 10,000 years ago) shorelines (Oviatt, 1997[8]). Two of these shorelines, the Stansbury and Provo, are present within the proposed PFS facility site area. Another datum useful for estimating the age of late Pleistocene fault movement is the Promontory soil. This soil was formed on alluvium and eolian deposits prior to the start of the Bonneville lake cycle. The relative degree of soil-profile development suggests that the Promontory soil is at least 50,000 to 60,000 years old and formed over a period of at least 20,000 to 30,000 years.

Q. 8: Are capable faults found in the area of the proposed PFS site?

A. 8: Because Skull Valley is typical of basins within the Basin and Range physiographic province and is located along the western edge of the seismically active Intermountain seismic belt, capable faults are found in the proposed PFS facility site area. However, capable faults are commonly expected to bound the basin and not lie within it. Many capable faults of this type are well-documented in the region, one of which is the Stansbury fault located about 6 miles east of the proposed site at the base of the Stansbury Mountains. Evidence suggests that the most recent event on the southern segment of the Stansbury fault is middle Holocene (Geomatrix Consultants, Inc., 2001[9]). The history of the East Cedar Mountains fault, located about 10 miles west of the proposed site at the base of the Cedar Mountains, is not documented as well, but the most recent movement on the East Cedar Mountains fault is assumed to be Quaternary (Geomatrix Consultants, Inc. 2001).

Q. 9: Please describe Geomatrix's geologic investigation of the proposed PFS site.

A. 9: Geomatrix – consultants to PFS – began geological and seismological investigations for the proposed facility in 1996 (Geomatrix Consultants, Inc., 2001). After field work and laboratory analyses, Geomatrix concluded that no capable faults lay closer to the proposed site than the Stansbury fault, and seismic hazard analysis and site design were based on this assumption. In response to questions raised by the State of Utah, PFS had Geomatrix perform additional work in 1998. This additional work identified two unnamed capable faults in the site vicinity, informally named the East and West faults, and a zone of distributive fault offset between the two faults. These faults are collectively referred to as the mid-valley faults. The East fault lies 0.6 miles east of the site and the West fault lies 1.2

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[8] Oviatt, C.G., 1997, Lake Bonneville fluctuations and global climate change: Geology, v. 25, no. 2, p. 155-158.

[9] Geomatrix Consultants, Inc., 2001, Final report – Fault evaluation study and seismic hazard assessment, revision 1: Oakland, California, unpublished consultant's report for the Private Fuel Storage Facility, Skull Valley, Utah, prepared for Stone & Webster Engineering Corp.

miles west of the site. Seismic-hazard analysis by the consultants shows that the dominant seismic sources are the Stansbury fault and the East fault (with its assumed northward projection into the Springline fault in northern Skull Valley). Concurrent with fault studies were studies to evaluate soil properties. Field investigations to obtain data for evaluation of soil properties were designed, in part, assuming relatively persistent Lake Bonneville stratigraphy.

Geomatrix also performed a probabilistic seismic hazard analysis, based on a 2,000-year design basis earthquake, to assess vibratory ground motion at the site. In 1999, Geomatrix calculated peak ground accelerations for the design basis earthquake to be 0.53g horizontal and 0.53g vertical. After further seismic site investigations in 2001, Geomatrix calculated peak ground accelerations for the design basis earthquake to be 0.711g horizontal and 0.695g vertical. These ground motions are approximately thirty five percent higher than those calculated in 1999.

Q. 10: To the best of your knowledge, does the geotechnical work to date satisfy the State's concerns?

A. 10: Even after the additional work, the State of Utah continues to question the conclusions of PFS and its consultants regarding characterization of subsurface soils and seismic design. These concerns are presented in the following testimony, which is being filed concurrently with this testimony:

(1) Dr. Steven F. Bartlett, characterization of subsurface soils.
(2) Dr. Steven F. Bartlett & Dr. James K. Mitchell, soil cement.
(3) Dr. Steven F. Bartlett & Dr. Farhang Ostadan, dynamic analysis.
(4) Dr. Farhang Ostadan & Dr. Mohsin Khan, cask stability analysis.
(5) Dr. Walter Arabasz, seismic exemption.
(6) Dr. Farhang Ostadan & Dr. Steven F. Bartlett, lack of design conservatism.
(7) Dr. Marvin Resnikoff, radiation exposure.

Q. 11: Does this conclude your testimony?

A. 11: Yes.

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BACKGROUND SUMMARY:

Twenty-six years of successful development, implementation, and management of geologic studies for evaluation of geologic hazards and for site screening, selection, and characterization of hazardous and nuclear waste, construction, and mining projects to comply with governmental regulations.

EDUCATION:

M.S. Geology, San Jose State University, 1979
B.A. Geology, University of California, Santa Barbara, 1972

ADDITIONAL COURSES, SEMINARS, AND TRAINING:

Geological Engineering
Geostatistics and Multivariate Analysis
Construction Management
Well Logging
Remote Sensing Techniques
Soils and Applied Geology
Quaternary Dating Methods
Reducing Radon in Structures

PUBLICATIONS:

More than fifty publications on geologic studies including quaternary geology and active faulting. Co-author of regulatory documents related to nuclear-waste and power-plant site characterization, selection, and screening.

PROFESSIONAL ASSOCIATIONS:

Association of Engineering Geologists (former Chairman, Utah Section)
Utah Geological Association (former Treasurer)
Geological Society of America
American Association of Petroleum Geologists
Northern California Geological Association (former Vice-President)
Registered Professional Geologist, State of Florida, No. PG0000318

State's
Exhibit 91

EMPLOYMENT HISTORY:

UTAH GEOLOGICAL SURVEY, Salt Lake City, Utah (1988-present)

Senior Geologist–Applied Geology Program: Principal investigator for National Earthquake Hazards Reduction Program grants to study active faults and seismic hazards in northern Utah. Principal investigator for U.S. Environmental Protection Agency grants to study geology related to the indoor-radon hazard in Utah. Responsible for review of geotechnical portion of license applications for low- and high-level radioactive waste disposal and storage sites to ensure compliance with state and federal regulations. Responsible for conducting site-specific and regional assessments of geologic hazards.

BATTELLE PROJECT MANAGEMENT DIVISION, Columbus, Ohio, and Hereford, Texas (1985 to 1988)

Geotechnical Advisor: Responsible for planning and direction of geotechnical surface-based site activities of the salt characterization program for siting of a high-level radioactive waste repository. Monitored the cost and technical status of geotechnical elements by preparing planning documents and work scopes.

BRECKINRIDGE MINERALS, INC., Salt Lake City, Utah (Southern Pacific Petroleum, Brisbane, Australia) (1980 to 1985)

Senior Geologist: Directed all phases of exploration for oil-shale and tar-sand deposits in the United States and Canada; participated in oil-shale exploration program in Australia. Managed program of lease acquisition and established field office. Conducted comprehensive geologic studies using mapping, core logging, geochemical analyses, and geophysical data to characterize potential mine sites.

U.S. GEOLOGICAL SURVEY, Menlo Park, California (1976 to 1980)

Geologist: Conducted resource evaluations and stratigraphic studies of minerals considered leasable by the U.S. Government. Provided recommendations to federal agencies regarding proper use of mineral resources on federal land.

FUGRO, INC., Long Beach, California (1973-1975)

Engineering Geologist: Conducted regional and site-specific fault investigations and engineering-geologic studies of Quaternary deposits for potential nuclear power-plant sites in Arizona and Puerto Rico. Responsible for site mapping, logging of core and soil samples, and trenching to evaluate structure.

STATE OF UTAH'S PREFACE TO STEVEN F. BARTLETT TESTIMONY ON CONTENTION UTAH L/QQ - Soils Characterization

I. Important aspects of Soils Characterization

A. Purpose of soil characterization: show that the soils have an adequate margin against potential failure during a seismic event.
B. PFS must show: soil conditions are adequate for the proposed foundation loading. 10 CFR § 72.102(d).
C. Showing made *inter alia* by meeting 1.1 factor of safety against sliding for foundation failure modes.
D. Sliding, overturning and bearing capacity are the failure modes for the pads and CTB.
E. Soils must be adequately sampled and characterized to establish their capacity to resist foundation loading with an acceptable factor of safety.
F. Soil variability must be taken into account in conservatively selecting design soil properties.
G. The upper Lake Bonneville sediments are of critical importance because PFS relies on the shear strength of this layer to provide resistance to sliding.

II. Factor of Safety Against Sliding (capacity over demand)

A. Capacity (soil's shear strength) over demand (strong ground motions from earthquake).
B. PFS has adopted a factor of safety of 1.1 (10% margin against foundation failure).
C. PFS stability calculation have small margins (about 6-15%) against seismic failure.
D. Safety implications if there are small decreases in the soil's shear strength below those used in design.

III. Inadequate Soil Sampling and Characterization

A. Inadequate borehole spacing - only 9 boreholes were drilled in or near the pad area.
B. PFS has not continuously sampled or characterized with depth the critical upper Lake Bonneville sediments.
C. Extreme undersampling of upper Lake Bonneville sediments.
   1. Sliding resistance of the pads based on sample taken from one borehole and one set of direct shear tests.
   2. No evidence that single datum in a 51 acre area is representative of upper Lake Bonneville sediments.
   3. Undersampling subject to severe bias and potential overestimation of shear strength.
D. No accounting for variation of shear strength properties across pad emplacement area.
   1. For static loading conditions under the pads, PFS estimates 2.1 ksf sliding shear strength.
   2. Shear strength of upper Lake Bonneville sediments may vary by factor of 2.
   3. Based on CPT logs undrained shear strength values could range from 1.4 ksf to 2.8 ksf.
   4. Unacceptable factor of safety against sliding will be obtained if undrained shear strength is 1.82 ksf or less (based on PFS's 1.27 factor of safety against sliding).
   5. Compared with one of the two direct shear tests for the CTB (1.75 ksf), shear strength values below 1.82 ksf are possible.
E. Data for only two undrained shear strength tests used in CTB dynamic bearing capacity calculations were taken from an area outside the footprint of the CTB.
   1. For the upper 28 ft., PFS used an weighted average of 3.18 ksf - unconsolidated

undrained (UU) test of 2.2 ksf adjusted by 1.64 for deeper soils (12-28 ft).

2. 1.64 adjustment factor gives potentially erroneous results based on sampling location.

F. Other tests that PFS should have conducted.

1. Strain-controlled cyclic triaxial tests and triaxial extension tests.
    a. Earthquake loadings are cyclic.
    b. PFS has used one directional loading without cycling to represent soil's shear resistance for the design of the pads and CTB foundations.
    c. Need to perform cyclic lab testing on undisturbed soil samples to ensure no significant loss/degradation of shear strength due to cycling.
2. Stress-strain behavior of native foundation soils under a range of cyclic loads.
    a. PFS relies upon pseudo-static analyses in its sliding and bearing capacity analyses of the foundations for the pads and CTB.
    b. For relatively heavy structures (casks, CTB) resting on deformable Lake Bonneville deposits, need to use soil structure interaction analysis to estimate dynamic stresses imposed on the soil, soil cement and cement-treated soil.

IV. Conclusion

A. Based on PFS's design values, the upper Lake Bonneville sediments have inadequate shear strength to resist earthquake loading.

B. PFS has not demonstrated acceptable factors of safety against seismic sliding and bearing capacity failure for the pads or the CTB.

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	)	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel Storage Installation)	)	April 1, 2002

STATE OF UTAH TESTIMONY OF DR. STEVEN F. BARTLETT ON UNIFIED CONTENTION UTAH L/QQ (Soils Characterization)

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

A. 1: My name is Dr. Steven F. Bartlett. I am an Assistant Professor in the Civil and Environmental Engineering Department of the University of Utah, where I teach undergraduate and graduate courses in geotechnical engineering and conduct research. I hold a B.S. degree in Geology from Brigham Young University and a Ph.D. in Civil Engineering from Brigham Young University. I am a licensed professional engineer in the State of Utah.

Prior to this University of Utah faculty position, I worked for the Utah Department of Transportation ("UDOT") as a research project manager and have held a number of other positions with UDOT and other employers where I have applied my expertise in geotechnical engineering, earthquake engineering, geoenvironmental engineering, applied statistics, and project management. My curriculum vitae is included as State's Exhibit 92.

I have also worked as a consulting engineer for 1996-1996 for Woodward-Clyde Consultants in Salt Lake City, mainly as a geotechnical designer for the I-15 Reconstruction Project.

Prior to my position at Woodward-Clyde Consultants, I worked from 1991-1995 for Department of Energy's ("DOE") contractor, Westinghouse, at the DOE Savannah River Site ("SRS"), near Aiken, South Carolina. I was Westinghouse's principal geotechnical investigator on a multi-disciplinary team overseeing the seismic qualification of the ITP/H-Area high-level radioactive waste storage tank farm for the SRS; the principal geotechnical investigator reviewing the Safety Analysis Report ("SAR") for the seismic qualification of the Defense Waste Processing Facility ("DWPF"), which is a high-level radioactive waste vitrification and storage facility at the SRS, and the project manager for the design of a hazardous waste landfill closure at the SRS. I used NRC regulatory guidance documents for my review of these projects.

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

A. 2: The purpose of my testimony is to explain the basis of my professional opinion that PFS has not adequately sampled or characterized the subsurface soils at the Skull Valley site, especially with respect to the upper Lake Bonneville sediments.

Q. 3: What has been your involvement in reviewing PFS's soils characterization and analysis?

A. 3: I have been assisting the State since 1999 and have reviewed PFS's soils investigation, boring logs, cone penetrometer testing, sliding and stability calculation. I assisted and gave technical support to the State in filing Contention Utah QQ and the two modifications thereto. I am familiar with sections of the SAR and calculation packages with respect to PFS's characterization of soils, the cone penetrometer testing, PFS's stability analyses and it seismic exemption request. Some of these topics are described in other testimony being filed concurrently with this testimony relating to soil cement, dynamic analysis and seismic exemption (lack of design conservatism).

Q. 4: What is the purpose of characterizing subsurface soils?

A. 4: The purpose of characterization of subsurface soils is to show that the soils have adequate margins against potential failure during a seismic event. The requirement is given in 10 CFR § 72.102(d):

> Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading.

Q. 5: How do you demonstrate that soil conditions are adequate for the proposed foundation loading?

A. 5: This demonstration is usually done by calculating a factor of safety against failure for a particular foundation failure mode.

Q. 6: What are the possible foundation failure modes?

A. 6: The possible failure modes considered by the applicant in its seismic design calculations for the foundations of the pads and Canister Transfer Building ("CTB") are sliding, overturning and bearing capacity failure.

Q. 7: What are "factors of safety" and how are they expressed?

A. 7: In general, factors of safety are expressed as the capacity of the system to resist failure divided by the demand placed on the system by the seismic event and other foundation loads.

Factor of safety (FS) = capacity of system / demand placed on the system.

For foundation systems, the capacity of the foundation is primarily a function of the soil's shear strength and the type, flexibility and embedment of the foundation. The demand is primarily a function of the intensity (*i.e.*, amplitude) of the earthquake strong ground motion and the mass and frequency of vibration of the foundation and the overlying structure.

For extreme environmental events, such as earthquakes, a generally accepted factor of safety against failure is 1.1. A factor of safety of 1.1 implies that there is a 10 percent margin against failure of the foundation system due to the extreme environmental event. This factor of safety is widely used by the engineering profession and is the same acceptance criterion found in NUREG-0800[1], 3.8.5, Section II, Subpart 5, Structural Acceptance Criteria for seismic Category I structures, p. 3.8.5-7, excerpt included as State's Exhibit 93.

Q. 8: What criterion for design has PFS adopted?

A. 8: PFS has adopted this criterion – the 1.1. factor of safety – for design of the PFS foundations as found in NUREG-75/87, which is an earlier version of NUREG-0800. PFS discusses the recommended factor of safety of 1.1 in its safety analysis report. SAR[2] at 2.6-45 (Rev. 21). It is expressed as a minimum design requirement in the seismic stability calculations for the storage pads (Stone and Webster Calculation G(B)04[3], Revision 9, p. 15-17) and for the Canister Transfer Building (SWEC Calc. G(B)13[4], Revision 6, p. 23).

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[1] NUREG-0800, *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants*.

[2] Excerpts included as State's Exhibit 94.

[3] G(B)04, Rev. 9, *Stability Analyses of Cask Storage Pads*, Stone and Webster ("SWEC"), July 26, 2001, excerpts included as State's Exhibit 95.

[4] G(B)13, Rev. 6, *Stability Analyses of Canister Transfer Building*, SWEC, July 26, 2001, excerpts included as State's Exhibit 96.

Q. 9: Is a factor of safety below 1.1 generally acceptable for extreme environmental events, such as earthquakes?

A. 9: The use of factors of safety below 1.1 for extreme environmental events is usually not allowed by the engineering profession. The primary reason for this is that factors of safety below this threshold can constitute an unstable or uncontrolled condition which can lead to unacceptable performance and significant damage or deformation of the foundation and its supported structures.

Q. 10: What can happen to the storage pads when subject to strong ground motion if their design does not meet a factor of safety of 1.1?

A. 10: In the case of the pads, sliding failure will cause out-of-phase motion of the pads and will significantly increase pad-to-pad interaction, especially in the longitudinal direction. Pad-to-pad interaction can be detrimental to cask stability, if the pounding effect is large, causing a significant transfer of inertial forces to adjacent pads and casks. Sliding failure can also change the frequency of vibration of the pads in the horizontal direction. Such failure will decrease the horizontal frequency of vibration and this frequency shift could have deleterious effects if the decreased frequency more closely matches the rocking or tipping frequency of the casks. Bearing capacity failure can cause a tilting or rotation of the pad, which will affect cask sliding. Any slight tilting of the pads will introduce asymmetrical cask sliding and increase the potential for cask-to-cask impact and overturning.

Q. 11: Does characterizing the soil have any effect on the sliding analysis of the pads and the CTB?

A. 11: Yes. The primary purpose of soil characterization is to gather sufficient information regarding the characteristics, properties and variability of the soils to establish their capacity to resist foundation loading with an acceptable factor of safety. The primary mechanism the Applicant has used to resist sliding and bearing capacity failure of the pads and Canister Transfer Building is the shear resistance (*i.e.*, shear strength) of the soil. Thus, for this site, it is extremely important that the soil's shear strength properties are accurately estimated at the PFS site.

Q. 12: Does soil variability affect their shear strength?

A. 12: Yes. Because soils are deposited and influenced by natural processes, there is inherent variability in their shear strength. This variability results from vertical and horizontal changes in soil type and is strongly influenced by other geological factors and processes such as soil density, void ratio, degree of consolidation, in situ moisture content, dessication (drying) and degree of natural cementation.

Q. 13: What is important in conservatively selecting design soil properties?

A. 13: Large sites with complex layering, such as the PFS site, require sufficient data and statistical analyses of critical layers to ensure that design soil properties have been conservatively selected and are supported by site-specific data. The applicant has not done this for the pad emplacement area.

Q. 14: What are the primary deficiencies in PFS's soil characterization for the pad emplacement area?

A. 14: i. The Applicant has not performed the recommended spacing of borings for the pad emplacement area as outlined in NRC Reg. Guide 1.132[5], Appendix C.

ii. The Applicant has not performed continuous sampling of critical soil layers important to foundation stability for each major structure as recommended by Reg. Guide 1.132, Part C6, Sampling. State's Exh. 97.

iii. The Applicant's design of the foundation systems is based on an insufficient number of tested samples, and on a laboratory shear strength testing program that does not include strain-controlled cyclic triaxial tests and triaxial extension tests.

iv. The Applicant has not adequately described the stress-strain behavior of the native foundation soils under the range of cyclic strains imposed by the design basis earthquake.

Q. 15: What is the significance of the deficiencies you just listed?

A. 15: The deficiencies and uncertainties in soil characterization and laboratory testing are important when viewed in relation to the small margins against seismic failure that have been calculated by the Applicant. For example, the Applicant has calculated factors of safety against sliding and bearing capacity failure of the storage pads of 1.27 and 1.17, respectively for specific loading cases (Calc. G(B)04-9, p. 23 and 59, respectively; State's Exh. 95). Similarly, the factor of safety against sliding of the Canister Transfer Building is 1.26 (Calc. G(B)13-6, p. 23; State's Exh. 96).

As more fully described in testimony by Dr. Farhang Ostadan and myself on dynamic analysis, we have several concerns with these calculations and believe them to be

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[5] Reg. Guide 1.132, *Site Investigations for Foundations of Nuclear Power Plants*, Rev. 1 (March 1979), excerpts included as State's Exhibit 97.

fraught with errors, omissions and unconservative assumptions which make the Applicant's conclusions about seismic stability incorrect. Many of the disputed issues deal with improper application of the seismic loading to the design of the foundations and their supported structures and the failure to consider soil-structure and pad-to-pad interaction.

However, delaying the discussion of these issues for a moment, it is clear that if the soil's capacity to resist earthquake forces has only about a 6 to 15 percent margin above the value required to produce an acceptable factor of safety, then variations or small decreases (about 5 to 15 percent) in the soil's shear strength below the values used in design are important and can lead to potentially unsafe conditions or conditions not considered and analyzed by the Applicant in the design of the storage pads and CTB.

Q. 16: Does PFS have adequate borehole spacings for the pad emplacement area?

A. 16: No. The Applicant has used guidance provided in Reg. Guide 1.132 to plan its field and laboratory investigations for the Canister Transfer Building (Trudeau and Chang Tr.[6], p. 39, lines 18-23; State's Exhibit 98). Appendix C of Reg. Guide 1.132 provides a table of spacing and depth of subsurface explorations for various types of safety related foundations. The Applicant has met the recommended density of sampling for the Canister Transfer Building, but has not done so for the pad emplacement area.

For linear structures (such as a row of storage pads), the recommended spacing is 1 boring per every 100 linear feet for favorable, uniform geologic conditions, where continuity of subsurface strata is found (Reg. Guide 1.132, p. 1.132-3, 1.132-21, 1.132-22; State's Exh. 97). Thus, based on this Reg. Guide table, a borehole spacing of 100 feet on-center seems appropriate.

Q. 17: What spacing did PFS use?

A. 17: The Applicant has used an approximate borehole and cone penetrometer ("CPT") spacing of about 221 feet for this area (SAR Figure 2.6-19, Rev. 22; State's Exh. 94). This approximate spacing was calculated by dividing the square foot area of the pad emplacement area (approximately 2,240,000 ft^2) by the number of boreholes (9) and CPT soundings (37) for a total of 46. This is about 1 boring or sounding for every 48,696 ft^2, or about 1 boring for every 221 feet for a regular grid pattern.

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[6] Panel Deposition Transcript of Dr. Thomas Y. Chang and Dr. Paul Trudeau, November 15, 2000.

Q. 18: What do you conclude about sampling in the pad area?

A. 18: The pad emplacement area has been significantly undersampled when compared with the Canister Transfer Building and with the borehole spacings recommended by Reg. Guide 1.132. This undersampling is even more acute when one considers that only 9 boreholes (A1, B1, C1, A2, B2, C2, A3, B3, C3) were drilled in or near the pad emplacement area for the purpose of retrieving samples for laboratory testing and analysis. It is my opinion that this is significant under-sampling of this approximate 51-acre site.

Q. 19: Has PFS performed Continuous Sampling?

A. 19: No. The Applicant has relied on results of laboratory shear strength testing to define that resistance of the soil to dynamic loading. Cone penetrometer soundings taken in the pad emplacement area show a notable decrease in penetration resistance in a zone beginning at a depth of about 3 feet below ground surface and extending to a depth of about 10 feet (Figures 1-8[7]). This layer is a silty-clay and clayey silt that has been identified as the upper Lake Bonneville sediments.

Q. 20: Are the engineering properties of the upper Lake Bonneville sediments important, and, if so, why?

A. 20: The engineering properties of this layer are very important because the Applicant relies on the shear strength of this layer to provide resistance to sliding (Calc. G(B)04-9, p. 11 and Calc. G(B)13-6, p. 9-10), State's Exhs. 95-96, respectively.

For critical layers, such as this one, Reg. Guide 1.132, pp. 1.132-5 and 1.132-6 (State's Exh. 97), recommends:

> Relatively thin zones of weak or unstable soils may be contained within more competent materials and may affect the engineering characteristics or behavior of the soil or rock. Continuous sampling in subsequent borings is needed through these suspect zones. Where it is not possible to obtain continuous samples in a single boring, samples may be obtained from adjacent closely spaced borings in the immediate vicinity and may be used as representative of the material in the omitted depth intervals. Such a set of borings should be considered equivalent to one principal boring.

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[7] State's Exhibit 99; graphs prepared by myself using data from ConeTec, Inc's Cone Penetration Testing Results of Soils at the PFS Facility, G(PO30), Rev. 1 (May 1999), as explained in my deposition of November 16, 2000 at 241-243 and Exh. 59 (UT-45647-45654) to that deposition, transcript excerpt included in Exh. 99.

Q. 21: How did PFS conduct its sampling?

A. 21: The Applicant's sampling strategy for the pad emplacement area consisted of drilling using a regular grid pattern and sampling at 5-foot depth intervals (i.e., taking a sample every 5 feet in the borehole).

Q. 22: Do you see any problems arising from the way in which PFS conducted its sampling?

A. 22: Yes. The upper Lake Bonneville sediments have not been continuously sampled and characterized with depth. This incomplete characterization adds additional uncertainty to the Applicant's estimate of the shear strength of this important layer and subsequently to the factors of safety calculated for seismic sliding and bearing capacity of the pads.

Q. 23: Do you see any weaknesses in PFS's sampling program?

A. 23: The most egregious weakness of the Applicant's sampling program is the extreme undersampling that has been performed of the upper Lake Bonneville sediments. The Applicant has calculated the sliding resistance of the pads based on one set of direct shear tests obtained from borehole C-2 from a depth interval of 5.7 to 6 feet (Calc. G(B)04-9, p. 11). This set of tests results in a sliding shear strength value of 2.1 ksf for the static loading condition under the pads (Calc. G(B)04-9, p. 32). State's Exh. 95.

Q. 24: Do you consider one set of direct shear tests to be representative of the upper Lake Bonneville sediments?

A. 25: No. The Applicant has not demonstrated that this single datum is representative of the upper Lake Bonneville sediments for the 51-acre pad emplacement area. The volume of the upper Lake Bonneville sediments in the pad emplacement area is approximately 7 feet x 51 acres x 43,560 ft^2 or about 15,550,920 ft^3. The Applicant has not demonstrated how this one set of direct shear tests is applicable to such a large volume of soil.

Q. 25: What are the consequences of undersampling?

A. 25: Such extreme undersampling of the pad emplacement area may be subject to severe bias and could potentially lead to overestimation of shear strength capacity available to resist earthquake forces.

Further, the seismic stability calculations have not accounted for the potential horizontal variation of shear strength properties of the upper Lake Bonneville sediments across the pad emplacement area.

Q. 26: Please give a specific example to illustrate the variation of shear strength properties?

A. 26: One example is the cone penetrometer tests that have been performed in the pad emplacement area (State's Exh. 99, Figures 1-8). These data suggest that the penetration resistance (i.e., tip stress values, Qt) vary by a factor of about 2 across the pad emplacement area in the depth interval between 3 and 10 feet below the ground surface. The Applicant has made no statistical assessment of this horizontal variation and how this variation may impact the single shear strength value of 2.1 ksf used in the seismic sliding stability calculations for the pad emplacement area.

Studies have shown that the shear strength of a given soil type is directly related to the CPT penetration resistance (EPRI[8], p. 4-55). Thus, it is reasonable to believe that the shear strength of the upper Lake Bonneville sediments may vary by a factor of 2 in the pad emplacement area. Clearly, it is possible that many areas may have undrained shear strength values somewhat below the 2.1 ksf value used in design and some areas may have undrained shear strength values considerably below the 2.1 ksf value. If the 2.1 ksf value is assumed to be an average value for this layer, then based on the variability suggested by the CPT logs, it is possible to have undrained direct shear strength values ranging from about 1.4 ksf to 2.8 ksf.

Q. 27: Is potential variability of shear strength in the upper Lake Bonneville sediments important and if so why?

A. 27: The potential variability of shear strength in the upper Lake Bonneville sediments is of critical importance because it is possible to have an unsafe sliding condition if the undrained shear strength value changes approximately 15 percent below the assumed design value of 2.1 ksf. Thus, using the Applicant's assumed factor of safety against sliding of 1.27 for the pads, an unacceptable factor of safety against sliding will be obtained if the undrained shear strength is 1.82 ksf, or less. This is certainly possible, considering the potential range in shear strength values suggested by the CPT data taken from the pad emplacement area.

Q. 28: Is there other evidence to suggest that PFS's shear strength value for the pad area may be unconservative?

A. 28: Yes. Evidence that the design shear strength value of 2.1 ksf used by the Applicant may be unconservative for the pad emplacement area is found by examining the

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[8] Electric Power Research Institute (1990). "Manual on Estimating Soil Properties for Design," EPRI Report No. EL-6800, Research Project 1493-6; excerpts included as State's Exhibit 100.

direct shear test results for the Canister Transfer Building for the upper Lake Bonneville sediments. Only two sets of direct shear tests were performed for the CTB footprint area in the upper Lake Bonneville sediments. One of these sets has an undrained shear stress of about 1.75 ksf at a normal stress of 2.0 ksf (SWEC Calc. G(B)05-2[9], p. 35). I would note that the normal stress of 2.0 ksf is the approximate vertical static stress at the base of the cask storage pads. Thus, the CTB direct shear test results for the upper Lake Bonneville sediments suggests that shear strength values below the critical value of 1.82 ksf are certainly possible.

Q. 29: **Has PFS used potentially unconservative estimates of the undrained shear strength for the CTB?**

A. 29: Yes. PFS has used potentially unconservative estimates of the undrained shear strength in the dynamic bearing capacity calculations for the Canister Transfer Building and has used data that are not located near this building. A weighted average for the undrained shear strength of 3.18 ksf was used for the upper 28 feet of the profile based on a unconsolidated undrained ("UU") test of 2.2 ksf from boring # 4 and C-2 and adjusting this value by 1.64 for the deeper soils from 12 to 28 feet (Calc. G(B)13-6, p. 9; State's Exh. 96). However, boring # 4 and C-2 are not within the footprint of the Canister Transfer Building (SAR Figure 2.6-19; State's Exh. 94). Both are located more than 1000 feet away from the building. CPT Sounding 37 was used to adjust for the undrained shear strength for the deeper layer beneath the CTB. This sounding is located within the footprint of the Canister Transfer Building; however it is more than a 1,000 feet from the location of the boreholes for the UU testing. This separation distance is too great and makes the adjustment factor of 1.64 applied to the UU data potentially erroneous.

Q. 30: **Are there tests that, in your opinion, the Applicant should conduct?**

A. 30: Yes. Let me explain. Earthquake loadings are cyclic in nature with several reversals in the direction of loading during a large earthquake. However, the Applicant has used the peak undrained strength determined from a monotonic test (*i.e.*, one directional loading without cycling) to represent the soil's shear resistance for the design of the pads and CTB foundations. This is not consistent with state-of-practice. It is important to perform cyclic laboratory testing on undisturbed soil samples to ensure there is no significant loss or degradation of shear strength due to cycling. These cyclic tests are commonly performed in

[9] SWEC Calc. No. G(B)05, Rev. 2, *Document Bases for Geotechnical Parameters Provided in Geotechnical Design Criteria*, June 15, 2000, excerpts included as State's Exhibit 101.

a strain-controlled manner at various strain levels. Makdisi and Seed (1978)[10] report that significant shear strains can develop in the laboratory samples when the cyclic loading approaches about 80 to 90 percent of the peak monotonic shear strength.

The Applicant has not performed cyclic triaxial testing of the upper Lake Bonneville sediments to ensure that there is no significant degradation of shear strength at shear strain levels caused by the design basis earthquake. Thus, the Applicant's testing approach is potentially unconservative.

Q. 31: Is there any way in which PFS could compensate for its failure to conduct cyclic testing?

A. 31: Yes. When cyclic testing is absent, it is common practice to reduce the monotonic peak shear strength by about 10 to 20 percent to conservatively account for any potential strain softening of the soil due to cycling (Makdisi and Seed, 1978).

Q. 32: Do you have other concerns about PFS's testing of the upper Lake Bonneville sediments?

A. 32: Yes. The upper Lake Bonneville sediments have anisotropic shear strength properties. This means that the shear strength is a function of the direction of shear (State's Exhibit 103, Figure 9[11]). The upper Lake Bonneville sediments are strongest in triaxial compression ("TC") and weakest in triaxial extension ("TE"). They have intermediate shear strength values when tested in direct simple shear ("DSS"). Previous studies performed on Lake Bonneville sediments have shown that the undrained shear strength in triaxial extension is approximately 60 percent of the undrained shear strength in triaxial compression (Saye and Ladd, 2000, p. 11)[12].

The Applicant has primarily used triaxial compression tests to calculate the soil's resistance to bearing capacity failure. No consideration has been given to performing triaxial extension tests to determine the degree of anistropy of the foundation soils. If significant anistropy is present, then the use of triaxial compression tests is unconservative and overestimates the average shear resistance along the potential failure plane (State's Exh. 103,

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[10] Makdisi, F. I., and Seed, H. B. (1978), "Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformation," American Society of Engineers Journal of Geotechnical Engineering Division, pp. 849 - 867, July 1978; State's Exhibit 102.

[11] Figure 4-6 from EPRI 1990, with my additional explanatory caption.

[12] Saye, S. and Ladd, C. C. (2000). "Design and Performance of the Foundation Stabilization Treatments for the Reconstruction of Interstate 15 in Salt Lake City," URS Corporation Speciality Conference, June 24, 2000; excerpts included as State's Exhibit 104.

Figure 9 e). This issue has the greatest significance in analyzing the bearing capacity of the storage pads, due to their relatively narrow width (30 feet) and the small margin (i.e., 5 percent) against seismic bearing capacity failure estimated by the Applicant.

Q. 33: Has PFS adequately analyzed the stress-strain behavior of the native foundation soils under a range of cyclic strains imposed by the design basis earthquake?

A. 33: No. The Applicant has relied on simple pseudo-static analyses to calculate the factor of safety against sliding and bearing capacity of the foundations for the pads and CTB. Such simple analyses do not consider the magnitude or the cyclic strains imposed by the earthquake and the effects that these cyclic strains have on the soil's shear strength properties and potential interaction with adjacent structures. For the case of relatively heavy structures (e.g., casks and CTB) resting on a deformable soil such as the Lake Bonneville deposits, it is more appropriate to perform soil-structure interaction analysis to estimate the dynamic stresses and strains imposed on the soil, soil cement and cement-treated soil.

Q. 34: Based on your testimony, do you have an opinion about PFS's Soil Characterization, and if so, what is it?

A. 34: The considerations discussed in this testimony have led me to conclude that the Applicant has not demonstrated that the upper Lake Bonneville sediments have adequate shear strength to resist the earthquake loadings imposed by the overlying foundations and structures for the pads and CTB. I have concluded that the Applicant has not demonstrated acceptable factors of safety against seismic sliding and bearing capacity failure and has not met the requirement of 10 CFR § 72.102(d).

Q. 35: Does this complete your testimony on soil characterization?

A. 35: Yes.

Areas of Research

Geotechnical Earthquake Engineering
Ground Response Modeling
Geotechnical Instrumentation
Site Characterization
Behavior of Soft Soils
Risk Assessment
Hazard Mapping

Areas of Expertise

Geotechnical Engineering
Earthquake Engineering
Transportation Engineering
Geoenvironmental Engineering
Applied Statistics
Project Management

Education

Ph.D., Civil Engineering (geotechnical emphasis), Brigham Young University, 1992.

B.S., Geology, Brigham Young University, 1983.

Professional History

Assistant Professor, Civil and Environmental Engineering Department, University of Utah, 2000-current.

Adjunct Assistant Professor, Brigham Young University, Department of Civil and Environmental Engineering, Brigham Young University, 2001.

Instructor, Civil and Environmental Engineering Department, University of Utah,

Utah Dept. of Transportation, Research Project Manager, Research Division, 1998 - 2000.

Woodward-Clyde Consultants, Project Engineer, 1996-1998.

Westinghouse Savannah River Company, Senior Engineer, 1991-1995.

Professional Experience

- **Assistant Professor, Civil and Environmental Engineering** - Teaching of graduate and undergraduate courses in geotechnical engineering and performing research.
- **Research Project Manager, I-15 Reconstruction Testbed** - UDOT Project manager for I-15 research involving construction and instrumentation of innovative embankment systems, foundation treatments and ground modification; long-term settlement monitoring and performance of embankments, mechanically stabilized earth walls, geofoam fills, etc.; response of pile and geopier foundation systems to lateral and uplift loads; carbon fiber retrofitting and non-destructive testing of bridges.
- **I-15 Design-Build Project Geotechnical Designer** - Design engineer for Woodward-Clyde Consultants responsible for geotechnical design from 800 South to 2100 South of I-15 in Salt Lake City, Utah. Design included foundation treatments, ground modification, slope stability, settlement considerations, geofoam fills, liquefaction assessments, and seismic modeling of embankment and MSE wall systems.
- **Value Engineering and Design Team** - Geotechnical member of the Value Engineering and Concept Design Team for the University Parkway Interchange (1300 South) at I-15, Orem, Utah.
- **Private Fuels Storage Facility** - Geotechnical expert witness for the State of Utah in the proceedings before the Nuclear Regulatory Commission for thePrivate Fuel Storage, LLC proposed interim high-level radioactive waste storage facility. Lead reviewer of the safety analysis report (SAR) and supporting calculations for geotechnical investigations, Skull Valley, Utah.
- **Kennecott Utah Copper Tailing Impoundment Modernization Project** - Performed steady state and transient seepage analyses for dewatering system for the upgrade and expansion of Kennecott's tailings impoundment, Magna, Utah.

State's
Exhibit 92

Brigham Young University, Research Assistant, 1988-1991.

Utah Department of Transportation, Preconstruction Materials Engineer, 1987-1988.

Utah Department of Transportation, Construction Technician, 1984-1987.

Geokinetics In-Situ Oil Shale Development, Retort Engineer and Technical Writer, 1984.

Awards and Recognitions

BYU Presidential Scholar (University Scholarship).

Alvin Barrett Scholar (Geology Department).

Civil Engineering Departmental Scholar.

BYU Scientific Research Society (Sigma-Chi) Recipient, Outstanding Ph.D. Dissertation, 1992.

Total Quality Achievement Award, Environmental Restoration Department, Westinghouse Savannah River Company, 1992, 1993.

Finalist for outstanding paper, ASCE Journal Geotechnical Engineering, 1995.

Vice President's Award, Westinghouse Engineering and Construction Services Division, 1995.

Excellence in Research Award, Utah Dept. of Transportation, 1999, 2000.

Registrations

Professional Engineer: Utah.

Affiliations

American Society of Civil Engineers.

Transportation Research Board.

National Council of Examiners for Engineering and Surveying.

American Society of Engineering Educators?

Boy Scouts of America.

- **Wasatch County Water Efficient Project** - Performed geologic and geotechnical assessments of canal stability and pump station locations, Heber Valley, Utah.
- **Bear River Pipeline** - Performed geologic and geotechnical assessments of pipeline route alternatives for the Salt Lake Water Conservancy District, Weber, Davis and Salt Lake Counties, Utah.
- **Cainville Dam Investigation** - Project Engineer responsible for preliminary geologic and geotechnical assessments of foundation conditions at this proposed dam site. Performed drilling of abutment areas, pump testing, and seepage assessments, Wayne County, Utah.
- **DMAD and Gunnison Bend Dam Investigations** - Performed geotechnical investigations and assessments to determine the piping potential and seismic stability of these embankment dams for the State of Utah, Dam Safety Program, Delta, Utah.
- **Seismic Retrofit of Salt Lake City Waste Water Treatment Plant** - Lead geotechnical design engineer and field oversight engineer of jet grouting operations to stabilize potentially liquefiable soils under an effluent pump station, North Salt Lake City, Utah.
- **Hurricane Bridge Foundation Investigation** - Performed geologic and bridge foundation investigations and analyses for UDOT, Hurricane Bridge Crossing, Hurricane, Utah.
- **ITP/H-Area Tank Farm Geotechnical Investigation and Seismic Qualification, Department of Energy, Savannah River Site** - Westinghouse's principal geotechnical investigator on a multi-disciplinary team overseeing the seismic qualification of the ITP/H-Area high-level radioactive waste storage tank farm. This project included extensive subsurface investigations, strong ground motion modeling, probabilistic liquefaction hazard evaluations, dynamic settlement and slope stability calculations, and risk assessment.
- **Review Team for the Seismic Design of the Defense Waste Processing Facility, Department of Energy Savannah River Site** -

Committees and Panels

Chairman of Utah Strong Motion Advisory Committee, 2001-current.

Member of Transportation Research Board, Committee on Soils and Rock Instrumentation, 2000-current.

Member of Utah Seismic Safety Commission Lifelines Subcommittee, 1998-current.

Program Committee Chair, EPS Geofoam 2001 3rd International Conference, Salt Lake City, December 10-12, 2001.

Member of Organizing Committee, Geologic Hazards in Utah, Salt Lake City, Utah, April 12-13, 2001.

Member of FEMA Project Impact and Salt Lake City Seismic Hazard Ordinance Committee, 2000.

Member of Organizing Committee, 34th Annual Symposium on Engineering Geology and Geotechnical Engineering, Logan, Utah, April, 1999.

Member of Organizing Committee, Environmental Geotechnology, ASCE, Salt Lake City, Utah, March, 1997.

Member of Municipal Landfill Site Selection Committee, Columbia County, Georgia, 1993.

Training and Certifications

OSHA 1910.120 Health and Safety Training for Hazardous Waste Operations and Emergency Response.

Department of Energy, Radiation Worker Training, Westinghouse Savannah River Company.

U.S. Department of Labor Mine Safety and Health Administration (MSHA) Underground Mining Training.

Peer Reviewed Publications and Reports

Bartlett S. F., and Farnsworth, C. "Performance of Lime Cement Stabilized Soils for the I-15 Reconstruction Project, Salt Lake City, Utah, "Transportation Research Board Annual Meeting, Jan. 2002, Washington, D.C. (in press).

Bartlett S. F., Farnsworth, C., Negussey, D., and Stuedlein, A. W., 2001, "Instrumentation and Long-

Westinghouse's principal geotechnical investigator reviewing the Safety Analysis Report (SAR) for the seismic qualification and start-up of this high-level radioactive waste vitrification and storage facility, Savannah River Site, Aiken, South Carolina.

- **Department of Energy Savannah River Site Hazardous Waste Landfill Closure** -Project manager and lead design engineer for the RCRA Facility Investigation and closure of a 51-acre hazardous waste landfill. Also, oversaw the preparation of CERCLA feasibility study for the same closure, Savannah River Site, Aiken South Carolina.
- **RCRA/CERCLA Investigations** - Project Manager for hazardous waste investigations at the Bingham Pump Outage Pits, Burma Road Rubble Pits, and H-Area Retention Ponds, Savannah River Site, Aiken, South Carolina.
- **UDOT Region 2 Preconstruction Materials Engineer** - Performed material testing and pavement design for highway alignment and urban interchanges in West Valley City and the I-215 interchange at California Avenue. Evaluated compaction and quality of subgrade for east-side I-215 between 2700 South and 4500 South. Conducted geologic investigations on new and existing highway alignments in Salt Lake and Wasatch Counties, located fill and gravel sources for construction. Instrumented and monitored I-215 fill slopes for settlement and slope stability.
- **Construction/Survey Technician** - Survey of highway projects and construction inspection. Development of construction project accounting system for UDOT.
- **Retort Engineer** - Monitored process control of underground retorting of oil shale for Geokinetics under Syn-Fuels research contracts for the Department of Energy, Vernal, Utah.

Research and Educational Experience

- **Development of Design ResponseSpectra for Soft Soil Site from Probabilistic Based Bedrock Specta** - Principal Investigator, Utah

Term Monitoring of Geofoam Embankments, I-15 Reconstruction Project, Salt Lake City, Utah," Proceedings of EPS 2001, (in press).

Youd, T.L., Hansen, C.M., Bartlett S.F., 2001, "Revised MLR Equations for Prediction of Lateral Spread Displacement," Journal of Geotechnical (in press).

Bartlett, S.F., Monley, G., Soderborg, A., Palmer, A., 2001, "Instrumentation and Construction Performance Monitoring for the I-15 Reconstruction Project, Salt Lake City, Utah," Transportation Research Board Annual Meeting, Jan. 2001, Washington, D.C.

Bartlett, S. F., Negussey, D., Kimball, M., 2000, "Design and Use of Geofoam on the I-15 Reconstruction Project," Transportation Research Board Annual Meeting, Jan. 2000, Washington, D.C.

Bartlett, S. F. and Youd, T. L., April 1995, "Empirical Prediction of Liquefaction-Induced Lateral Spread," Journal of Geotechnical Engineering, ASCE.

Bartlett, S. F., 1992, "Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads," Ph.D. dissertation and report published by National Earthquake Engineering Research Center, NCEER Report #92-0021.

Bartlett, S. F. and Youd, T. L., 1992, "Case Histories of Lateral Spreads from the 1964 Alaska Earthquake," NCEER Report #92-0002.

Other Publications and Reports

Saye, S. R., Esrig, M. I., Williams, J. L., Pilz J., Bartlett S.F., "Lime Cement Columns for the Reconstruction of Interstate 15 in Salt Lake City, Utah." ASCE Geo-Odessey, Blacksburg, VA., June 10 - 13th, 2001.

Bartlett, S. F., 1999, "Research Initiatives for Monitoring Long Term Performance of I-15 Embankments, Salt Lake City, Utah," 34th Annual Symposium on Engineering Geology and Geotechnical Engineering, Logan, Utah, April, 1999.

Youd, T. L., Hansen C. M., Bartlett, S. F., 1999, "Improved MLR Model for Predicting Lateral Spread Displacement," 7th US-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, Seattle, Washington, August, 1999.

Department of Transportation (2000-2001).

- **I-15 Long Term Monitoring of Embankments and Innovative Foundation Treatments** - Principal Investigator, Utah Department of Transportation (1998 - 2008).
- **Deformation and Modeling of MSE Wall Behavior** - Co-Principal Investigator, Utah Department of Transportation and Utah State University (1999-2000).
- **Evaluation of Properties and Long-Term Performance of Geofoam Fills** - Co-Principal Investigator, Utah Department of Transportation and Syracuse University (1998-2000).
- **Geostatistical Assessment of In-Situ and Engineering Properties at H-Tank Farm** - Co-Principal Investigator, Westinghouse Savannah River Company and Georgia Institute of Technology (1994 - 1995).
- **Evaluation of Geopiers and Pile Foundation to Lateral and Uplift Loads** - Project Manager, Utah Department of Transportation, University of Utah, and Brigham Young University (1999).
- **Design, Application, and Use of Carbon-Fiber Composites in Bridge Repair and Seismic Retrofitting** - Project Manager, Utah Department of Transportation and University of Utah (1998-2000).
- **Use of Forced Vibration Testing to Assess Bridge Damage** - Project Manager, Utah Department of Transportation and Utah State University (1998).
- **Identification and Ranking of UDOT Lifelines** - Project Manager, Utah Department of Transportation (1998 - 2000).
- **Wick Drain Performance** - Project Manager, Utah Department Transportation (1998 - 1999).
- **Assessment of Dynamic Soil Properties for the Savannah River Site** - Geotechnical Reviewer, Westinghouse Savannah River Company and University of Texas at Austin.
- **Research Assistant, Brigham Young University,** "Empirical Prediction of Liquefaction-Induced Lateral Spread," U.S. Army Corps of Engineers and National Center

Simon, D. B., Shlemon, R .J., and Bartlett, S.F., 1999, "Holocene Ground Failure in Downtown Salt Lake City, Utah," Geological Society of America, Cordilleran Section, Vol. 31, Number 6, Berkeley, California, May 1999.

WSRC, 1995, "In-Tank Precipitation Facility (ITP) and H-Tank Farm (HTF) Geotechnical Report," Report No. WSRC-TR-95-0057, Rev. 0, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., 1995, "Probabilistic Liquefaction Settlement Evaluation for the In-Tank Precipitation Facility (ITP)," Report No. C-CLC-H-00815, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., 1995, "Geotechnical Seismic Assessment Report for the Defense Waste Processing Facility (DWPF)," Report No. SRC-TR-95-0072, Westinghouse Savannah River Company, Aiken, S.C.

Rouhani, S., Lin, Y. P., and Bartlett, S. F., 1995, "H-Area/ITP Geostatistical Assessment of In-Situ and Engineering Properties," Final Technical Report, ERDA Project No. 93044, Site Geotechnical Services, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., 1994, "Determination of Soft Zones and Consolidation Properties for the Santee Formation," Report No. K-CLC-H-00058, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F. and Youd, T. L., 1993, "Prediction of Liquefaction-Induced Ground Displacement Near Bridges," Proceedings from the U.S. National Earthquake Conference, Memphis, Tenn., May, 1993.

Bartlett, S. F., 1993, "RCRA Facility Investigation / CERCLA Remedial Investigation for the Burma Road Rubble Pit," Environmental Restoration Department, Westinghouse Savannah River Company, Aiken, S.C.

Bartlett, S. F., McMullin, S. R., and Serrato, M., 1993, "State of the Art Design: A Closure System for the Largest Hazardous Waste Landfill at the Savannah River Site," Proceedings of Waste Management '93 Symposium.

Bartlett, S. F. and Youd, T. L., 1992, "Empirical Prediction of Lateral Spread Displacement," Proceedings of 4th Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, May, 1992.

Bartlett, S. F. and Youd, T. L., 1990, "Evaluation of

for Earthquake Engineering Research (1988-1991).

- **Thesis Committee Member**, Kiehl, S.J. "Distribution of Ground Displacements and Strains Induced by Lateral Spread During the 1964 Niigata Earthquake, Brigham Young University (1996).
- **Thesis Committee Member,** Hansen C. M, "Improved MLR Model for Predicting Lateral Spread Displacement, Brigham Young University (1999).

Teaching Experience

- **Assistant Professor**, University of Utah, Fall 2000 to current.
- **Teaching Assistant**, Earthquake Engineering, Brigham Young University, Winter Semester, 1989.
- **Teaching Assistant**, Soil Mechanics, Brigham Young University, Fall Semester, 1989.
- **Teaching Assistant**, Field and Laboratory Testing of Soil, Brigham Young University, Spring Term, 1989.
- **Missionary** - Church of Jesus Christ of Latter-Day Saints, Catania, Italy, 1979 - 1981.

Graduate Courses Taught

- CVEEN 7330 Geotechnical Earthquake Engineering (1 time)
- CVEEN 7340 Advanced Geotechnical Testing

Undergraduate Courses Taught

- CVEEN 3310 Geotechnical Engineering I (2 times)
- CVEEN 3320 Geotechnical Engineering II (2 times)

Papers Reviews

Ground Failure Displacement Associated with Soil Liquefaction: Compilation of Case Histories," Miscellaneous Paper S-73-1, U.S. Army Corps of Engineers.

Invited Lectures

"UDOT Guidance for Developing Design Response Spectra for Soft Soils," Geologic Hazards in Utah, Sponsored by AEG and ASCE, Salt Lake City, Utah, April 12 -13, 2001.

"Instrumentation and Research of Geofoam Embankments for the I-15 Reconstruction," Huntsman Chemical Geofoam Seminar"May 16th, 2000, Salt Lake City, Utah

"Design of Geofoam Embankment for the I-15 Reconstruction," Conference on Application and Design of Expanded Polystrene, Sponsored by Taiwan Area National Expressway Engineering Bureau and China Engineering Consultants, Inc., March 3rd, 2000, Taipei, Taiwan.

"Issues Related to the Seismic Design of I-15 Reconstruction Project - A Geotechnical Perspective," Association of Engineering Geologist 42nd Annual Meetings, Sept. 28, 1999, Salt Lake City, Utah.

"Assessment of the Hazard Potential for the East Side of I-80," Conference on the Sesimic Retrofit of Utah's Highway Bridges, sponsored by the Utah Department of Transportation, January 20-22, 1999. Salt Lake City, Utah.

"Geofoam Design, Construction and Research on the I-15 Corridor Reconstruction Project," Annual Meeting of the Society of the Plastics Industry, Inc., April 23 and 24, 1998, New Orleans, La.

NUREG-0800
(Formerly NUREG-75/087)

U.S. NUCLEAR REGULATORY COMMISSION

STANDARD REVIEW PLAN

OFFICE OF NUCLEAR REACTOR REGULATION

3.8.5 FOUNDATIONS

REVIEW RESPONSIBILITIES

Primary - Structural Engineering Branch (SEB)

Secondary - None

I. AREAS OF REVIEW

The following areas related to the foundations of all seismic Category I structures are reviewed.

1. Description of the Foundations

   The descriptive information, including plans and sections of each foundation, is reviewed to establish that sufficient information is provided to define the primary structural aspects and elements relied upon to perform the foundation function. Also reviewed is the relationship between adjacent foundations, including the methods of separation provided where such separation is used to minimize seismic interaction between the buildings. In particular, the type of foundation is identified and its structural characteristics are examined. Among the various types of foundations reviewed are mat-foundations and footings, including individual column footings, combined footings supporting more than one column, and wall footings supporting bearing walls.

   Other types of foundations that may also be used are pile foundations, drilled caissons, caissons for water front structures, such as a pumphouse, and rock anchor systems. These types of foundation are reviewed on a case-by-case basis.

   The major plant Category I foundations that are reviewed, together with the descriptive information, are listed below:

State's
Exhibit 93

d. For the containment foundation, the design and analysis procedures referenced in subsection II.4 of SRP Section 3.8.1 are acceptable.

e. The design report is found acceptable if it satisfies the guidelines contained in Appendix C to SRP Section 3.8.4.

f. The structural audit is conducted as described in Appendix B to SRP Section 3.8.4.

For determining the overturning moment due to an earthquake, the three components of the earthquake should be combined in accordance with methods described in SRP Section 3.7.2. Computer programs are acceptable if the validation provided is found in accordance with procedures delineated in subsection II.4.e of SRP Section 3.8.1.

5. Structural Acceptance Criteria

For each of the loading combinations referenced in subsection II.3 of this SRP Section, the allowable limits which constitute the acceptance criteria are referenced in subsection II.5 of SRP Section 3.8.1 for the containment foundation, and are listed in subsection II.5 of SRP Section 3.8.4 for all other foundations. In addition, for the five additional load combinations delineated in subsection II.3 of this SRP section, the factors of safety against overturning, sliding and floatation are acceptable if found in accordance with the following:

	Minimum Factors of Safety		
For Combination	Overturning	Sliding	Floatation
a. ------------------------	1.5	1.5	--
b. ------------------------	1.5	1.5	--
c. ------------------------	1.1	1.1	--
d. ------------------------	1.1	1.1	--
e. ------------------------	--	--	1.1

6. Materials, Quality Control, and Special Construction Techniques

For the containment foundation, the acceptance criteria for materials, quality control, and any special construction techniques are referenced in subsection II.6 of SRP Section 3.8.1. For all other seismic Category I foundations, the acceptance criteria are similar to those referenced in subsection II.6 of SRP Section 3.8.4.

7. Testing and Inservice Surveillance Requirements

At present there are no special testing or in-service surveillance requirements for seismic Category I foundations other than those required for the containment foundation, which are covered in subsection II.7 of SRP Section 3.8.1. However, should some requirements become necessary for special foundations, they will be reviewed on a case-by-case basis.

III. REVIEW PROCEDURES

The reviewer selects and emphasizes material from the review procedures described below, as may be appropriate for a particular case.

2.6.1.12 Stability of Foundations for Structures and Embankments

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

The minimum factor of safety against a bearing capacity failure due to static loads (dead load plus maximum live loads) is 3.0.

In accordance with the requirements of NUREG-75/087, Section 3.8.5, "Foundations," Section II.5, "Structural Acceptance Criteria," the recommended minimum factor of safety against overturning or sliding failure from static loads (dead load plus maximum live loads) is 1.5 and due to static loads plus loads from extreme environmental conditions, such as the design basis ground motion, is 1.1. In addition, it is recommended that a factor of safety of 1.1 be used to design footings against a bearing capacity failure from static loads plus loads due to the design basis ground motion.

If the factor of safety against sliding is less than 1 due to the design basis ground motion, additional analyses of the displacements the structure may experience are performed using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes to demonstrate that such displacements, if they did occur, would not have an adverse impact on the performance of the Important-to-Safety structures.

Recommended design earth pressure distributions are presented in Figure 2.6-7. Lateral earth pressures for determining driving forces shall be based on K_o, the at-rest earth pressure coefficient. These can be reduced to "active" earth pressures if the yield ratio exceeds 0.1%, where yield ratio, S/H, is defined as shown for the active case in Figure 2.6-8. In determining "passive" pressures resisting lateral movement, assume

State's
Exhibit 94

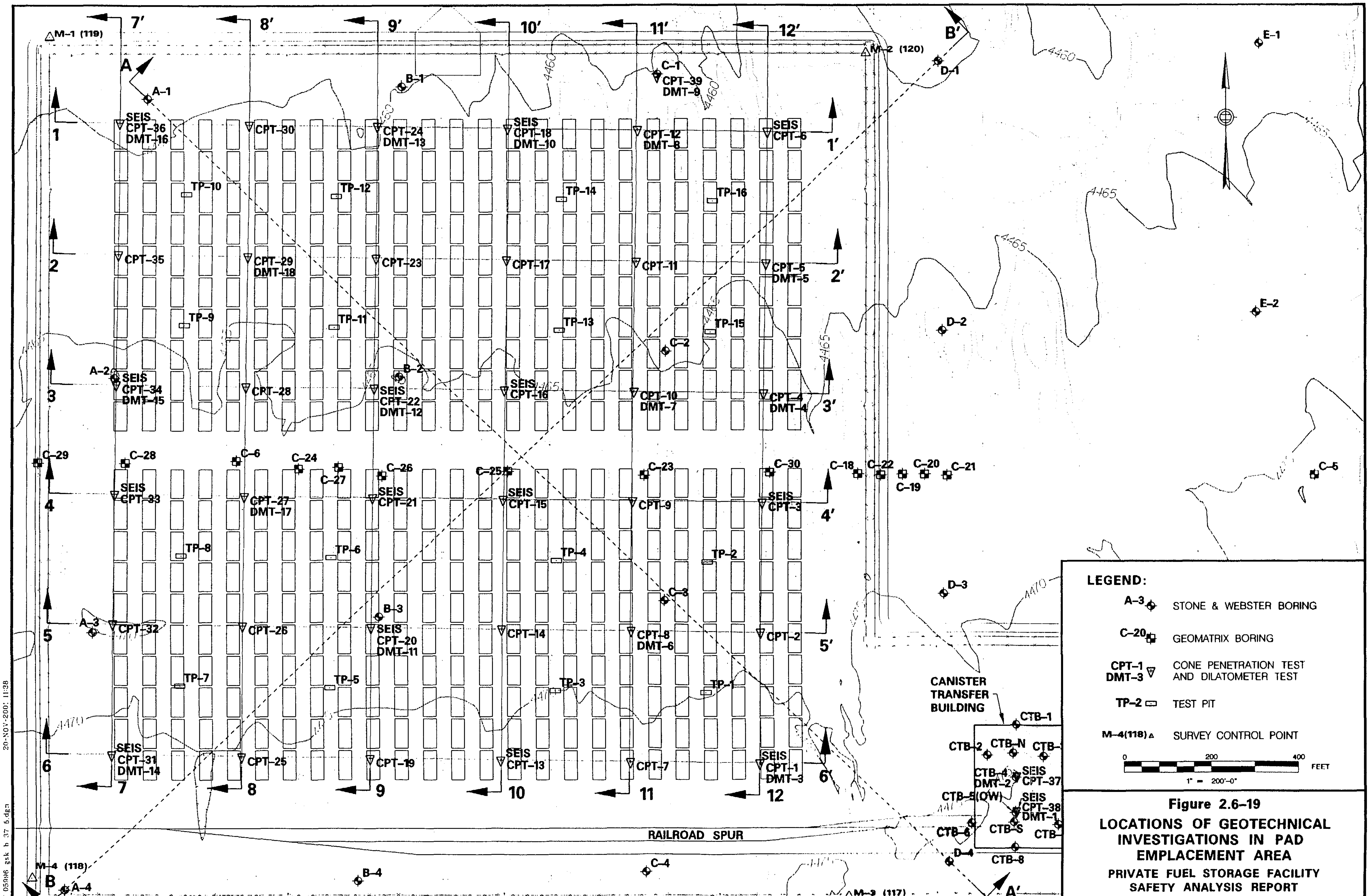

Figure 2.6-19
LOCATIONS OF GEOTECHNICAL INVESTIGATIONS IN PAD EMPLACEMENT AREA
PRIVATE FUEL STORAGE FACILITY SAFETY ANALYSIS REPORT

Revision 22

STONE & WEBSTER, INC.

5010.64

CALCULATION SHEET

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

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

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

APPROVALS - SIGNATURE & DATE			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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STONE & WEBSTER, INC.

CALCULATION SHEET

5010.64

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

CALCULATION IDENTIFICATION NUMBER

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

APPROVALS - SIGNATURE & DATE

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

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	FIRE FILE - Denver	☑			
Geotechnical	PJTrudeau – Stoughton/3	☑			

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

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

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

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

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

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

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.

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

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

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

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

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

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

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

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

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

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

The factor of safety against sliding is calculated as follows:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Allowable Bearing Capacity of Cask Storage Pads

PSHA 2,000-Yr Earthquake: Case II

Based on Inertial Forces Combined: 100 % N-S, 0 % Vert, 100 % E-W

Soil Properties:	$c =$	**2,200**	Cohesion (psf)	Footing Dimensions:		
	$\phi =$	**0.0**	Friction Angle (degrees)	B =	**30.0**	Width - ft (E-W)
	$\gamma =$	**80**	Unit weight of soil (pcf)	L =	**67.0**	Length - ft (N-S)
	$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)			
Foundation Properties:	$B' =$	**15.6**	Effective Ftg Width - ft (E-W)	L' =	**52.6**	Length - ft (N-S)
	$D_f =$	**3.0**	Depth of Footing (ft)			
					0.711	$g = a_H$
	$FS =$	**1.1**	Factor of Safety required for $q_{allowable}$		**0.695**	$g = a_V$
	$F_{V\ Static} =$	**3,757**	k & $EQ_V =$ 0 k →		**3,757**	k for F_V
	$EQ_{H\ E\text{-}W} =$	**2,671**	k & $EQ_{H\ N\text{-}S} =$ 2,671 k →		**3,777**	k for F_H

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

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

$N_c = (N_q - 1)\ \cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\ (N_q + 1)\ \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.06	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\ (B/L)$	=	0.88	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\ D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1 - d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\ (D_f/B)$	=	1.08	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.69	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.31	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	=	0.79 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.39	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	5,338	psf =	5,038	+	300	+	0
q_{all} =	4,850	psf = q_{ult} / FS					
q_{actual} =	4,565	psf = $(F_{v\ Static} + EQ_v) / (B' \times L')$					
FS_{actual} =	1.17	= q_{ult} / q_{actual}			> 1.1		Hence OK

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5010.64

CALCULATION SHEET

CLIENT & PROJECT				PAGE 1 OF 59 + 6 pp of ATTACHMENTS
PRIVATE FUEL STORAGE, LLC – PFSF				
CALCULATION TITLE				QA CATEGORY (✓)
STABILITY ANALYSES OF CANISTER TRANSFER BUILDING				☑ I NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ (other)
CALCULATION IDENTIFICATION NUMBER				
JOB ORDER NO.	DISCIPLINE	CURRENT CALC NO	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	**G(B)**	**13**		

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

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

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

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

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

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

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

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

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

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

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

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

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

Soil Cement Properties:

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

PFS is developing the soil-cement mix design using standard industry practice, in accordance with the criteria specified by the Portland Cement Association. This effort includes performing laboratory testing of soils obtained from the site. This on-going laboratory testing is being performed in accordance with the requirements of Engineering Services Scope of Work (ESSOW) for Laboratory Testing of Soil-Cement Mixes, ESSOW 05996.02-G010, Rev. 0. This program includes measuring gradations and Atterberg limits of samples of the near-surface soils obtained from the site. It includes testing of mixtures the state of stress existing under the Canister Transfer Building mat. Note, that the average post-peak strength reduction for normal stress of 1.5 ksf for the three direct shear tests is only 15.6% for these very high shear displacements in the direct shear tests. The maximum value of the average the post-peak strength reductions for normal stress of 1.5 ksf occurred for Sample U-3B&C in CTB-6, and it equaled 20.8%. If the results of this test were used to define the residual strength of these soils, the analyses would be performed at c = 1.5 ksf, the average of the post-peak strengths measured at the maximum shear displacements in these tests for normal stresses of 1 ksf and 2 ksf. This would result in higher factors of safety than are calculated and presented in Table 2.6-14, based on c = 1.36 ksf.

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

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

Average = 15.6

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

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REGULATORY GUIDE

OFFICE OF STANDARDS DEVELOPMENT

REGULATORY GUIDE 1.132

SITE INVESTIGATIONS FOR FOUNDATIONS OF NUCLEAR POWER PLANTS

A. INTRODUCTION

*Paragraph 100.10(c) and Appendix A, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," to 10 CFR Part 100, "Reactor Site Criteria," establish requirements for conducting site investigations to permit evaluation of the site and to provide information needed for seismic response analyses and engineering design. Requirements include the development of geologic information relevant to the stratigraphy, lithology, geologic history, and structural geology of the site and the evaluation of the engineering properties of subsurface materials.

Safety-related site characteristics are identified in detail in Regulatory Guide 1.70, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants." Regulatory Guide 4.7, "General Site Suitability Criteria for Nuclear Power Stations," discusses major site characteristics that affect site suitability.

This guide describes programs of site investigations that would normally meet the needs for evaluating the safety of the site from the standpoint of the performance of foundations and earthworks under most anticipated loading conditions, including earthquakes. It also describes site investigations required to evaluate geotechnical parameters needed for engineering analysis and design. The site investigations discussed in this guide are applicable to both land and offshore sites. This guide does not discuss detailed geologic fault investigations required under Appendix A to 10 CFR Part 100, nor does it deal with hydrologic investigations, except for groundwater measurements.

*Lines indicate substantive changes from previous issue.

This guide provides general guidance and recommendations for developing site-specific investigation programs as well as specific guidance for conducting subsurface investigations, the spacing and depth of borings, and sampling. Because the details of the actual site investigations program will be highly site dependent, the procedures described herein should be used only as guidance and should be tempered with professional judgment. Alternative and special investigative procedures that have been derived in a professional manner will be considered equally applicable for conducting foundation investigations.

Appendix A to this guide provides definitions for some of the terms used in this guide. These terms are identified in the text by an asterisk. Appendix B tabulates methods of conducting subsurface investigations, and Appendix C gives guidelines for the spacing and depth of borings for safety-related structures in regions of favorable or uniform conditions. References cited in the text and appendices are listed in Appendix D.

The Advisory Committee on Reactor Safeguards has been consulted concerning this guide and has concurred in the regulatory position.

B. DISCUSSION

1. General

Site investigations for nuclear power plants are necessary to determine the geotechnical* characteristics of a site that affect the design, performance, and safety of plants. The investigations produce the information needed to define the overall site geology to a degree that is necessary for an understanding of subsurface conditions and for identifying potential

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Exhibit 97

s. Personal communication with local inhabitants and local professionals.

Special or unusual problems such as swelling soils and shales (subject to large volume changes with changes in moisture), occurrences of gas, cavities in soluble rocks, subsidence caused by mining or pumping of water, gas, or oil from wells, and possible uplift due to pressurization from pumping of water, gas, or oil into the subsurface may require consultation with individuals, institutions, or firms having experience in the area with such problems.

The site investigation includes detailed surface studies and exploration of the immediate site area and adjacent environs. Further detailed surface exploration also may be required in areas remote to the immediate plant site to complete the geologic evaluation of the site or to conduct detailed investigations of surface faulting or other features. Surface exploration needed for the assessment of the site geology is site dependent and may be carried out with the use of any appropriate combination of geological, geophysical, or engineering techniques. Normally this includes the following:

a. Detailed mapping of topographic, hydrologic, and surface geologic features, as appropriate for the particular site conditions, with scales and contour intervals suitable for analysis and engineering design. For offshore sites, coastal sites, or sites located near lakes or rivers, this includes topography and detailed hydrographic surveys to the extent that they are needed for site evaluation and engineering design.

b. Detailed geologic interpretations of aerial photographs and other remote-sensing imagery, as appropriate for the particular site conditions, to assist in identifying rock outcrops, soil conditions, evidence of past landslides or soil liquefaction, faults, fracture traces, geologic contacts, and lineaments.

c. Detailed onsite mapping of local engineering geology and soils.

d. Mapping of surface water features such as rivers, streams, or lakes and local surface drainage channels, ponds, springs, and sinks at the site.

3. Groundwater Investigations

Knowledge of groundwater conditions, their relationship to surface waters, and variations associated with seasons or tides is needed for foundation analyses. Groundwater conditions are normally observed in borings* at the time they are made; however, for engineering applications, such data are supplemented by groundwater observations made by means of properly installed wells or piezometers* that are read at regular intervals from the time of their installation at least through the construction period. The U.S. Army Corps of Engineers' manual on groundwater and pore pressure observations in embankment dams and their foundations (Ref. 1) provides guidance on acceptable methods for the installation and maintenance of piezometer and observation well* instrumentation. Criteria for measuring groundwater conditions at a site and for assessing dewatering requirements during construction are given in regulatory position 3 of this guide. This guide does not cover groundwater monitoring needed during construction in plants that have permanent dewatering systems incorporated in their design.

4. Subsurface Investigations

a. General

The appropriate depth, layout, spacing, and sampling requirements for subsurface investigations are dictated by the foundation requirements and by the complexity of the anticipated subsurface conditions. Methods of conducting subsurface investigations are tabulated in Appendix B to this guide, and recommended guidelines for the spacing and depth of borings for safety-related structures, where favorable or uniform geologic conditions exist, are given in Appendix C.

Subsurface explorations for less critical foundations of power plants should be carried out with spacing and depth of penetration as necessary to define the general geologic and foundation conditions of the site. Subsurface investigations in areas remote from plant foundations may be needed to complete the geologic description of the site and confirm geologic and foundation conditions and should also be carefully planned.

Subsurface conditions may be considered favorable or uniform if the geologic and stratigraphic features to be defined can be correlated from one boring or sounding* location to the next with relatively smooth variations in thicknesses or properties of the geologic units. An occasional anomaly or a limited number of unexpected lateral variations may occur. Uniform conditions permit the maximum spacing of borings for adequate definition of the subsurface conditions at the site.

Occasionally, soil or rock deposits may be encountered in which the deposition patterns are so complex that only the major stratigraphic boundaries are correlatable, and material types or properties may vary within major geologic units in an apparently random manner from one boring to another. The number and distribution of borings needed for these conditions are determined by the degree of resolution needed in the definition of foundation care is necessary in interpreting results from the Standard Penetration Test in these materials. Often such data are misleading and may have to be disregarded. When sampling of these coarse soils is difficult, information that may be lost when the soil is later classified in the laboratory should be recorded in the field. This information should include observed estimates of the percentage of cobbles, boulders, and coarse material and the hardness, shape, surface coating, and degree of weathering of coarse materials.

(3) Moderately Compressible or Normally Consolidated Clay or Clayey Soils. The properties of a fine-grained soil are related to the in situ structure of the soil,* and therefore the recovery and testing of good undisturbed samples are necessary. Criteria for obtaining undisturbed samples are discussed in regulatory position 6 of this guide.

(4) Subsurface Cavities. Subsurface cavities may occur in water-soluble rocks, lavas, weakly indurated sedimentary rocks, or in other types of rocks as the result of subterranean solutioning and erosion. Cavities can also be found where mining has occurred or is in progress. Because of the wide distribution of carbonate rocks in the United States, the occurrence of features such as cavities, sinkholes, and solution-widened joint openings is common. For this reason, it is best to thoroughly investigate any site on carbonate rock for solution features to determine their influence on the performance of foundations. Because of the possibility that incomplete or inaccurate records exist on mining activities, it is equally important to investigate areas where mining has or may have occurred.

Investigations may be carried out with borings alone or in conjunction with accessible excavations, soundings, pumping tests, pressure tests, geophysical surveys, or a combination of such methods. The investigation program will depend on the details of the site geology and the foundation design. Various geophysical techniques used for detecting subsurface cavities are discussed in Reference 2.

Indications of the presence of cavities (e.g., zones of lost drilling fluid circulation, water flowing into or out of drillholes, mud fillings, poor core recovery, dropping or settling of drilling rods, anomalies in geophysical surveys, or in situ tests* that suggest voids) should be followed up with more detailed investigations. These investigations should include excavation to expose solution features or additional borings that define the limits and extent of such features.

The occurrence, distribution, and geometry of subsurface cavities are highly unpredictable, and no preconstruction exploration program can ensure that all significant subsurface voids will be fully revealed. Experience has shown that solution features may remain undetected even where the area has been investigated by a large number of borings. The fact that cavities are often filled or partially filled with residual material and debris makes it particularly difficult to detect cavities on the basis of boring data and results of fluid pressure and grout-take tests. Therefore, where a site is on solution-susceptible rock, it may sometimes be necessary to inspect the rock after stripping or excavation is complete and the rock is exposed.

(5) Materials Unsuitable for Foundations. Borings and representative sampling and testing should be completed to delineate the boundaries of unsuitable materials. These boundaries should be used to define the required excavation limits.

(6) Borrow Materials. Exploration of borrow sources requires the determination of the location and amount of borrow fill materials available. Investigations in the borrow areas should be at horizontal and vertical intervals sufficient to determine the material variability and should include adequate sampling of representative materials for laboratory testing.

Investigations of problem foundation conditions are discussed in Appendix A to Reference 3 and in Reference 4.

c. Sampling

Representative samples* of all soil and rock should be obtained for testing. In many cases, to establish physical properties it is necessary to obtain undisturbed samples that preserve the in situ structure of the soil. The recovery of undisturbed samples is discussed in Section B.6 of this guide.

Sampling of soils should include, as a minimum, recovery of samples for all principal borings at regular intervals and at changes in strata. A number of samples sufficient to permit laboratory determination of average material properties and to indicate their variability is necessary. Alternating split spoon and undisturbed samples with depth is recommended. Where sampling is not continuous, the elevations at which samples are taken should be staggered from boring to boring so as to provide continuous coverage of samples within the soil column. In supplementary borings, sampling may be confined to the zone of specific interest.

Relatively thin zones of weak or unstable soils may be contained within more competent materials and may affect the engineering characteristics or behavior of the soil or rock. Continuous sampling in subsequent borings is needed through these suspect zones. Where it is not possible to obtain continuous samples in a single boring, samples may be obtained from adjacent closely spaced borings in the immediate vicinity and may be used as representative of the material in the omitted depth intervals. Such a set of borings should be considered equivalent to one principal boring.

d. Determining the Engineering Properties of Subsurface Materials

A general discussion of the classifications of soils and rocks and methods of determining their engineering properties is included in Reference 5.

The shear strengths of foundation materials in all zones subjected to significant imposed stresses should be determined to establish whether they are adequate to support the imposed loads with an appropriate margin of safety. Similarly, it is necessary both to determine the compressibilities and swelling potentials of all materials in zones subjected to significant changes of compressive stresses and to establish that the deformations will be acceptable. In some cases, these determinations may be made by suitable in situ tests and classification tests. Other situations may require the laboratory testing of undisturbed samples. Determination of dynamic moduli and damping ratios over applicable strain ranges of soil strata is needed for earthquake response analyses. Dynamic moduli and damping may be evaluated in situ, but usual procedures provide information only for low shear strain amplitudes. Laboratory tests on undisturbed samples can provide additional modulus and damping values to cover the range of strains anticipated under earthquake loading conditions.

5. Methods and Procedures for Exploratory Drilling

In nearly every site investigation, the primary means of subsurface exploration are borings and borehole sampling. Drilling methods and procedures should be compatible with sampling requirements and the methods of sample recovery.

The top of the hole should be protected by a suitable surface casing where needed. Below ground surface, the borehole should be protected by drilling mud or casing, as necessary, to prevent caving and disturbance of materials to be sampled. The use of drilling mud is preferred to prevent disturbance when obtaining undisturbed samples of coarse-grained soils.

However, casing may be used if proper steps are taken to prevent disturbance of the soil being sampled and to prevent upward movement of soil into the casing. Washing with open-ended pipe for cleaning or advancing sample boreholes should not be permitted. Bottom-discharge bits should be used only with low-to-medium fluid pressure and with upward-deflected jets.

In addition to pertinent information normally recorded for groundwater measurements and the results of field permeability tests, all depths and amounts of water or drilling mud losses, together with depths at which circulation is recovered, should be recorded and reported on boring logs and on geological cross sections. Logs and sections should also reflect incidents of settling or dropping of drill rods; abnormally low resistance to drilling or advance of samplers, core losses, instability or heave of the side and bottom of boreholes; influx of groundwater; and any other special feature or occurrence. Details of information that should be presented on logs of subsurface investigations are given in regulatory position 2.

Depths should be measured to the nearest tenth of a foot (3 cm) and should be correlatable to the elevation datum used for the site. Elevations of points in the borehole should also be determined with an accuracy of ±0.1 ft (±3 cm). Surveys of vertical deviation should be run in all boreholes that are used for crosshole seismic tests and in all boreholes where vertical deviations are significant to the use of data obtained. After use, it is advisable to grout each borehole with cement to prevent vertical movement of groundwater through the borehole.

6. Recovery of Undisturbed Soil Samples

The best undisturbed samples are often obtained by carefully performed hand trimming of block samples in accessible excavations. However, it is normally not practical to obtain enough block samples at the requisite spacings and depths by this method alone. It is customary, where possible, to use thin-wall tube samplers in borings for the major part of the undisturbed sampling. Criteria for obtaining undisturbed tube samples are given in regulatory position 6.

The recovery of undisturbed samples of good quality is dependent on rigorous attention to details of equipment and procedures. Proper cleaning of the hole by methods that minimize disturbance of the soil is necessary before sampling. The sampler should be advanced in a manner that minimizes disturbance. For example, when using fixed-piston-type samplers, the drilling rig should be firmly anchored or the piston should be fixed to an external anchor to prevent its moving upward during the push of the sampling tube. Care should be taken to ensure that the sample is not disturbed during its removal from the borehole or in disassembling the sampler. References 6 and 7 provide descriptions of suitable procedures for obtaining undisturbed samples.

APPENDIX C

SPACING AND DEPTH OF SUBSURFACE EXPLORATIONS FOR SAFETY-RELATED[1] FOUNDATIONS

TYPE OF STRUCTURE	SPACING OF BORINGS[2] OR SOUNDINGS	MINIMUM DEPTH OF PENETRATION
General	For favorable, uniform geologic conditions, where continuity of subsurface strata is found, the recommended spacing is as indicated for the type of structure. At least one boring should be at the location of every safety-related structure. Where variable conditions are found, spacing should be smaller, as needed, to obtain a clear picture of soil or rock properties and their variability. Where cavities or other discontinuities of engineering significance may occur, the normal exploratory work should be supplemented by borings or soundings at a spacing small enough to detect such features.	The depth of borings should be determined on the basis of the type of structure and geologic conditions. All borings should be extended to a depth sufficient to define the site geology and to sample all materials that may swell during excavation, may consolidate subsequent to construction, may be unstable under earthquake loading, or whose physical properties would affect foundation behavior or stability. Where soils are very thick, the maximum required depth for engineering purposes, denoted d_{max}, may be taken as the depth at which the change in the vertical stress during or after construction for the combined foundation loading is less than 10% of the in situ effective overburden stress. It may be necessary to include in the investigation program several borings to establish the soil model for soil-structure interaction studies. These borings may be required to penetrate depths greater than those depths required for general engineering purposes. Borings should be deep enough to define and evaluate the potential for deep stability problems at the site. Generally, all borings should extend at least 30 feet (9 meters) below the lowest part of the foundation. If competent rock is encountered at lesser depths than those given, borings should penetrate to the greatest depth where discontinuities or zones of weakness or alteration can affect foundations and should penetrate at least 20 feet (6 meters) into sound rock. For weathered shale or soft rock, depths should be as for soils.

[1] As determined by the final locations of safety-related structures and facilities.
[2] Includes shafts or other accessible excavations that meet depth requirements.

APPENDIX C (Continued)

SPACING AND DEPTH OF SUBSURFACE EXPLORATIONS FOR SAFETY-RELATED[1] FOUNDATIONS

TYPE OF STRUCTURE	SPACING OF BORINGS[2] OR SOUNDINGS	MINIMUM DEPTH OF PENETRATION
Structures including buildings, retaining walls, concrete dams	Principal borings: at least one boring beneath every safety-related structure. For larger, heavier structures, such as the containment and auxiliary buildings, at least one boring per 10,000 ft^2 (900 m^2) (approximately 100-foot (30-meter) spacing). In addition, a number of borings along the periphery, at corners, and other selected locations. One boring per 100 linear feet (30 linear meters) for essentially linear structures.[3]	At least one-fourth of the principal borings and a minimum of one boring per structure to penetrate into sound rock or to a depth equal to d_{max}. Others to a depth below foundation elevation equal to the width of structure or to a depth equal to the foundation depth below the original ground surface, whichever is greater.[3]
Earth dams, dikes, levees, and embankments	Principal borings: one per 100 linear feet (30 linear meters) along axis of structure and at critical locations perpendicular to the axis to establish geological sections with groundwater conditions for analysis.[3]	Principal borings: one per 200 linear feet (60 linear meters) to d_{max}. Others should penetrate all strata whose properties would affect the performance of the foundation. For water-impounding structures, to sufficient depth to define all aquifers and zones of underseepage that could affect the performance of structures.[3]
Deep cuts,[4] canals	Principal borings: one per 200 linear feet (60 linear meters) along the alignment and at critical locations perpendicular to the alignment to establish geologic sections with groundwater conditions for analysis.[3]	Principal borings: one per 200 linear feet (60 linear meters) to penetrate into sound rock or to d_{max}. Others to a depth below the bottom elevation of excavation equal to the depth of cut or to below the lowest potential failure zone of the slope.[3] Borings should penetrate previous strata below which groundwater may influence stability.[2]
Pipelines	Principal borings: This may vary depending on how well site conditions are understood from other plant site borings. For variable conditions, one per 100 linear feet (30 linear meters) for buried pipelines; at least one boring for each footing for pipelines above ground.[5]	Principal borings: For buried pipelines, one of every three to penetrate into sound rock or to d_{max}. Others to 5 times the pipe diameters below the invert elevation. For pipelines above ground, depths as for foundation structures.[3,5]
Tunnels	Principal borings: one per 100 linear feet (30 linear meters),[3] may vary for rock tunnels, depending on rock type and characteristics, and planned exploratory shafts or adits.	Principal borings: one per 200 linear feet (60 linear meters) to penetrate into sound rock or to d_{max}. Others to 5 times the tunnel diameter below the invert elevation.[4,5]

[3]Also supplementary borings or soundings that are design dependent or necessary to define anomalies, critical conditions, etc.
[4]Includes temporary cuts that would affect ultimate site safety.
[5]Supplementary borings or soundings as necessary to define anomalies.

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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of:	Docket No. 72-22-ISFSI ASLBP No. 97-732-02-ISFSI
PRIVATE FUEL STORAGE, LLC	Deposition of:
(Independent Spent Fuel Storage Installation)	**PAUL TRUDEAU** and **THOMAS Y. CHANG**

Wednesday, November 15, 2000 - 9:14 a.m.

Location: Utah Attorney General's Office
160 E. 300 S.
Salt Lake City, Utah

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

CitiCourt
THE REPORTING GROUP

50 South Main, Suite 920
Salt Lake City, Utah 84144

801.532.3441 TOLL FREE 877.532.3441 FAX 801.532.3414

State's
Exhibit 98

building. With respect to that aspect of the soils program, what was the goal, what was the purpose of that investigation?

A. (Mr. Trudeau) For the cone penetration test program, I was asked to provide input on how to best demonstrate that we had tested the weakest and most compressible soils in the upper layer. The NRC had asked that we make some field vein measurements in a few locations to demonstrate that that statement was correct. And I argued that we could get much more bang for the buck to do the cone penetration testing work, and that that program would also demonstrate that we have fairly consistent properties for that upper layer across the pad emplacement area, by spending that money to do the cone penetration work rather than the eight vein shear tests that the NRC had been suggesting might be the right way to go.

The canister transfer building borings were laid out with the intention of providing adequate samples, undisturbed samples, to get properties for that -- the design of that safety related structure and to comply with 1.132 type requirements, Regulatory Guide requirements.

Q. And in terms of the objectives of both the CPT and the CTP borings, was that a -- prior to

CONDENSED TRANSCRIPT

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of:	Docket No. 72-22-ISFSI ASLBP No. 97-732-02-ISFSI
PRIVATE FUEL STORAGE, LLC	Deposition of:
(Independent Spent Fuel Storage Installation)	**DR. STEVEN F. BARTLETT** and
	DR. FARHANG OSTADAN
	Vol. I

Thursday, November 16, 2000 - 10:11 a.m.

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

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

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50 South Main, Suite 920
Salt Lake City, Utah 84144

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MR. TRAVIESO-DIAZ: Just go off the record for a second while we look for those.
Let's go back on the record.
I'm going to mark as Exhibit 59 a document that I cannot identify because I didn't prepare it, but I'm going to ask the witness to identify. I would say for the record that it consists of one, two, three, four, five, six, seven -- eight pages of plots, hand plots, and also further state for the record that this document was provided to me by Counsel for the state yesterday.
(Exhibit 59 marked.)
And for the record, the reason they're not colored is I couldn't get copies in color in the time we had, so we're going through black and white copies.
Q. (By Mr. Travieso-Diaz) Now, could you explain to us, identify what this Exhibit 59 is?
A. (Dr. Bartlett) These are the CPT data, and plotted is the tip resistance. These are actually data from the SAR that have been enlarged on a photocopier, and then I traced over them with a pen. It's just a way to try to see what is the variation from CPT to CPT across -- I think all CPT's are represented here. At least it goes to CPT-39. I did this roughly in groups of five, because if you get too many lines it gets

difficult to even understand what they mean.
Maybe it would be easier to do this plot by plot, if you so choose.
Q. Before we go plot by plot, let me see if we can get some description in the record of how this particular document was prepared. First, what was your original source for the preparation of these plots?
A. (Dr. Bartlett) Your diagrams in the SAR, CPT diagrams in the SAR.
Q. The diagrams, do you mean the foundation plots that we looked at before?
A. (Dr. Bartlett) No, these came from actually -- no, these did not come from the SAR. These came from the ConeTec report. Excuse me. These were the plots from the ConeTec and then enlarged on the photocopier.
Q. And when you say "from ConeTec," again, for the record, what is that you're talking about?
A. (Dr. Bartlett) The ConeTec report to provide the cone penetrometer data.
Q. So this is taken from the report done by the contractor that performed the cone penetration tests?
A. (Dr. Bartlett) That's correct.
Q. And this is a reproduction of those plots?
A. (Dr. Bartlett) This is -- yeah, hand

reproduction of those plots.
Q. Well, you said by hand reproduction. How did you do it?
A. (Dr. Bartlett) I simply took the plot, enlarged it on the photocopier, then laid an overhead transparency on top of it and traced down the tip stress.
Q. All right. Now, let's take a look at the first document in this package, which --
MS. CHANCELLOR: Could I just go on the record? What Dr. Bartlett actually prepared were transparencies, and what I gave you was a color photo of the transparency because I couldn't reproduce this transparency.
MR. TRAVIESO-DIAZ: Well, let me ask the witness so that we know what's the best source.
Q. (By Mr. Travieso-Diaz) Would the best source for the original copy of the record be the transparency as opposed to the color copy?
A. (Dr. Bartlett) The best source of the original?
Q. Yeah, the best --
A. (Dr. Bartlett) I would say the color photocopies. I think they're adequate. I don't think they've been distorted markedly.

Q. Fine. Now, let us look at the first of these sets of plots.
A. (Dr. Bartlett) Sure.
Q. For some reason, the way I have them, the first one is for CPT-6 through 10.
A. (Dr. Bartlett) No. Actually, the first one should be CPT-1 through 5.
Q. But the way that this document is numbered, the first one that appears is 6 through 10. On my copy, anyhow.
A. (Dr. Bartlett) Yeah, they're just out of order.
Q. All right. So you are directing my attention, then, to the last page of the exhibit?
A. (Dr. Bartlett) I always, just for some reason, want to start at one.
Q. No problem. Just so the record is clear as to what we're talking about.
A. (Dr. Bartlett) Let's go through the plot leg with CPT-1 through 5, and it's in brown in the color versions.
Q. Are all the plots in brown?
A. (Dr. Bartlett) All of the CPT-1 through 5 are all plotted in brown, yes.
Q. So you don't lose any quality just by having

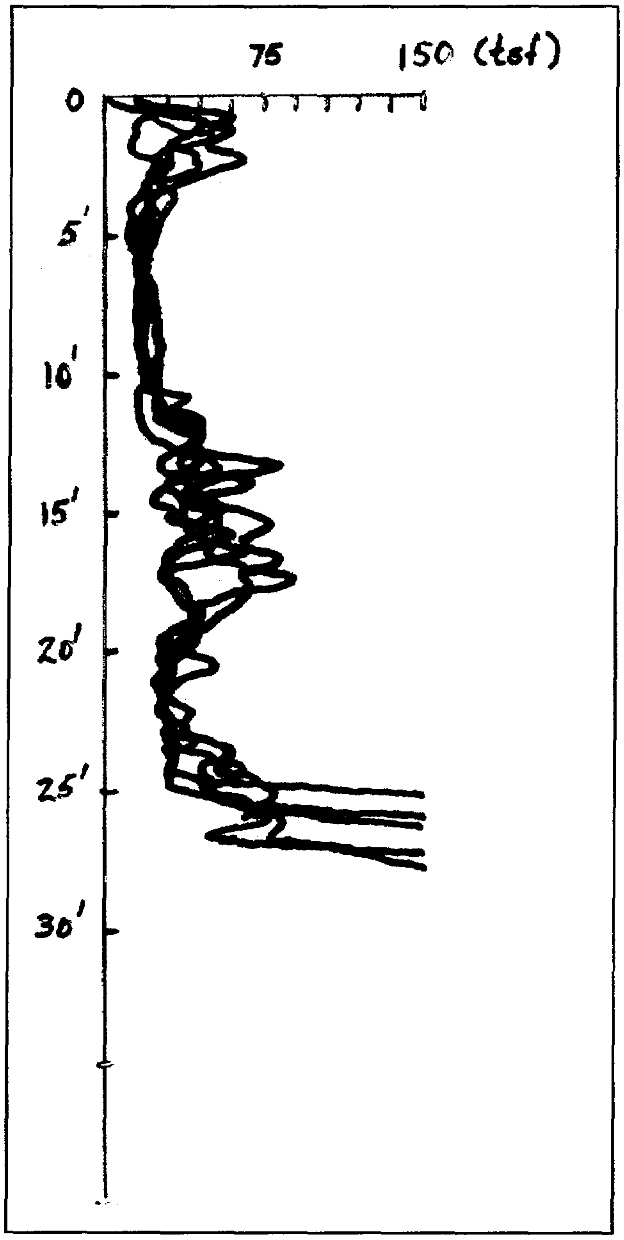

Figure 1. Composite Plot of Cone Penetrometer Test (CPT) traces of tip stress (tons per square foot - x axis) versus depth (feet - y axis) for CPT soundings 1 through 5.

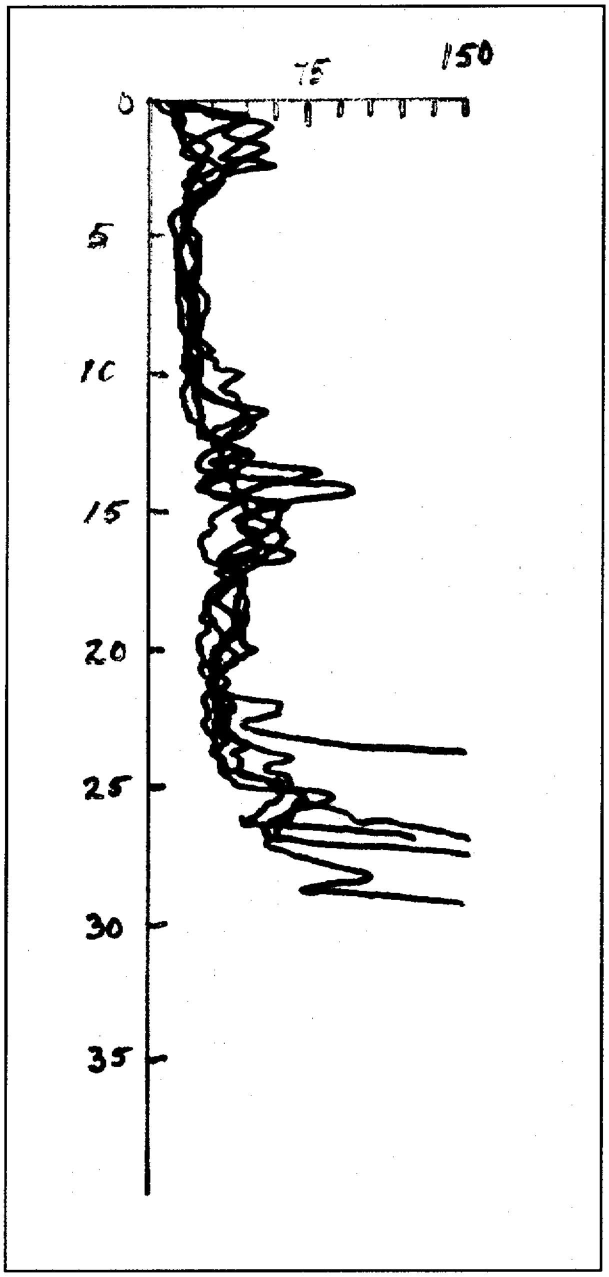

Figure 2. Composite Plot of Cone Penetrometer Test (CPT) traces of tip stress (tons per square foot - x axis) versus depth (feet - y axis) for CPT soundings 6 through 10.

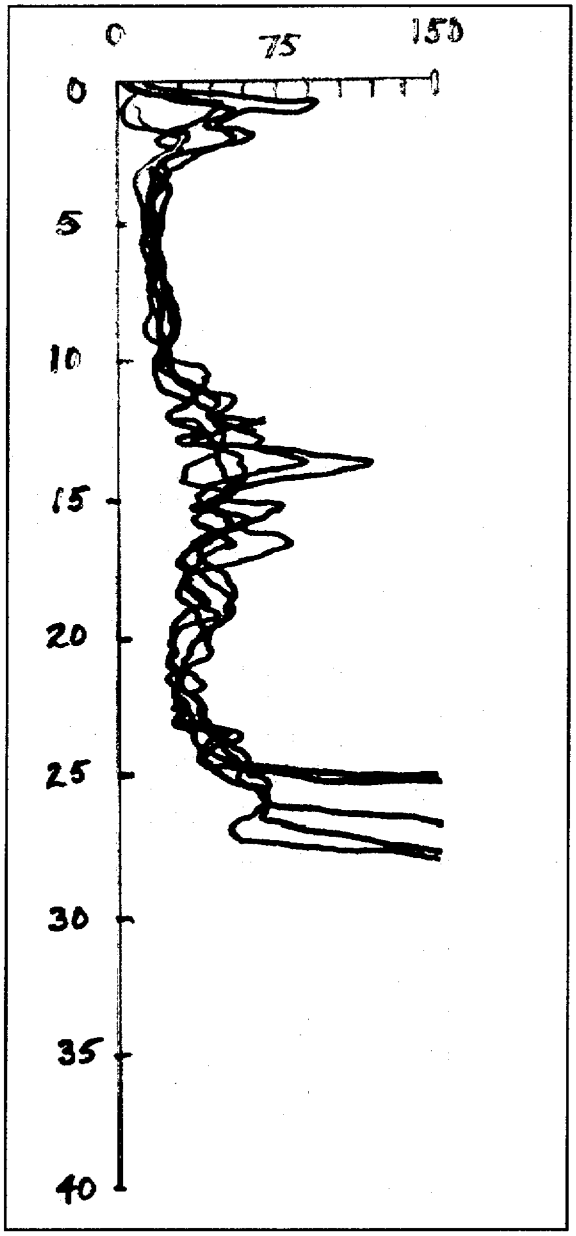

Figure 3. Composite Plot of Cone Penetrometer Test (CPT) traces of tip stress (tons per square foot - x axis) versus depth (feet - y axis) for CPT soundings 11 through 15.

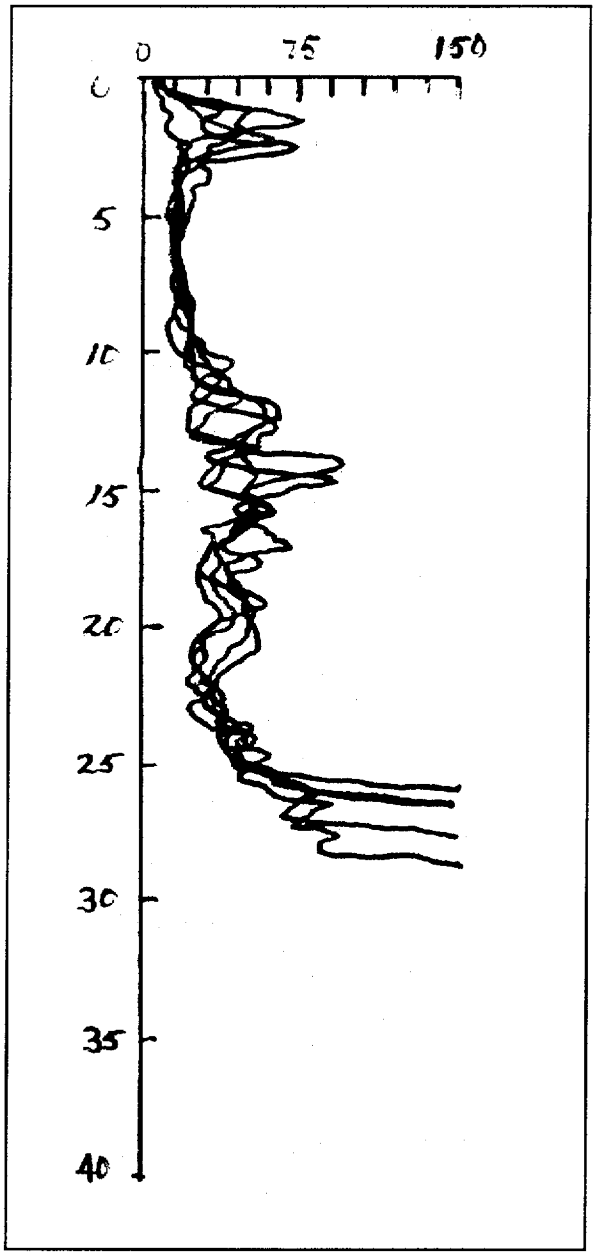

Figure 4. Composite Plot of Cone Penetrometer Test (CPT) traces of tip stress (tons per square foot - x axis) versus depth (feet - y axis) for CPT soundings 16 through 20.

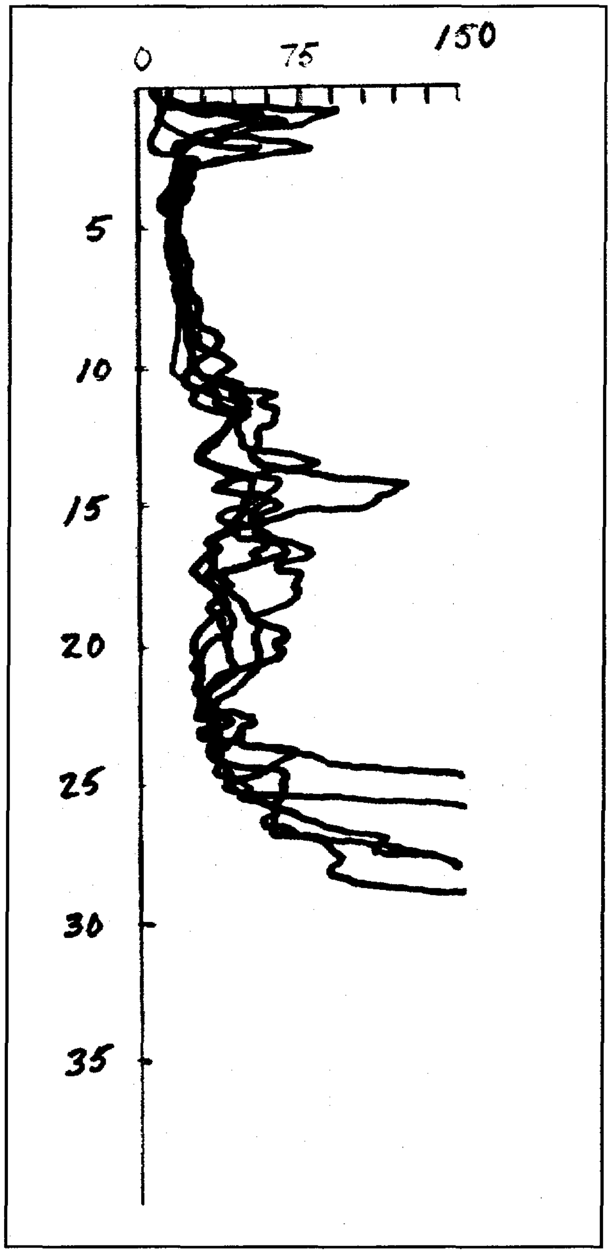

Figure 5. Composite Plot of Cone Penetrometer Test (CPT) traces of tip stress (tons per square foot - x axis) versus depth (feet - y axis) for CPT soundings 21 through 25.

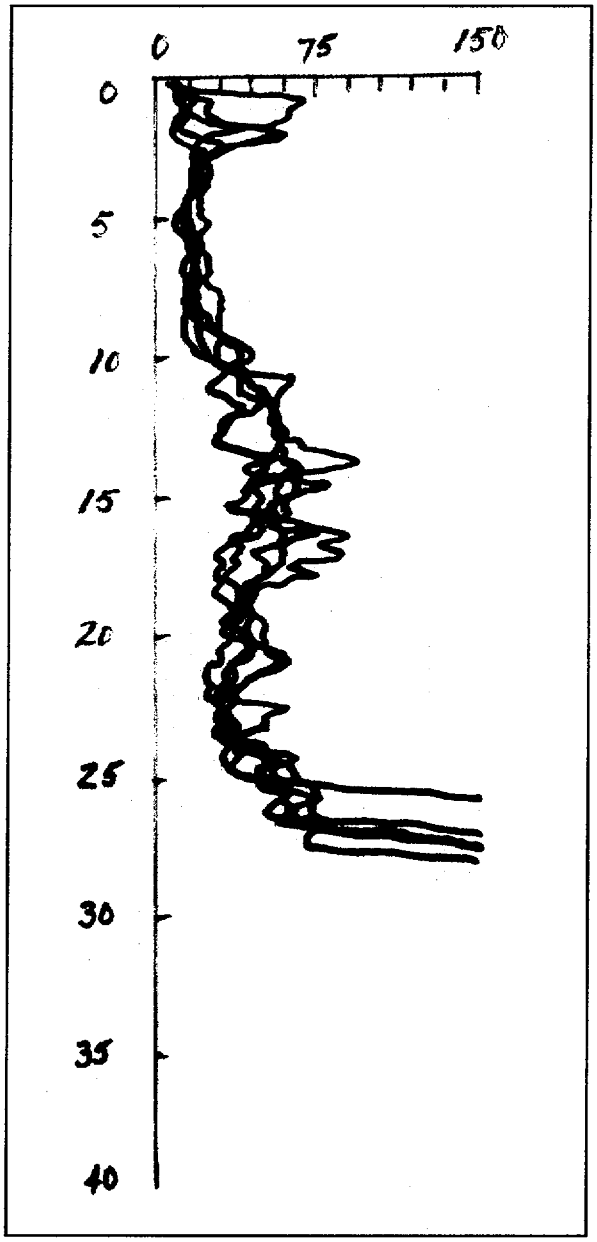

Figure 6. Composite Plot of Cone Penetrometer Test (CPT) traces of tip stress (tons per square foot - x axis) versus depth (feet - y axis) for CPT soundings 26 through 30.

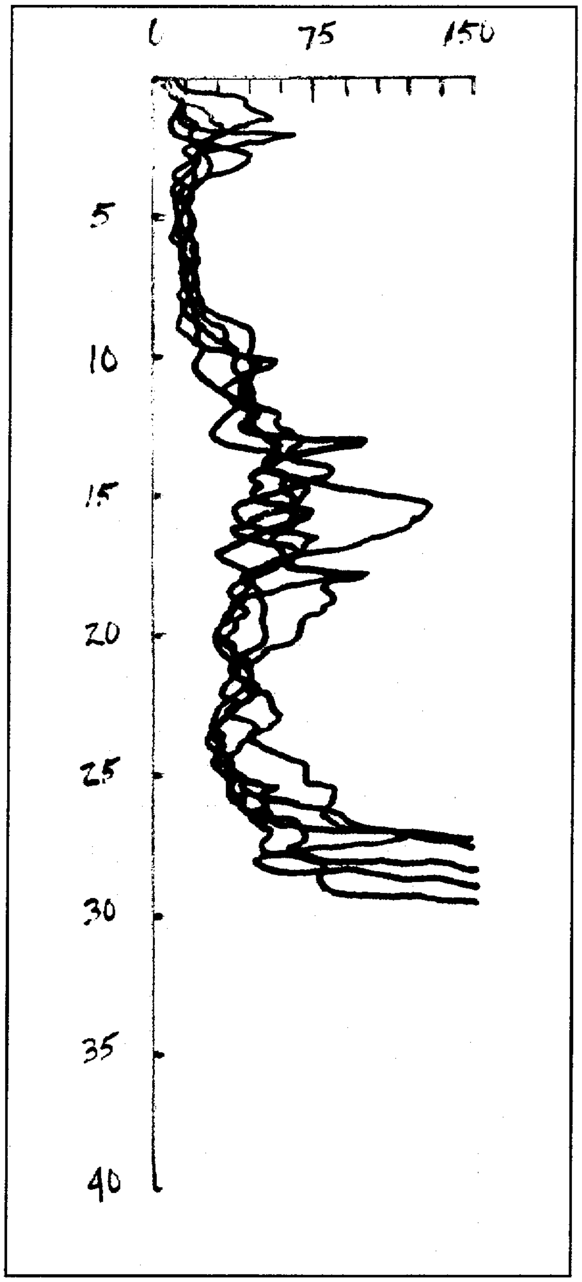

Figure 7. Composite Plot of Cone Penetrometer Test (CPT) traces of tip stress (tons per square foot - x axis) versus depth (feet - y axis) for CPT soundings 31 through 35.

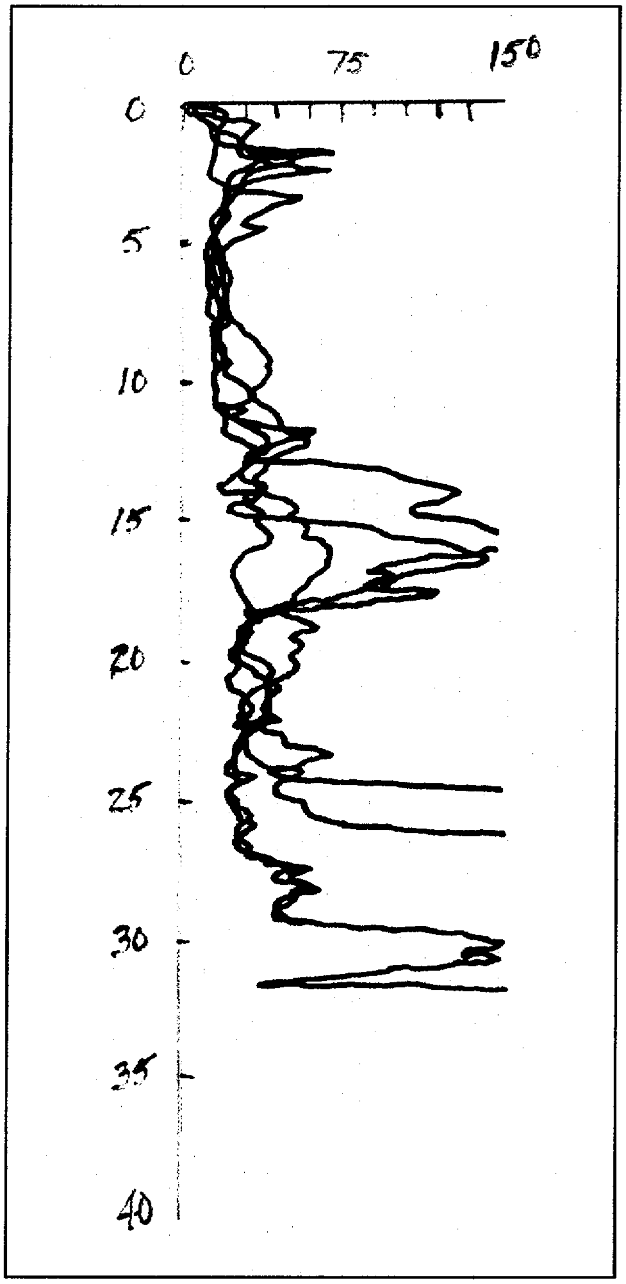

Figure 8. Composite Plot of Cone Penetrometer Test (CPT) traces of tip stress (tons per square foot - x axis) versus depth (feet - y axis) for CPT soundings 36 through 39.

Manual on Estimating Soil Properties for Foundation Design

EL-6800
Research Project 1493-6

Final Report, August 1990

Prepared by

CORNELL UNIVERSITY
Geotechnical Engineering Group
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Exhibit 100

clay can affect the N value greatly, as shown in Figure 4-51. Apparently, the penetration process causes temporary excess pore water stresses which reduce the effective stresses in the vicinity of the sampler, thereby resulting in an apparently lower N value.

However, for clays within a given geology, a reasonable correlation might be expected between s_u and N. Figure 4-52 indicates this behavior over a wide range of N values where the same drilling equipment, SPT procedure, and consistent reference strength (UU triaxial) were employed. For these data, the reported regression is given by:

$$s_u/p_a = 0.29\ N^{0.72} \qquad (4\text{-}60)$$

This equation tends to predict s_u/p_a on the high side of the relationships shown in Figure 4-50.

Correlations with CPT q_c Value

The theoretical relationship for the cone tip resistance in clay is given by:

$$q_c = \overset{40}{\underset{10}{N_k}}\ s_u + \sigma_{vo} \qquad s_u = \frac{q_c - \sigma_{vo}}{N_k} \qquad (4\text{-}61)$$

tsf to 0.5 tsf

in which q_c = cone tip resistance, σ_{vo} = total overburden stress, and N_k = cone bearing factor. The application of classical plasticity theory to this bearing capacity problem suggests N_k on the order of 9 for a general shear model. Cavity

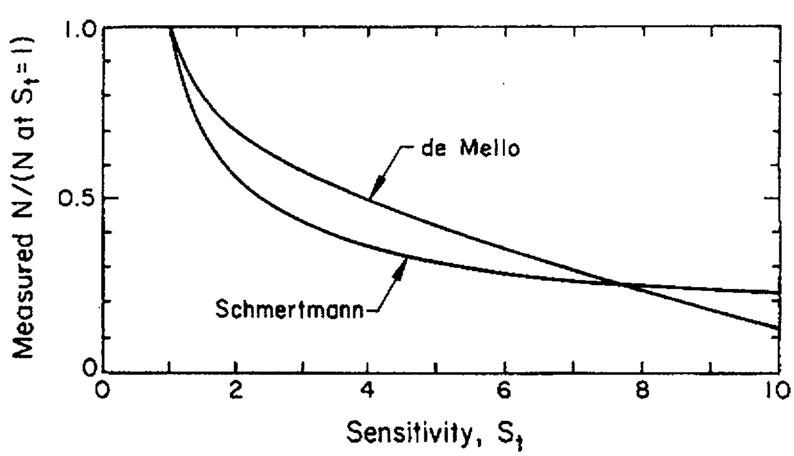

Figure 4-51. Apparent Decrease of N with Increasing Sensitivity

Source: Schmertmann (14), p. 66.

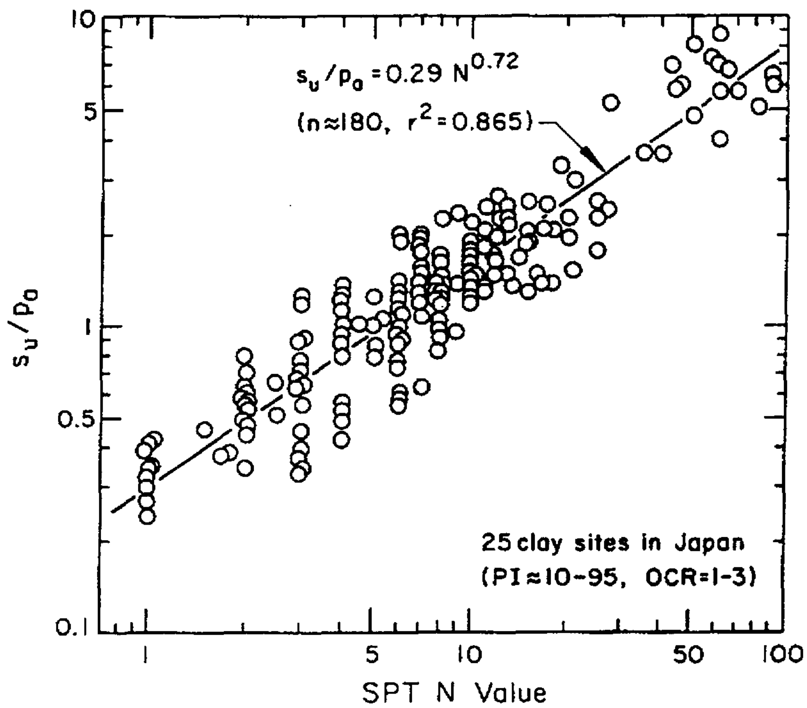

Figure 4-52. Relationship Between s_u and SPT N Value

Source: Hara, et al. (72), p. 9.

expansion theories give N_k increasing in the range of 7 to 13 for increasing values of rigidity index ($I_r = G/s_u$, with G = shear modulus). Steady penetration theory provides a narrow range for N_k between 14 and 18 for a wide range of I_r.

With the various uncertainties in choosing appropriate theoretical models, it is not surprising that N_k usually is determined empirically by calibrating CPT data with a known measured value of s_u. The range of values of N_k back-calculated from CPT data is presented in Figure 4-53. This wide range of N_k values must be scrutinized for several reasons: (1) inconsistent reference strengths, (2) mixing of different type cones (electric and mechanical), and (3) need for correction of q_c for pore water stress effects (Appendix B). These factors can change N_k dramatically.

The importance of correcting q_c for pore water stress effects has been discussed previously and is illustrated by Figure 4-54 for two piezocones with different area ratios. The corrected cone tip resistance (q_T) can be obtained only by use of piezocones with porous elements located behind the tip. Consequently, the large scatter observed in empirical determinations of N_k may result, in part, from use of

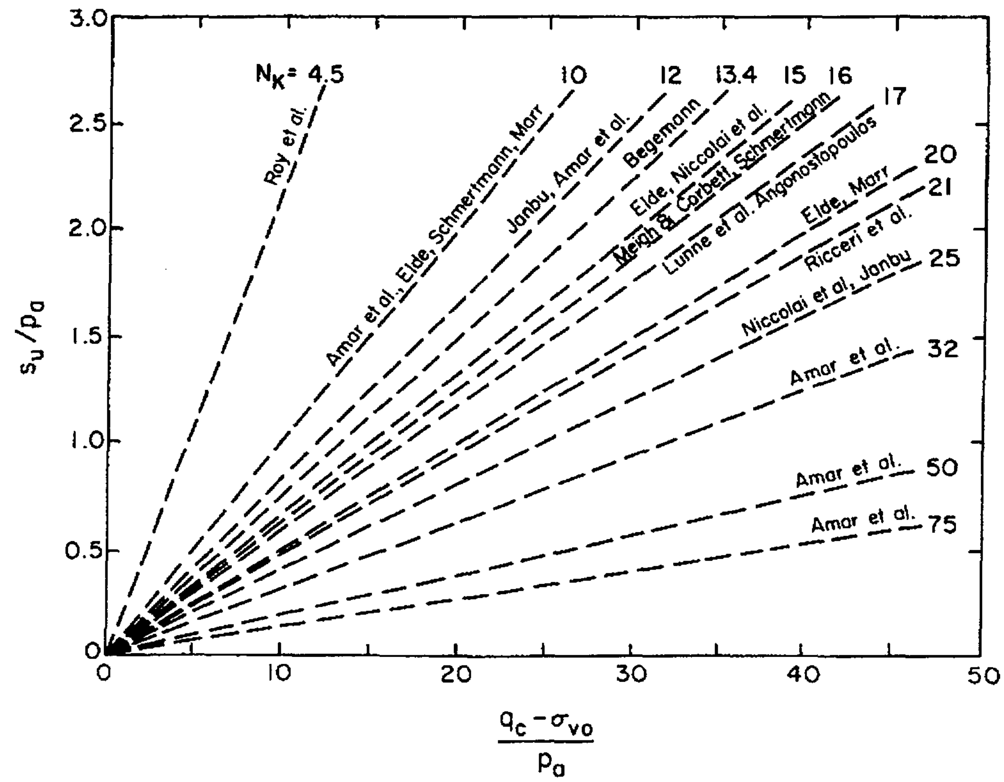

Figure 4-53. Reported Range of N_k Factors from CPT Data

Source: Djoenaidi (71), p. 5-83.

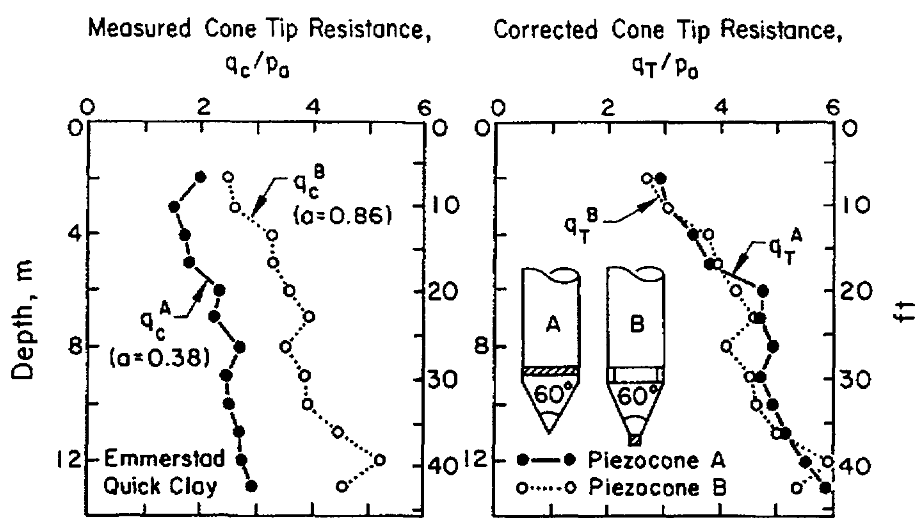

Figure 4-54. Effect of Pore Water Stress on Cone Tip Resistance

Source: Aas, et al. (67), p. 19.

an uncorrected q_c.

The value of N_k ideally should be determined experimentally by comparison with a consistent reference strength. Often, the field VST is used as the reference. In this regard, it is important to recall that the VST requires a correction for s_u in itself. Early correlations (e.g., Battaglio, et al., 73) for N_k using uncorrected VST data suggested a trend for N_k in terms of the plasticity index (PI). However, upon later re-analysis of the same data using the corrected VST strength [μ s_u(VST)], N_k apparently was independent of PI.

Subsequent studies by Keaveny and Mitchell (74) and Konrad and Law (75) have demonstrated that Vesić's cavity expansion theory (76) provides a reasonable estimate for N_k, as given below:

$$N_k = 2.57 + 1.33 (\ln I_r + 1) \tag{4-62}$$

Keaveny and Mitchell suggest using CK_oUC triaxial compression tests to evaluate I_r, while Konrad and Law recommend using the self-boring pressuremeter test.

Recent theoretical developments (Houlsby and Teh, 77) suggest that more refined procedures for determining s_u from the CPT may be appropriate. However, these models currently require a number of parameters that are difficult to determine. Further testing in the future may allow convenient determination of these parameters and a better estimation of s_u.

Correlations with CPTU Results

The piezocone penetration test (CPTU) permits determination of s_u from the corrected cone tip resistance (q_T), as described previously, and also allows for a separate estimate of s_u from the pore water stress measurement. Research on this subject (e.g., Robertson, et al., 78) has suggested the following:

$$s_u = \Delta u / N_{\Delta u} \tag{4-63}$$

in which Δu = measured excess pore water stress (u_m - u_o) and $N_{\Delta u}$ = pore water stress ratio, which may be estimated from A_f and either the PI or rigidity index, as shown in Figure 4-55. Alternative recommendations by Konrad and Law (75) suggest a more complex relationship, including a number of parameters which are somewhat difficult to evaluate.

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CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

ID'D ON TOC

5010.64 (FRONT)

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PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY	36 + 23 INSERTS + 57 pp OF ATT'S
CALCULATION TITLE (Indicative of the Objective): DOCUMENT BASES FOR GEOTECHNICAL PARAMETERS PROVIDED IN GEOTECHNICAL DESIGN CRITERIA	QA CATEGORY (✓) ☑ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER

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1	05996.02	G(B)	05		119

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PAUL J. TRUDEAU 5/7/97 Paul J. Trudeau; TOM SPONSELLER 5/7/97 Tom Sponseller	ALAN F. BROWN 5/8/97 Alan F Brown	ALAN F. BROWN 5/8/97 Alan F Brown	0			✓
Alan C. Smith Alan C Smith; PAUL J. TRUDEAU ~~12-16-99~~ Paul J. Trudeau 1-21-2000	Thomas Y. Chang ~~12-20-99~~ TYC Thomas Y. Chang 1-21-2000	Thomas Y. Chang ~~12-20-99~~ TYC Thomas Y. Chang 1-21-2000	1	0 & 05996.01-G(B)-01, REV 3		✓
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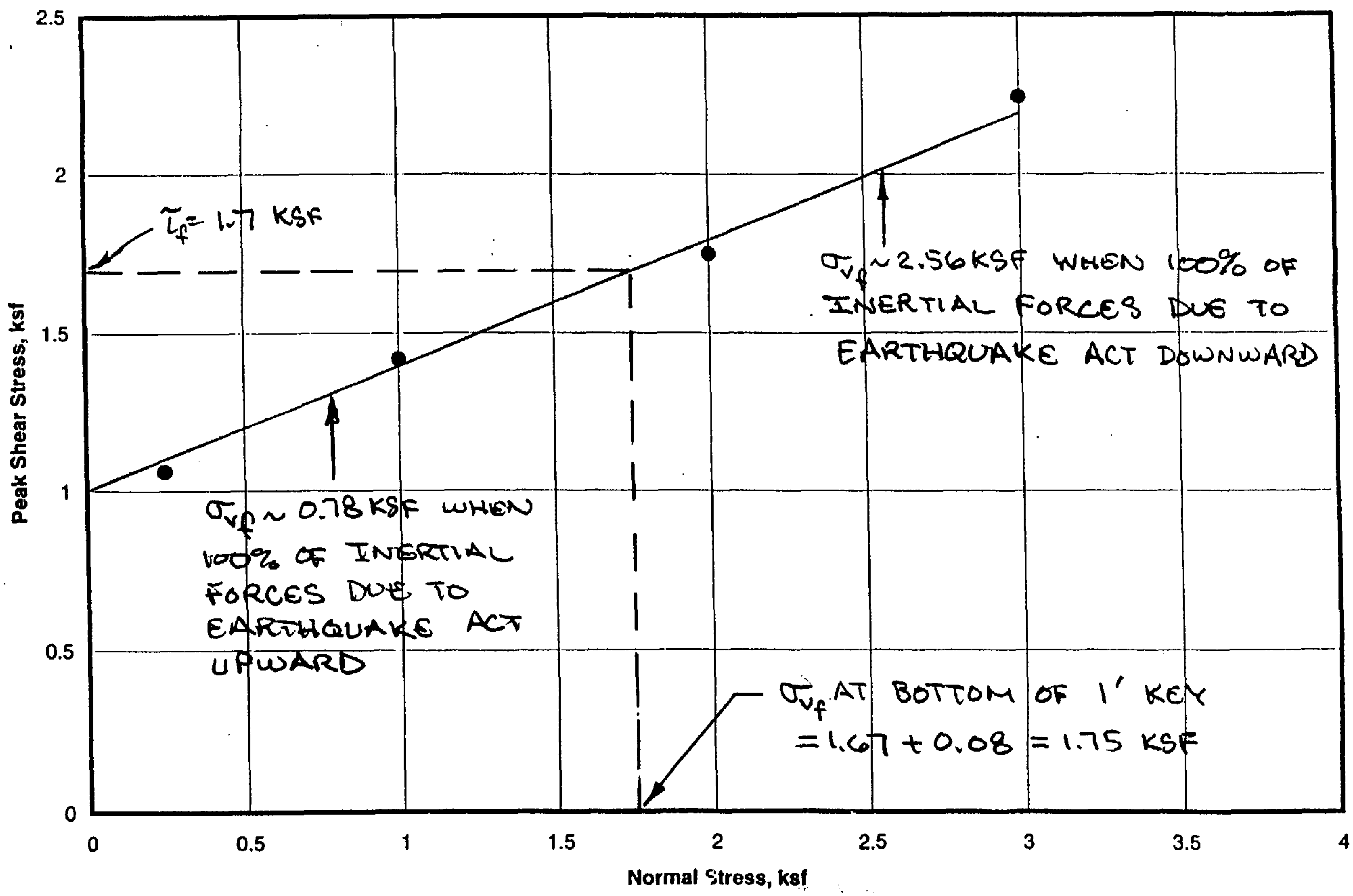
FIGURE 10

DIRECT SHEAR TEST

Boring CTB-S, Sample U-1AA&C

CANISTER TRANSFER BUILDING AREA



CALC 05996.02-G(B)-05-2 P 35

JOURNAL OF THE
GEOTECHNICAL
ENGINEERING DIVISION

Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations

By Faiz I. Makdisi,[1] A. M. ASCE and H. Bolton Seed,[2] F. ASCE

Introduction

In the past decade major advances have been achieved in analyzing the stability of dams and embankments during earthquake loading. Newmark (13) and Seed (18) proposed methods of analysis for predicting the permanent displacements of dams subjected to earthquake shaking and suggested this as a criterion of performance as opposed to the concept of a factor of safety based on limit equilibrium principles. Seed and Martin (26) used the shear beam analysis to study the dynamic response of embankments to seismic loads and presented a rational method for the calculation of dynamic seismic coefficients for earth dams. Ambraseys and Sarma (1) adopted the same procedure to study the response of embankments to a variety of earthquake motions.

Later the finite element method was introduced to study the two-dimensional response of embankments (5,7) and the equivalent linear method (21) was used successfully to represent the strain-dependent nonlinear behavior of soils. In addition the nature of the behavior of soils during cyclic loading has been the subject of extensive research (10,20,23,29). Both the improvement in the analytical tools to study the response of embankments and the knowledge of material behavior during cyclic loading led to the development of a more rational approach to the study of stability of embankments during seismic loading. Such an approach was used successfully to analyze the Sheffield Dam failure during the 1925 Santa Barbara earthquake (24) and the behavior of the San Fernando Dams during the 1971 earthquake (25). This method has since been used extensively in the design and analysis of many large dams in the State of California and elsewhere.

Note.—Discussion open until December 1, 1978. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 104, No. GT7, July, 1978. Manuscript was submitted for review for possible publication on August 30, 1977.

[1] Project Engr., Woodward-Clyde Consultants, San Francisco, Calif.
[2] Prof. of Civ. Engrg., Univ. of California, Berkeley, Calif.

From the study of the performance of embankments during strong earthquakes, two distinct types of behavior may be discerned: (1) That associated with loose to medium dense sandy embankments, susceptible to rapid increases in pore pressure due to cyclic loading resulting in the development of pore pressures equal to the overburden pressure in large portions of the embankment, associated reductions in shear strength, and potentially large movements leading to almost complete failure; and (2) the behavior associated with compacted cohesive clays, dry sands, and some dense sands; here the potential for buildup of pore pressures is much less than that associated with loose to medium dense sands, the resulting cyclic strains are usually quite small, and the material retains most of its static undrained shearing resistance so that the resulting post-earthquake behavior is a limited permanent deformation of the embankment.

The dynamic analysis procedure proposed by Seed, et al. (25) has been used to predict adequately both types of embankment behavior using the "Strain Potential" concept. Procedures for integrating strain potentials to obtain the overall deformation of an embankment have been proposed by Seed, et al. (25), Lee (9), and Serff, et al. (27).

The dynamic analysis approach has been recommended by the Committee on Earthquakes of the International Commission on Large Dams (3): "high embankment dams whose failure may cause loss-of-life or major damage should be designed by the conventional method at first, followed by a dynamic analysis in order to investigate any deficiencies which may exist in the pseudo-statical design of the dam." For low dams in remote areas the Committee recommended the use of conventional pseudostatic methods using a constant horizontal seismic coefficient selected on the basis of the seismicity of the area. However, the inadequacy of the pseudostatic approach to predict the behavior of embankments during earthquakes has been clearly recognized and demonstrated (19,24,25,26, 28). Furthermore in the same report (3) the Commission refers to the conventional method as follows: "There is a need for early revision of the conventional method since the results of dynamic analyses, model tests and observations of existing dams show that the horizontal acceleration due to earthquake forces varies throughout the height of the dam . . . . in several instances, this method predicts a safe condition for dams which are known to have had major slides."

It is this need for a simple yet rational approach to the seismic design of small embankments that prompted the development of the simplified procedure described herein.

This approximate method uses the concept originally proposed by Newmark (13) for calculating permanent deformations but it is based on an evaluation of the dynamic response of the embankment as proposed by Seed and Martin (26) rather than rigid body behavior. It assumes that failure occurs on a well-defined slip surface and that the material behaves elastically at stress levels below failure but develops a perfectly plastic behavior above yield. The method involves the following steps:

1. A yield acceleration, i.e., an acceleration at which a potential sliding surface would develop a factor of safety of unity is determined. Values of yield acceleration are a function of the embankment geometry, the undrained strength of the material (or the reduced strength due to shaking), and the location of the potential sliding mass.

2. Earthquake induced accelerations in the embankment are determined using dynamic response analyses. Finite element procedures using strain-dependent soil properties can be used for calculating time histories of acceleration, or simpler one-dimensional techniques might be used for the same purpose. From these analyses, time histories of average accelerations for various potential sliding masses can be determined.

3. For a given potential sliding mass, when the induced acceleration exceeds the calculated yield acceleration, movements are assumed to occur along the direction of the failure plane and the magnitude of the displacement is evaluated by a simple double integration procedure.

The method has been applied to dams with heights in the range of 100 ft-200 ft (30 m-60 m), and constructed of compacted cohesive soils or very dense cohesionless soils, but may be applicable to higher embankments. A similar approach has been proposed by Sarma (16) using the assumption of a rigid block on an inclined plane rather than a deformable earth structure that responds with differential motions to the imposed base excitation.

In the following sections the steps involved in the analyses will be described in detail and design curves prepared on the basis of analyzed cases will be presented, together with an example problem to illustrate the use of the method. Note, however, that the method is an approximate one and involves simplifying assumptions. The design curves are averages based on a limited number of cases analyzed and should be updated as more data become available and more cases are studied.

Determination of Yield Acceleration

The yield acceleration, k_y, is defined as that average acceleration producing a horizontal inertia force on a potential sliding mass so as to produce a factor of safety of unity and thus cause it to experience permanent displacements.

For soils that do not develop large cyclic strains or pore pressures and maintain most of their original strength after earthquake shaking, the value of k_y can be calculated by stability analyses using limiting equilibrium methods. In conventional slope stability analyses the strength of the material is defined as either the maximum deviator stress in an undrained test, or the stress level that would cause a certain allowable axial strain, say 10%, in a test specimen. However, the behavior of the material under cyclic loading conditions is different than that under static conditions. Due to the transient nature of the earthquake loading, an embankment may be subjected to a number of stress pulses at levels equal to or higher than its static failure stress that simply produce some permanent deformation rather than complete failure. Thus the yield strength is defined, for the purpose of this analysis, as that maximum stress level below which the material exhibits a near elastic behavior (when subjected to cyclic stresses of numbers and frequencies similar to those induced by earthquake shaking) and above which the material exhibits permanent plastic deformation of magnitudes dependent on the number and frequency of the pulses applied. Fig. 1 shows the concept of cyclic yield strength. The material in this case has a cyclic yield strength equal to about 90% of its static undrained strength and as shown in Fig. 1(*a*) the application of 100 cycles of stress amounting to 80%

of the undrained strength resulted in essentially an elastic behavior with very little permanent deformation. On the other hand, the application of 10 cycles of stress level equal to 95% of the static undrained strength led to substantial permanent strain as shown in Fig. 1(*b*). On loading the material monotonically to failure after the series of cyclic stress applications, the material was found to retain the original undrained strength. This type of behavior is associated with various types of soils that exhibit small increases in pore pressure during cyclic loading. This would include clayey materials, dry or partially saturated cohesionless soils, or very dense saturated cohesionless materials that will not undergo significant deformations, even under cyclic loading conditions, unless the undrained static strength of the soil is exceeded.

Seed and Chan (20) conducted cyclic tests on samples of undisturbed and compacted silty clays and found that for conditions of no stress reversal and for different values of initial and cyclic stresses, the total stress required to produce large deformations in 10 cycles and 100 cycles ranged between 90%–110% of the undrained static strength.

Sangrey, et al. (15) investigated the effective stress response of clay under repeated loading. They tested undisturbed samples of clay (LL = 28, PI = 10) and found that the cyclic yield strength of this material was of the order of 60% of its static undrained strength.

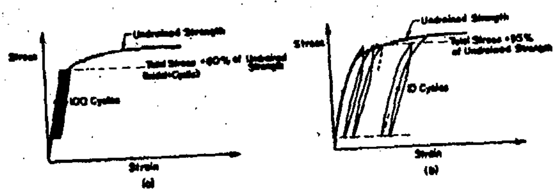


FIG. 1.—Determination of Dynamic Yield Strength

Rahman (14) performed similar tests on remolded samples of a brittle silty clay (LL = 91, PI = 49) and found that the cyclic yield strength was a function of the initial effective confining pressure. For practical ranges of effective confining pressures the cyclic yield strength for this material ranged between 80%–95% of its static undrained strength. At cyclic stress levels below the yield strength, in all cases, the material reached equilibrium and assumed an elastic behavior at strain levels less than 2% irrespective of the number of stress cycles applied.

Thiers and Seed (28) performed tests on undisturbed and remolded samples of different clayey materials to determine the reduction in static undrained strength due to cyclic loading. Their results are summarized in Fig. 2 which shows the reduction in undrained strength after cyclic loading as a function of the ratio of the "maximum cyclic strain" to the "static failure strain." These results were obtained from strain controlled cyclic tests; after the application of 200 cycles of a certain strain amplitude, the sample was loaded to failure monotonically at a strain rate of 3%/min. Thus from Fig. 2 it could be argued that if a clay is subjected to 200 cycles of strain with an amplitude less than half its static failure strain, the material may be expected to retain at least 90% of its original static undrained strength.

Andersen (2), on the basis of cyclic simple shear tests on samples of Drammen clay, determined that the reduction in undrained shear strength was found to be less than 25% as long as the cyclic shear strain was less than ±3% even after 1,000 cycles. Some North Sea clays, however, have shown a strength reduction of up to 40% for the same level of cyclic loading.

On the basis of the experimental data reported previously and for values

TABLE 1.—Maximum Cyclic Shear Strains Calculated from Dynamic Finite Element Response Analyses

Magnitude (1)	Embankment height, in feet (2)	Slope, *H*:*V* (3)	Maximum base acceleration, *g* (4)	Maximum shear strain, as a percentage (5)
6-1/2 (Caltech record)	75	2:1	0.5	0.2–0.4
6-1/2 (Caltech record)	150	2:1	0.2	0.1–0.15
6-1/2 (Caltech record)	150	2:1	0.5	0.2–0.3
6-1/2 (Lake Hughes record)	150	2:1	0.2	0.1–0.15
6-1/2 (Caltech record)	150	2-1/2:1	0.5	0.2–0.3
7-1/2 (Taft record)	150	2:1	0.5	0.2–0.5
7-1/2 (Taft record)	150	2:1	0.2	0.1–0.2
8-1/4 (S-I record)	150	2:1	0.75	0.4–1.0
8-1/4 (S-I record)	135	—	0.4	0.2–0.5

Note: 1 ft = 0.305 m.

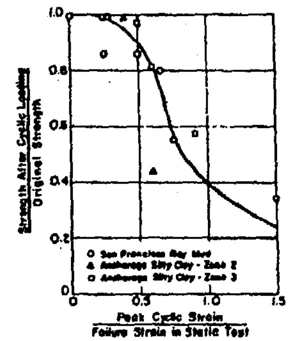


FIG. 2.—Reduction in Static Undrained Strength Due to Cyclic Loading (29)

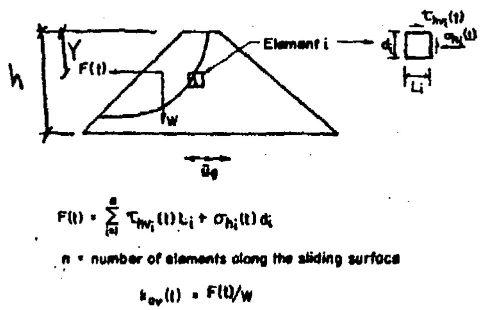


FIG. 3.—Calculation of Average Acceleration from Finite Element Response Analysis

of cyclic shear strains calculated from earthquake response analyses, the value of cyclic yield strength for a clayey material can be estimated. In most cases this value would appear to be 80% or more of the static undrained strength. This value in turn may be used in an appropriate method of stability analysis to calculate the corresponding yield acceleration.

Finite element response analyses (as will be described later) have been carried out to calculate time histories of crest acceleration and average acceleration for various potential sliding masses. The method of analysis employs the equivalent linear technique with strain-dependent modulus and damping. The ranges of calculated maximum shear strains, for different magnitude earthquakes and different embankment characteristics, are presented in Table 1. It can be seen from Table 1 that the maximum cyclic shear strain induced during the earthquakes ranged between 0.1% for a magnitude 6-1/2 earthquake with a base acceleration of 0.2 g and 1% for a magnitude 8-1/4 earthquake with a base acceleration of 0.75 g. For the compacted clayey material encountered in dam embankments "static failure strain" values usually range between 3%–10%, depending on whether the material was compacted on the dry or wet side of the optimum moisture content. Thus in both instances the ratio of the "cyclic strain" to "static failure strain" is less than 0.5. ⇒ 90% STRENGTH

It seems reasonable, therefore, to assume that for these compacted cohesive soils, very little reduction in strength may be expected as a result of strong earthquake loading of the magnitude described previously.

Once the cyclic yield strength is defined, the calculation of the yield acceleration can be achieved by using one of the available methods of stability analysis. In the present study the ordinary method of slices has been used to calculate the yield acceleration for circular slip surfaces using a pseudostatic analysis. As an alternative one of the writers (18) has suggested a method of combining both effective and total stress approaches, where the shear strength on the failure plane during the earthquake is considered to be a function of the initial effective normal stress on that same plane before the earthquake. This method is applicable to noncircular slip surfaces and the horizontal inertia force resulting in a factor of safety of unity can readily be calculated.

Having determined the yield acceleration for a certain location of the slip surface, the next step in the analysis is to determine the time history of earthquake-induced average accelerations for that particular sliding mass. This will be treated in the following section.

Determination of Earthquake Induced Acceleration

In order for the permanent deformations to be calculated for a particular slip surface, the time history of earthquake induced average accelerations must first be determined.

Two-dimensional finite element procedures using equivalent linear strain-dependent properties are available (6) and have been shown to provide response values in good agreement with measured values (8) and with closed-form one-dimensional wave propagation solutions (17).

For most of the case studies of embankments used in the present analysis, the response calculation was performed using the finite element computer program QUAD-4 (6) with strain-dependent modulus and damping. The program uses the Rayleigh damping approach and allows for variable damping to be used in different elements.

To calculate the time history of average acceleration for a specified sliding mass, the method described by Chopra (4) was adopted in the present study. The finite element calculation provides time histories of stresses for every element in the embankment. As shown in Fig. 3, at each time step the forces acting along the boundary of the sliding mass are calculated from the corresponding

normal and shear stresses of the finite elements along that boundary. The resultant of these forces divided by the weight of the sliding mass would give the average acceleration, $k_{av}(t)$, acting on the sliding mass at that instant in time. The process is repeated for every time step to calculate the entire time history of average acceleration.

For a 150-ft (46-m) high dam subjected to 30 sec of the Taft earthquake record scaled to produce a maximum base acceleration of 0.2 g, the variation

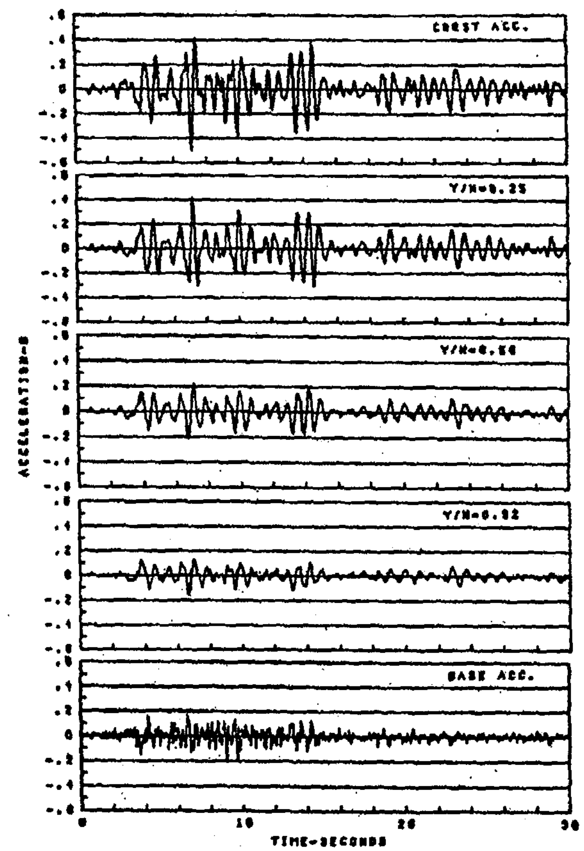


FIG. 4.—Time Histories of Average Acceleration for Various Depths of Potential Sliding Mass

of the time history of k_{av} with the depth of the sliding mass within the embankment, together with the time history of crest accelerations, is shown in Fig. 4.

Comparing the time history of crest acceleration with that of the average acceleration for different depths of the potential sliding mass, the similarity in the frequency content is readily apparent (it generally reflects the first natural period of the embankment), while the amplitudes are shown to decrease as the depth of the sliding mass increases towards the base of the embankment. The maximum crest acceleration is designated by $\ddot{u}_{max}$, and k_{max} is the maximum average acceleration for a potential sliding mass extending to a specified depth, y.

It would be desirable to establish a relationship showing the variation of the maximum acceleration ratio, $k_{max}/\ddot{u}_{max}$, with depth for a range of embankments and earthquake loading conditions. It would then be sufficient, for design purposes, to estimate the maximum crest acceleration in a given embankment due to a specified earthquake and use this relationship to determine the maximum average acceleration for any depth of the potential sliding mass. A simplified procedure to estimate the maximum crest acceleration and the natural period

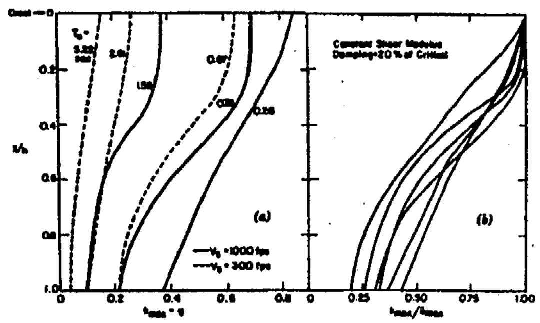


FIG. 5.—El Centro Record (12): (a) Variation of Maximum Average Acceleration with Depth of Sliding; (b) Variation of Ratio of Average Acceleration to Maximum Crest Acceleration with Depth of Sliding Surface

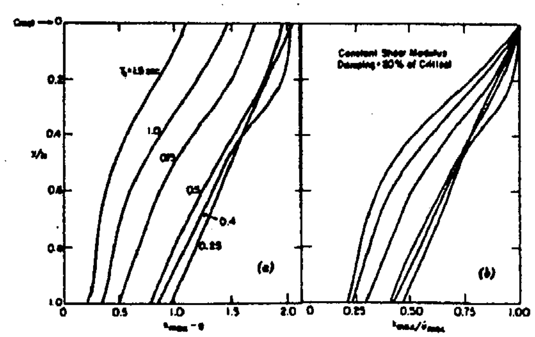


FIG. 6.—Average of Eight Strong Motion Records (1): (a) Variation of Maximum Average Acceleration with Depth of Sliding Mass; (b) Variation of Ratio of Maximum Average Acceleration to Maximum Crest Acceleration with Depth of Sliding Surface

of an embankment subjected to a given base motion is described in Appendix A of Ref. 11.

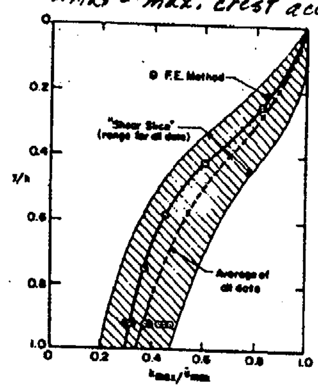
To determine the variation of maximum acceleration ratio with depth, use was made of published results of response computations using the one-dimensional shear slice method with visco-elastic material properties (1,26). Martin (12) calculated the response of embankments ranging in height between 100 ft–600 ft (30 m–180 m) and with shear wave velocities between 300 fps–1,000 fps (92 m/s–300 m/s). Using a constant shear modulus and a damping factor of 0.2,

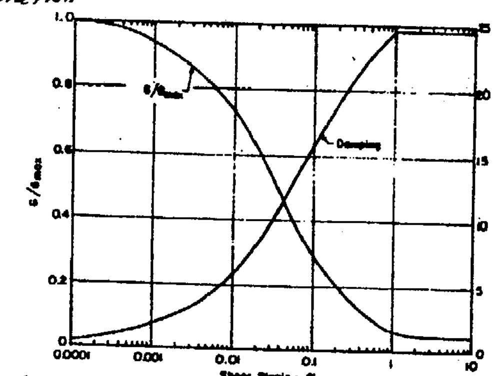
the average acceleration histories for various levels were computed for embankments subjected to ground accelerations recorded in the El Centro earthquake of 1940. The variation of the maximum average acceleration, k_{max}, with depth for these embankments with natural periods ranging between 0.26 sec–5.22 sec is presented in Fig. 5(*a*). The maximum average acceleration in Fig. 5(*a*) is normalized with respect to the maximum crest acceleration and the ratio, $k_{max}/\ddot{u}_{max}$, plotted as a function of the depth of the sliding mass is presented in Fig. 5(*b*).

Ambraseys and Sarma (1) used essentially the same method reported by Seed and Martin (26) and calculated the response of embankments with natural periods ranging between 0.25 sec and 3.0 sec. They presented their results in terms of average response for eight strong motion records. The variation of maximum average acceleration with depth based on the results reported by Ambraseys and Sarma (1) is shown in Fig. 6(*a*) and that for the maximum acceleration ratio, $k_{max}/\ddot{u}_{max}$, is shown in Fig. 6(*b*). A summary of the results obtained

$\ddot{u}_{max}$ = max. crest accleration

See fig 3 for y/h





k_{max} = max. avg. accleration of sliding mass

FIG. 7.—Variation of Maximum Acceleration Ratio with Depth of Sliding Mass

FIG. 8.—Shear Modulus and Damping Characteristics Used in Response Computations

from the different shear slice response calculations mentioned previously is presented in Fig. 7 together with results obtained from finite element calculations made in the present study. As can be seen from Fig. 7 the shape of the curves obtained using the shear slice method and the finite element method are very similar. The dashed curve in Fig. 7 is an average relationship of all data considered. The maximum difference between the envelope of all data and the average relationship ranges from ±10% to ±20% for the upper portion of the embankment and from ±20% to ±30% for the lower portion of the embankment.

Considering the approximate nature of the proposed method of analysis, the use of the average relationship shown in Fig. 7 for determining the maximum average acceleration for a potential sliding mass based on the maximum crest acceleration is considered accurate enough for practical purposes. For design computations where a conservative estimate of the accelerations is desired the upper bound curve shown in Fig. 7 may be used leading to values that are 10%–30% higher than those estimated using the average relationship.

Calculation of Permanent Deformations

Once the yield acceleration and the time history of average induced acceleration for a potential sliding mass have been determined, the permanent displacements can readily be calculated.

By assuming a direction of the sliding plane and writing the equation of

TABLE 2.—Embankment Characteristics for Magnitude 6-1/2 Earthquake

Case number (1)	Embankment description (2)	Height, in feet (3)	Base acceleration, g (4)	T_o, in seconds (5)[a]	k_{max}, g (6)[b]	Symbol[c] (7)
1	Example slope = 2:1 k_{2max} = 60	150	0.2 (Caltech record)	0.8	(1) 0.31 (2) 0.12	● ■
2	Example slope = 2:1 k_{2max} = 60	150	0.5 (Caltech record)	1.08	(1) 0.4 (2) 0.18	○ □
3	Example slope = 2:1 k_{2max} = 80	150	0.5 (Lake Hughes record)	0.84	(1) 0.33 (2) 0.16	⊙ △
4	Example slope = 2-1/2:1 k_{2max} = 80	150	0.5 (Caltech record)	0.95	(1) 0.49 (2) 0.22	◇ ▽
5	Example slope = 2:1 k_{2max} = 60	75	0.5 (Caltech record)	0.6	(1) 0.86 (2) 0.26	◑ ◪

[a]Calculated first natural period of the embankment.
[b]Maximum value of time history of: (1) Crest acceleration; and (2) average acceleration for sliding mass extending through full height of embankment.
[c]Legend used in Fig. 9(*a*).
Note: 1 ft = 0.305 m.

motion for the sliding mass along such a plane, the displacements that would occur any time the induced acceleration exceeds the yield acceleration may be evaluated by simple numerical integration. For the purposes of the soil types considered in this study, the yield acceleration was assumed to be constant throughout the earthquake.

The direction of motion for a potential sliding mass once yielding occurs was assumed to be along a horizontal plane. This mode of deformation is not uncommon for embankments subjected to strong earthquake shaking, and is manifested in many cases in the field by the development of longitudinal cracks along the crest of the embankment. However studies made for other directions of the sliding surface showed that this factor had little effect on the computed displacements (11).

To calculate an order of magnitude of the deformations induced in embankments due to strong shaking a number of cases have been analyzed during the course of this study. The height of embankments considered ranged between 75 ft–150 ft (23 m–46 m) with varying slopes and material properties. The embankments were subjected to ground accelerations representing three different earthquake magnitudes: 6-1/2, 7-1/2, and 8-1/4.

The method used for calculating the response, as mentioned earlier, is a time-step finite element analysis using the equivalent linear method. The strain-dependent modulus and damping relations for the soils used in this study are

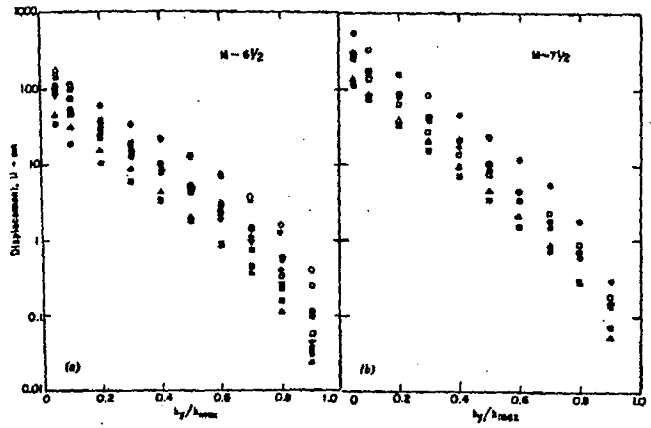


FIG. 9.—Variation of Permanent Displacement with Yield Acceleration: (*a*) Magnitude 6-1/2 Earthquake; (*b*) Magnitude 7-1/2 Earthquake

presented in Fig. 8. The response computation for each base motion was repeated for a number of iterations (mostly 3–4) until strain compatible material properties were obtained. In each case both time histories of crest acceleration and the average acceleration for a potential sliding mass extending through almost the full height of the embankment were calculated, together with the first natural period of the embankment. In one case however, time histories of average acceleration for sliding surfaces at five different levels in the embankment were obtained (see Fig. 4), and the corresponding permanent deformations for each time history were calculated for different values of yield acceleration. It was found that for the same ratio of yield acceleration to maximum average acceleration at each level, the computed deformations varied uniformly between a maximum value obtained using the crest acceleration time history to a minimum value obtained using the time history of average acceleration for a sliding mass extending through the full height of the embankment. Thus it was [illegible] sufficient for the remaining cases to compute the deformations only for these two levels.

Table 2 shows details of the embankments analyzed using ground motions representative of a magnitude 6-1/2 earthquake. The two rock motions used were those recorded at the Cal Tech Seismographic Laboratory (S90W Component) and at Lake Hughes Station No. 12 (N12E) during the 1971 San Fernando earthquake, with maximum accelerations scaled to 0.2 g and 0.5 g. The computed natural periods and maximum values of the acceleration time histories are also presented in Table 2. The computed natural periods ranged between a value of 0.6 sec for the 75-ft (23-m) high embankment to a value of 1.08 sec for the 150-ft (46-m) high embankment. Because of the nonlinear strain-dependent

TABLE 3.—Embankment Characteristics for Magnitude 7-1/2 Earthquake

Case number (1)	Embankment description (2)	Height, in feet (3)	Base acceleration, g (4)	T_0, in seconds (5)[a]	k_{max}, g (6)[b]	Symbol[c] (7)
1	Example slope = 2:1 k_{2max} = 60	150	0.2 (Taft record)	0.86	(1) 0.41 (2) 0.13	● ■
2	Example slope = 2:1 k_{2max} = 60	150	0.5 (Taft record)	1.18	(1) 0.54 (2) 0.21	○ □
3	Example slope = 2-1/2:1 k_{2max} = 80	150	0.2 (Taft record)	0.76	(1) 0.46 (2) 0.15	⊙ △

[a]Calculated first natural period of the embankment.
[b]Maximum value of time history of: (1) Crest acceleration; and (2) average acceleration for sliding mass extending through full height of embankment.
[c]Legend used in Fig. 9(b).
Note: 1 ft = 0.305 m.

behavior of the material, the response of the embankment is highly dependent on the amplitude of the base motion. This is clearly demonstrated in the first two cases in Table 2, where the same embankment was subjected to the same ground acceleration history but with different maximum accelerations for each case. In one instance, for a base acceleration of 0.2 g the calculated maximum crest accelerations was 0.3 g with a magnification of 1.5 and a computed natural period of the order of 0.8 sec. In the second case, for a base acceleration of 0.5 g the computed maximum crest acceleration was 0.4 g with an attenuation of 0.8 and a computed natural period of 1.1 sec.

From the time histories of induced acceleration calculated for all the cases

described in Table 2 and for various ratios of yield acceleration to maximum average acceleration, k_y/k_{max}, the permanent deformations were calculated by numerical double integration. The results are presented in Fig. 9(a) which shows that for relatively low values of yield acceleration, k_y/k_{max} of 0.2 for example, the range of computed permanent displacements was of the order of 10 cm–70 cm (4 in.–28 in.). However, for larger values of k_y/k_{max}, say 0.5 or more, the calculated displacements were less than 12 cm (4.8 in.). It should be emphasized that for very low values of yield accelerations (in this case $k_y/k_{max} \le 0.1$) the basic assumptions used in calculating the response by the finite element

TABLE 4.—Embankment Characteristics of Magnitude 8-1/4 Earthquake

Case number (1)	Embankment description (2)	Height, in feet (3)	Base acceleration, g (4)	T_0, in seconds (5)[a]	k_{max}, g (6)[b]	Symbol[c] (7)
1	Chabot Dam (average properties)	135	0.4 (S-I Synth. record)	0.99	(1) 0.57	○
	Chabot Dam (Lower bound)	135	0.4 (S-I Synth. record)	1.07	(1) 0.53	△
	Chabot Dam (Upper bound)	135	0.4	0.83	(1) 0.68	□
2	Example slope = 2:1 k_{2max} = 60	150	0.75	1.49	(1) 0.74 (2) 0.34	● ■

[a]Calculated first natural period of the embankment.
[b]Maximum value of time history of: (1) Crest acceleration; and (2) average acceleration for sliding mass extending through full height of embankment.
[c]Legend used in Fig. 10(a).
Note: 1 ft = 0.305 m.

method, i.e., the equivalent linear behavior and the small strain theory, become invalid. Consequently, the acceleration time histories calculated for such a case do not represent the real field behavior and the calculated displacements based on these time histories may not be realistic.

The procedure described previously was repeated for the case of a magnitude 7-1/2 earthquake. The base acceleration time history used for this analysis was that recorded at Taft during the 1952 Kern County earthquake and scaled to maximum accelerations of 0.2 g and 0.5 g. The details of the three cases analyzed are presented in Table 3 and the results of the computations of the permanent displacements are shown in Fig. 9(b). For a ratio of k_y/k_{max} of 0.2 the calculated displacements in this case ranged between 30 cm–200 cm (12 in.–80 in.), and for ratios greater than 0.5 the displacements were less than 25 cm (0.8 ft).

In the cases analyzed for the 8-1/4 magnitude earthquake, an artificial accelerogram proposed by Seed and Idriss (21) was used with maximum base accelerations of 0.4 g and 0.75 g. Two embankments were analyzed in this case and their calculated natural periods ranged between 0.8 sec and 1.5 sec. Table 4 shows the details of the calculations and in Fig. 10(a) the results of the permanent displacement computations are presented. As can be seen from Fig. 10(a) the permanent displacements computed for a ratio of k_y/k_{max} of 0.2 ranged between 200 cm–700 cm (80 in.–28 in.), and for ratios higher than 0.5 the values were less than 100 cm (40 in.). Note in this case that values of deformations calculated for a yield ratio less than 0.2 may not be realistic.

An envelope of the results obtained for each of the three earthquake loading

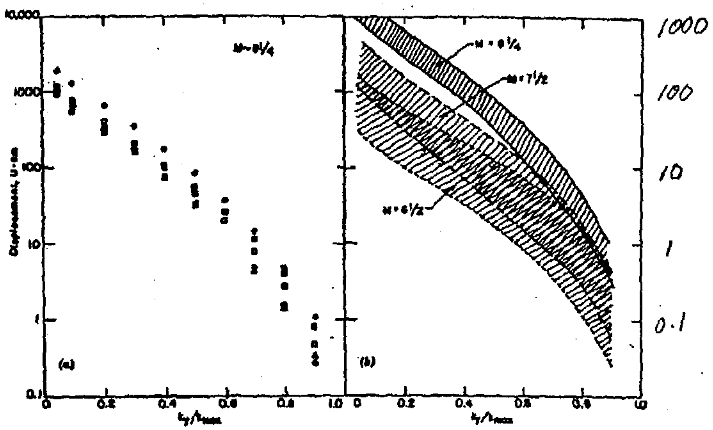


FIG. 10.—Variation of Permanent Displacement with Yield Acceleration: (a) Magnitude 8-1/4 Earthquake; (b) Summary of All Data

conditions is presented in Fig. 10(b) and reveals a large scatter in the computed results reaching, in the case of the magnitude 6-1/2 earthquake, about one order of magnitude.

It can reasonably be expected that for a potential sliding mass with a specified yield acceleration, the magnitude of the permanent deformation induced by a certain earthquake loading is controlled by the following factors: (1) The amplitude of induced average accelerations, which is a function of the base motion, the amplifying characteristics of the embankment, and the location of the sliding mass within the embankment; (2) the frequency content of the average acceleration time history, which is governed by the embankment height and stiffness characteristics, and is usually dominated by the first natural frequency of the embankment; and (3) the duration of significant shaking, which is a function of the magnitude of the specified earthquake.

Thus to reduce the large scatter exhibited in the data in Fig. 10(b), the permanent

displacements for each embankment were normalized with respect to its calculated first natural period, T_0, and with respect to the maximum value, k_{max}, of the average acceleration time history used in the computation. The resulting normalized permanent displacements for the three different earthquakes are presented in Fig. 11(a). It may be seen that a substantial reduction in the scatter of the data is achieved by this normalization procedure as evidenced by comparing the results in Figs. 10(b) and 11(a). This shows that for the ranges of embankment heights considered in this study [75 ft–150 ft (50 m–65 m)] the first natural period of the embankment and the maximum value of acceleration time history may be considered as two of the parameters having a major influence on the calculated permanent displacements. Average curves for the normalized permanent displacements based on the results in Fig. 11(a) are presented in Fig. 11(b). Although some scatter still exists in the results as shown in Fig. 11(a), the average curves presented in Fig. 11(b) are considered adequate to provide an order of magnitude of the induced permanent displacements for different

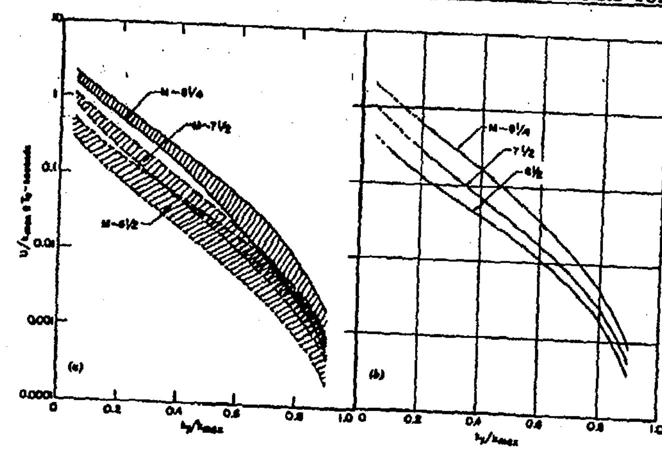


FIG. 11.—Variation of Yield Acceleration with: (a) Normalized Permanent Displacement—Summary of All Data; and (b) Average Normalized Displacement.

magnitude earthquakes. At yield acceleration ratios less than 0.2 the average curves are shown as dashed lines since, as mentioned earlier, the calculated displacements at these low ratios may be unrealistic.

Thus, to calculate the permanent deformation in an embankment constructed of a soil that does not change in strength significantly during an earthquake, it is sufficient to (1) determine its maximum crest acceleration, $\ddot{u}_{max}$, and (2) first natural period, T_0, due to a specified earthquake. Then by the use of the relationship (4) presented in Fig. 7, the maximum value of average acceleration (3) history, k_{max}, for any level of the specified sliding mass may be determined. (5) Entering the curves in Fig. 11(b) with the appropriate values of k_{max} and T_0, the permanent displacements can be determined for any value of yield acceleration associated with that particular sliding surface.

PLUS A PSEUDOSTATIC ANALYSIS TO GET k_y

It has been assumed earlier in this paper that in the majority of embankments, permanent deformations usually occur due to slip of a sliding mass on a horizontal failure plane. For those few instances where sliding might occur on an inclined failure plane it is of interest to determine the difference between the actual deformations and those calculated with the assumption of a horizontal failure plane having the same yield acceleration. A simple computation was made to investigate this condition using the analogy of a block on an inclined plane for a purely frictional material. It was found that for inclined failure planes with slope angles of 15° to the horizontal, the computed displacements were 10%–18% higher than those based on a horizontal plane assumption.

Application of Method to Embankment Subjected to 8-1/4 Magnitude Earthquake

To illustrate the use of the simplified procedure for evaluating earthquake-induced deformations, computations are presented herein for the 135-ft (41-m) high Chabot Dam, constructed of sandy clay and having the section shown in Fig. 12.

The shear wave velocity of the embankment was determined from a field investigation and the strain-dependent modulus and damping were determined from laboratory tests on undisturbed samples. The dam, located about 20 miles

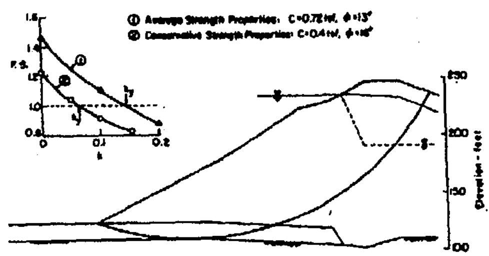


FIG. 12.—Yield Acceleration Values for Slide Mass Extending through Full Height of Embankment

(32 km) from the San Andreas fault, was shaken in 1906 by the magnitude 8-1/4 San Francisco earthquake with no significant deformations being noted; peak accelerations in the rock underlying the dam in this event are estimated to have been about 0.4 g. Accordingly the response of the embankment to ground accelerations representative of a magnitude 8-1/4 earthquake and having a maximum acceleration of 0.4 g was calculated by a finite element analysis. The maximum crest acceleration of the embankment, $\ddot{u}_{max}$, was calculated to be 0.57 g and the first natural period, T_0 = 0.99 sec. The maximum values of the calculated shear strain were less than 0.5%. On the basis of static undrained tests on the embankment material, the static failure strains ranged between 3%–8%, so that for the purposes of this analysis the cyclic yield strength of this material can be considered equal to its static undrained strength. From consolidated undrained tests on representative samples of the embankment material two interpretations were made for the strength of the material: (1) Based on an average of all the samples tested resulting in a cohesion value, c, of 0.72 tsf (69 kN/m^2) and a friction angle, ϕ, of 13°; and (2) a conservative interpretation, based on the minimum strength values with a cohesion of 0.4

tsf (38 kN/m^2) and a friction angle of 16°. Using these strength estimates, values of yield accelerations were calculated for a sliding mass extending through the full height of the embankment as shown in Fig. 12.

Considering the average relationship of $k_{max}/\ddot{u}_{max}$ with depth shown in Fig. 7, the ratio for a sliding mass extending through the full height of the embankment ($y/h \approx 0.95$) is 0.35, resulting in a maximum average acceleration, k_{max}, of $0.35 \times 0.57\ g = 0.2\ g$. From Fig. 12 the yield acceleration calculated for the average strength values is 0.14 g. Thus the parameters to be used in Fig. 11(b) to calculate the displacements for this particular sliding surface are as follows: magnitude ≈ 8-1/4; T_0 = 0.99 sec; k_{max} = 0.2; and $k_y/k_{max} = 0.14/0.20 = 0.7$. From Fig. 11($b$): $U/k_{max}\,g\,T_0$ = 0.013 sec, therefore, the displacement $U = 0.013 \times 0.2 \times 32.2 \times 0.99 = 0.08$ ft (0.02 m).

Using the most conservative value of $k_{max}/\ddot{u}_{max}$ shown in Fig. 7 of 0.47, the computed displacement would have been 0.58 ft (0.18 m). Similarly using the conservative strength parameters for the soil (giving k_y = 0.07) and the average curve for $k_{max}/\ddot{u}_{max}$ shown in Fig. 7, the computed displacement would have been 1.5 ft (0.45 m). All of these values are in reasonable accord with the observed performance of the dam during the 1906 earthquake.

The calculation was repeated for a sliding mass extending through half the depth of the embankment. The computed permanent displacements ranged between 0.02 ft–1.08 ft (0.006 m–0.33 m) indicating that the critical potential sliding mass in this case was that extending through the full height of the embankment.

Conclusions

A simple yet rational approach to the design of small embankments under earthquake loading has been described herein. The method is based on the concept of permanent deformations as proposed by Newmark (13) but modified to allow for the dynamic response of the embankment as proposed by Seed and Martin (26) and restricted in application to compacted clayey embankments and dry or dense cohesionless soils that experience very little reduction in strength due to cyclic loading. The method is an approximate one and involves a number of simplifying assumptions that may lead to somewhat conservative results.

On the basis of response computations for embankments subjected to different ground motion records, a relationship for the variation of induced average acceleration with embankment depth has been established. Design curves to estimate the permanent deformations for embankments, in the height range of 100 ft–200 ft (30 m–60 m), have been established based on equivalent linear finite element dynamic analyses for different magnitude earthquakes. The use of these curves requires a knowledge of the maximum crest acceleration and the natural period of an embankment due to a specified ground motion.

It should be noted that the design curves presented are based on averages of a range of results that exhibit some degree of scatter, and are derived from a limited number of cases. These curves should be updated and refined as analytical results for more embankments are obtained.

Finally, the method has been applied to an actual embankment that was subjected to a magnitude 8-1/4 earthquake at an epicentral distance of some 20 miles. Depending on the degree of conservatism in estimating the undrained strength of the material and in estimating the maximum accelerations in the embankment, the calculated deformations for this 135-ft (40-m) clayey embankment ranged between 0.1 ft–1.5 ft (0.3 m–0.46 m). These approximate displacement values are in good accord with the actual performance of the embankment during the earthquake.

Whereas the method described herein provides a rational approach to the design of embankments and offers a significant improvement over the conventional pseudostatic approach, the nature of the approximations involved requires that it be used with caution and good judgment especially in determining the soil characteristics of the embankment to which it may be applied.

For large embankments, for embankments where failure might result in loss of life or major damage and property loss, or where soil conditions cannot be determined with a significant degree of accuracy to warrant the use of the method, the more rigorous dynamic method of analysis described earlier might well provide a more satisfactory alternative for design purposes.

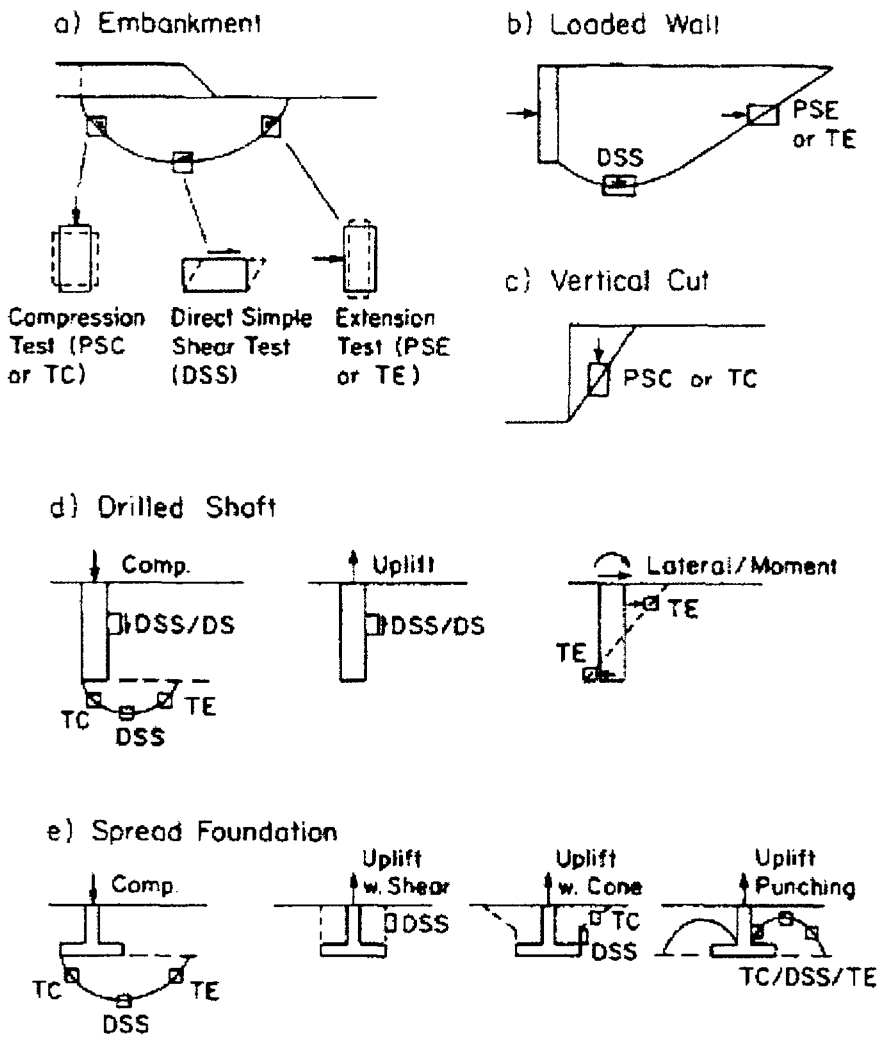
Acknowledgment

The study described in this paper was conducted under the sponsorship of the National Science Foundation (Grant ENV 75-21875). The support of the National Science Foundation is gratefully acknowledged.

Appendix.—References

1. Ambraseys, N. N., and Sarma, S. K., "The Response of Earth Dams to Strong Earthquakes," *Geotechnique*, London, England, Vol. 17, Sept., 1967, pp. 181–213.
2. Andersen, K. H., "Behavior of Clay Subjected to Undrained Cyclic Loading," *Proceedings*, Conference on Behavior of Off-Shore Structures, Trondheim, Norway, Vol. 1, 1976, pp. 392–403.
3. "A Review of Earthquake Resistant Design of Dams," *Bulletin 27*, International Commission on Large Dams, Mar., 1975.
4. Chopra, A. K., "Earthquake Effects on Dams," thesis presented to the University of California, at Berkeley, Calif., in 1966, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.
5. Clough, R. W., and Chopra, A. K., "Earthquake Stress Analysis in Earth Dams," *Journal of the Engineering Mechanics Division*, ASCE, Vol. 92, No. EM2, Proc. Paper 4793, Apr., 1966, pp. 197–212.
6. Idriss, I. M., et al., "QUAD-4, A Computer Program for Evaluating the Seismic Response of Soil Structures by Variable Damping Finite Elements," *Report No. EERC 73 16*, Earthquake Engineering Research Center, University of California, Berkeley, Calif., June, 1973.
7. Idriss, I. M., and Seed, H. B., "Response of Earth Banks During Earthquakes," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 93, No. SM3, Proc. Paper 5232, May, 1967, pp. 61–82.
8. Kovacs, W. D., Seed, H. B., and Idriss, I. M., "Studies of Seismic Response of Clay Banks," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 97, No. SM2, Proc. Paper 7878, Feb., 1971, pp. 441–455.
9. Lee, K. L., "Seismic Permanent Deformations in Earth Dams," *Report No. UCLA-ENG-7497*, School of Engineering and Applied Science, University of California at Los Angeles, Los Angeles, Calif., Dec., 1974.
10. Lee, K. L., and Seed, H. B., "Dynamic Strength of Anisotropically Consolidated Sand," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 93, No. SM5, Proc. Paper 5451, Sept., 1967, pp. 169–190.
11. Makdisi, F. I., and Seed, H. B., "A Simplified Procedure for Computing Maximum

Crest Acceleration and Natural Period for Embankments," *Report No. UCB/EERC-77/19*, Earthquake Engineering Research Center, University of California, Berkeley, Calif., 1977.
12. Martin, G. R., "The Response of Earth Dams to Earthquakes," thesis presented to the University of California, at Berkeley, Calif., in 1965, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.
13. Newmark, N. M., "Effects of Earthquakes on Dams and Embankments," *Geotechnique*, London, England, Vol. 5, No. 2, June, 1965.
14. Rahman, M. S., "Undrained Behavior of Saturated Normally Consolidated Clay Under Repeated Loading," thesis presented to the Indian Institute of Technology, at Kharahpur, India, in July, 1972, in partial fulfillment of the requirements for the degree of Master of Science.
15. Sangrey, D., Henkel, D., and Esrig, M., "The Effective Stress Response of a Saturated Clay Soil to Repeated Loading," *Canadian Geotechnical Journal*, Vol. 6, No. 3, Aug., 1969, pp. 241–252.
16. Sarma, S. K., "Seismic Stability of Earth Dams and Embankments," *Geotechnique*, London, England, Vol. 25, No. 4, Dec., 1975, pp. 743–761.
17. Schnabel, P. B., and Seed, H. B., "Accelerations in Rock for Earthquakes in the Western United States," *Report No. EERC 72-2*, Earthquake Engineering Research Center, University of California, Berkeley, Calif., July, 1972.
18. Seed, H. B., "A Method for Earthquake-Resistant Design of Earth Dams," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 92, No. SM1, Proc. Paper 4616, Jan., 1966, pp. 13–41.
19. Seed, H. B., "A Case Study of Seismic Instability and Terzaghi Foresight," Terzaghi Memorial Lecture Program, Bogazici University, Istanbul, Turkey, Aug. 14–16, 1973.
20. Seed, H. B., and Chan, C. K., "Clay Strength Under Earthquake Loading Conditions," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 92, No. SM2, Proc. Paper 4723, Mar., 1966, pp. 53–78.
21. Seed, H. B., and Idriss, I. M., "Rock Motion Accelerograms for High Magnitude Earthquakes," *Report No. EERC 67-7*, Earthquake Engineering Research Center, University of California, Berkeley, Calif., Apr., 1969.
22. Seed, H. B., and Idriss, I. M., "Influence of Soil Conditions on Ground Motions During Earthquakes," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 95, No. SM1, Proc. Paper 6347, Jan., 1969, pp. 99–137.
23. Seed, H. B., and Lee, K. L., "Liquefaction of Saturated Sands During Cyclic Loading," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 92, No. SM6, Proc. Paper 4972, Nov., 1966, pp. 105–134.
24. Seed, H. B., Lee, K. L., and Idriss, I. M., "Analysis of the Sheffield Dam Failure," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 95, No. SM6, Proc. Paper 6906, Nov., 1969, pp. 1453–1490.
25. Seed, H. B., et al., "Analysis of the Slides in the San Fernando Dams during the Earthquake of February 9, 1971," *Report No. EERC 73-2*, Earthquake Engineering Research Center, University of California, Berkeley, Calif., June, 1973.
26. Seed, H. B., and Martin, G. R., "The Seismic Coefficient in Earth Dam Design," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 92, No. SM3, Proc. Paper 4824, May, 1966, pp. 25–58.
27. Serff, N., et al., "Earthquake Induced Deformations of Earth Dams," *Report No. EERC 76-4*, Earthquake Engineering Research Center, University of California, Berkeley, Calif., Sept., 1976.
28. Terzaghi, K., "Mechanisms of Landslides," The Geological Society of America, Engineering Geology (Berkey) Volume, Nov., 1950.
29. Thiers, G. R., and Seed, H. B., "Strength and Stress-Strain Characteristics of Clays Subjected to Seismic Loads," *ASTM STP 450*, Symposium on Vibration Effects of Earthquakes on Soils and Foundations, American Society for Testing and Materials, 1969, pp. 3–56.



Note: Plane strain tests (PSC/PSE) used for long features
Triaxial tests (TC/TE) used for near symmetrical features
Direct shear (DS) normally substituted for DDS to evaluate $\bar{\phi}$

Figure 4-6. Relevance of Laboratory Strength Tests to Field Conditions

Figure 9. Laboratory Tests Commonly Performed to Determine the Anisotropy of the Soil (after EPRI 1990).

State's
Exhibit 103

DESIGN AND PERFORMANCE OF THE FOUNDATION STABILIZATION TREATMENTS FOR THE RECONSTRUCTION OF INTERSTATE 15 IN SALT LAKE CITY, UTAH

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ABSTRACT: This paper summarizes the design and initial performance of the foundation stabilization treatments adopted for the reconstruction of Interstate 15 in Salt Lake City, Utah that began in 1997. The roadway designs required extensive high fills over soft foundation soils to raise and widen existing embankments and to construct new embankments. The foundation treatments included prefabricated vertical drains, surcharge fills, high-strength geotextile reinforcement, stability berms, staged embankment construction, light weight fills, and lime cement columns in order to improve foundation stability, avoid damage to existing structures, and to reduce postconstruction pavement settlements. The relative success of the design is evaluated from geotechnical instrumentation results obtained during the first phase of the reconstruction in 1997 and 1998.

State's
Exhibit 104

Ladd (1989), as presented in Stewart et al. (1994) and shown in Figure 25, was used to estimate the design surcharge heights using the "average" relationship. Special testing was completed for the final design by Ng (1998) at MIT to develop the specific design relationship for Lake Bonneville Deposits presented in Figure 26. These data show a much larger reduction in $C_\alpha'/C_\alpha(NC)$ at the lower surcharge levels than reported by Ladd (1989) for other cohesive soils, as illustrated in Figure 25. Figure 27 combines the MIT data from both Figures 25 and 26 in the semi-log format proposed by Ng (1998). The maximum reduction curve shown in Figure 27 for $C_\alpha'/C_\alpha(NC)$ versus the adjusted amount of surcharge (AAOS) was selected for final design. Without the special testing at MIT, the mean reduction line in Figure 25 would have been used for design resulting in an increased thickness of surcharge fills. The mean line for t_s/t_r vs AAOS in Figure 27 was used in final design.

6 STABILITY ANALYSES

Stability was a major design concern at most locations along the alignment in Figure 1. Large surcharge fills were needed at most locations, further increasing the stability problems. Many loading conditions involved the widening of existing embankments with staged filling, and vertical drains, as shown in Figure 2. The 35% design reports also identified severe stability problems along the portion of the project shown in Figure 1. Examples of sections requiring stability analyses are illustrated in Figure 28.

The loading conditions associated with staged embankment construction required prediction of both the initial undrained shear strength (s_u) profile and increases due to consolidation. The SHANSEP method (Ladd and Foott 1974) was used to calculate these s_u profiles using the following relationship:

$$s_u = \sigma'_{vo} (S)(OCR)^m \qquad (4)$$

where: $\sigma'_{vc} = \sigma'_{vo}$ for virgin ground (stage I) and the calculated vertical effective stress with consolidation (under embankment for subsequent stages), $S = s_u/\sigma'_{vc}$ at OCR = 1, OCR = the overconsolidation ratio (σ'_p/σ'_{vo}), and m = an experimental exponent.

The SHANSEP parameters were developed for undrained shear in plane strain compression (PSC) and extension (PSE) and direct simple shear (DSS). These values were derived from CK_oU triaxial compression and extension, and DSS tests run at MIT using the SHANSEP reconsolidation technique to reduce sample disturbance effects and included the following "corrections" described by Ladd (1991): 1) increase triaxial strengths for plane strain (Section 4.6); 2) decrease peak strength to account for strain compatibility (Section 4.9); and 3) define s_u as the shear stress on the failure plane (i.e. $\tau = 0.5(\sigma_1 - \sigma_3)\cos\phi'$). The values of S and m selected for the cohesive layers of the Lake Bonneville Deposits are:

Shear in PSC	$S_c = 0.3$	$m_c = 0.8$
Shear in DSS	$S_d = 0.24$	$m_d = 0.8$
Shear in PSE	$S_e = 0.18$	$m_e = 0.8$

The design initial undrained strength profiles for a wide variety of site conditions were calculated from the effective overburden stress, which was adjusted for artesian pressures with depth, and values of the preconsolidation stress estimated with the approach described in Section 5.1. Beneath the existing embankments, the present, higher, undrained strengths were obtained using the embankment height, with appropriate influence factors, to calculate the change in σ'_{vc} for addition to the free-field effective overburden stress. The same approach was used for staged embankment loading to predict the improved strengths beneath the new embankment after the consolidation period, except that the values of S_c and S_d were reduced by 10 % since no aging effects would occur during the consolidation period following staged loading.

Figures 29a and 29b illustrate the development of undrained strengths for different sections of the embankment for: a) a wide extension of the embankment (virgin ground initial stress conditions) and b) a narrow widening of the existing embankment where improved strengths beneath the existing embankments increase the strengths. By separating the strengths in this manner, undrained strength profiles were calculated for a wide range of loading and embankment configurations.

The instrumentation observations during the first phase of construction in 1997 indicated that the alluvial soils encountered to depths of 5 to 6 m below original grade that were penetrated with prefabricated vertical drains developed limited, if any, excess pore water pressure. Hence stability assessments for the second phase of construction were made with strengths computed for drained shear in the upper 6 m of the natural soils, although this may overestimate the actual resistance during a rapid failure.

Reinforcement of the embankments with high-strength geotextile and staged embankment construction with prefabricated vertical drains were the primary methods to improve stability conditions. High-strength geotextile was used extensively in the 2400S area of the alignment where the embankments were constructed over virgin ground conditions and where embankment heights up to 18 m were needed. Representative stability calculations from the 2400S area, based on undrained loading, are shown in Figure 30 for 2H to 1V slopes with staged filling. Without reinforcement, an initial embankment height of 7 m was calculated at a factor of safety of 1.3, increasing to a height of 10 m for Stage 2 and only 10.5 m for Stage 3. The declining incremental increase in embankment height with successive stages was a significant factor in the geotechnical design that limited the usefulness of staged construction. Hence, installation of high-strength geotextile (GT No. 1) near the bottom of the embankment was used to improve global stability. Figure shows that three layers of reinforcement gives design heights about 20 percent higher than unreinforced embankments.

Global stability calculations were made using the Modified Bishop method with the UTEXAS3 or SLOPE/W programs. In many instances stability was a limiting condition for construction of walls and embankments. The UTEXAS3 program described by Wright (1991) was used to evaluate the reinforcement effects of geotextile in the global stability calculations. The geotextile provides an additional resisting moment at the intersection with the critical failure surface. The high-strength geotextile also helps the embankment to act as a unit that can produce a squeezing type failure of the foundation soils below the reinforced mass. Bonaparte and Christopher (1987) describe the methodology used to assess this type of failure for embankments with geotextile reinforcement.

STATE OF UTAH'S PREFACE TO TESTIMONY OF STEVEN F. BARTLETT AND JAMES K. MITCHELL ON CONTENTION UTAH L/QQ - Soil Cement

I. **Major Points**

A. Unique application of adding cement to soil to provide additional seismic sliding resistance and stability by buttressing shallowly embedded foundations from strong ground motions.

B. No prior precedent for PFS's proposed concept for use of soil-cement.

C. No site specific analyses and testing have been done to verify that the design concept will perform as intended.

D. No analysis has been made of the impact to the critical underlying native soils from the impact of construction and placement of cement-treated soil.

E. PFS's proposed post license soil cement program will not prove the design concept and there will be an inadequate and arbitrary basis for a licensing decision.

II. **No Direct Precedent for PFS's soil-cement program**

A. Cited examples of precedent in the SAR are not analogous to the PFS case.

1. Koeberg, South Africa case and the Houston case involve potentially liquefiable soils.

a. Soils at PFS are plastic fine grained materials that are not susceptible to liquefaction.

b. State witness, Dr. Mitchell, was a consultant on South African project which involved a 24 meters deep excavation, removal of 8 meter thick potentially liquefiable layer of sand, which was mixed with cement, replaced and recompacted.

2. Other examples of soil cement used in seismic design, such as deep soil mixing, are not applicable to the PFS site.

III. **Soil cement/cement-treated will not perform as intended**

A. Compared to the compressive strength of reinforced concrete (3,000-4,000 psi), cement-treated soil under the pads (~1% cement, 40 psi) and soil cement around the pads and CTB (~6% cement, 250 psi) are very weak in tension.

B. CEC performed soil-structure interaction (SSI) to evaluate the dynamic stresses on the concrete pads but not on the soil cement or cement-treated soil.

1. No SSI analysis to determine whether the soil cement and cement-treated soil can resist compressional, shear, bending, torsional and tensile stresses from DBE.

a. PFS has not analyzed magnitude or orientation of these stresses and how these forces will impact seismic performance. This is important because given low tensile strength of soil cement and cement-treated soil, even low tensile stresses can cause cracking.

C. Cracking from non-dynamic forces such as delaminating or debonding at various interfaces, shrinkage, differential settlement, frost, expansion or vehicle loads.

1. Shrinkage cracking (from curing and drying of soil cement) deleterious to seismic design.

a. Vertical/subvertical cracks that develop in soil cement/cement treated soil

(1) May cause loss of tensile capacity along the surface of the crack;

(2) Loss of tensile capacity is deleterious when the cement-treated soil or soil cement mat has to resist dynamic tensile stresses from strong ground motions.

(3) This loss of tensile capacity in turn degrades mat's capacity to act as an integral mat and resist out-of-phase motion between (a) individual pads and (b) CTB concrete mat foundation and perimeter soil cement mat.

b. Cracking can also be caused by differential settlement around perimeter of CTB & pads.

c. Consequences of cracking or interaction: loss of passive earth pressure (buttressing) and loss of treated soil's ability to transfer shear stresses to underlying native soils.

(1) For the CTB, PFS relies on passive earth pressure to resist foundation sliding.

(2) Loss of ability to transfer shear stresses will reduce factor of safety against sliding or

if magnitude of the loss is large it could lead to sliding.

D. No rational assessment for PFS's assumption that soil cement/cement-treated soil will act as an integral mat to keep each individual pad in place and in-phase with adjacent pads during strong ground motion.

IV. Requirements for soil cement/cement-treated soil

A. Target compressive strength in sliding calcs: 40 psi (cement treated soil) 250 psi (soil cement)

1. For the cask transporter, soil cement between pads target strength: 250 psi.

B. Constraints on cement-treated soil under the pads based on Holtec's cask tipover analysis.

a. Modulus of elasticity of the cement-treated soil beneath the pads has to be $\leq$75,000 psi

b. Depth of cement pad 3 feet and of cement-treated soil under pads 1-2 feet.

V. Disturbance of upper Bonneville sediments (native soils)

A. Engineering properties of native clays critical - PFS relies on their shear strength of provide resistance to sliding. Substantial decrease in their shear strength could result from any disturbance or remolding.

B. Construction activities have significant potential to disturb/remold the clays (native soils).

C. Cement cap (storage pad) can increase the moisture content of the underlying native soils.

1. More moisture = a decrease in shear strength of the native soils.

VI. PFS's Soil Cement Program

A. Very few tests performed to date; problems with last set of tests (durability).

B. PFS soil cement program now on hold; almost all testing will be conducted post license.

C. Even PFS admits that it needs testing to prove its design.

D. Even if PFS completes all tests, there will be no proof of design concept.

1. There could be cracking of cement-treated soil under the pads and separation of soil cement around the pads and the CTB.

VII. Conclusion: PFS has not shown that the use of soil cement and cement-treated soil will provide an acceptable seismic design for storage of spent nuclear fuel at the PFS site.

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	)	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel Storage Installation)	)	April 1, 2002

STATE OF UTAH TESTIMONY OF DR. STEVEN F. BARTLETT AND DR. JAMES K. MITCHELL ON UNIFIED CONTENTION UTAH L/QQ (Soil cement)

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

A. 1: (SFB) My name is Dr. Steven F. Bartlett. I am an Assistant Professor in the Civil and Environmental Engineering Department of the University of Utah, where I teach undergraduate and graduate geotechnical engineering courses and conduct research. I hold a B.S. degree in Geology from Brigham Young University, a Ph.D. in Civil Engineering from Brigham Young University and I am a licensed professional engineer in the State of Utah.

My qualifications are described in my soils testimony, which is being filed concurrently with this prefiled testimony. Relevant to this testimony, my tenure at the Utah Department of Transportation and Woodward-Clyde Consultants in Salt Lake City have given me a background knowledge and understanding of local soil conditions, especially the upper Lake Bonneville sediments. I have also been involved in the design and performance monitoring that used lime-cement column stabilization underneath a mechanically stabilized earth wall for the I-15 Reconstruction Project. My curriculum vitae is included with my soils testimony as State's Exh. 92.

Q. 2: Dr. Bartlett, do you consider it necessary to present testimony with another witness?

A. 2: (SFB) Yes. Dr. James K. Mitchell has expertise specific to soil cement. His testimony will overlap my testimony especially with respect to the effect soil cement may have on native soils. It would be expedient for the Board to hear our testimony together.

Q. 3: Please state your name, affiliation, and qualifications.

A. 3: (JKM) My name is Dr. James K. Mitchell. I hold a Sc.D. in civil engineering earned in 1956 from the Massachusetts Institute of Technology. Presently I am a University

Distinguished Professor Emeritus at Virginia Tech and Professor Emeritus at the University of California at Berkeley. I serve as an individual consultant on geotechnical problems and earthwork projects of many types, particularly soil stabilization, ground improvement for seismic risk mitigation, earthwork construction, and environmental geotechnology, to numerous national and international governmental and private organizations. My curriculum vitae listing my qualifications, experience, and training is included as State's Exhibit 105.

I have more than 40 years' experience in the field of geotechnical engineering. I was on the faculty of the University of California, Berkeley, Department of Civil Engineering for more than 35 years, serving as Department Chair for five years. I developed and taught graduate courses in soil behavior, soil and site improvement, and foundation engineering as part of the Geotechnical Engineering Program within the Civil Engineering Department. At the same time, I was Research Engineer in the Institute of Transportation Studies and in the Earthquake Engineering Research Center. Since 1994, I served on the faculty of Virginia Tech, Via Department of Civil and Environmental Engineering, and was appointed University Distinguished Professor in 1996 and University Distinguished Professor, Emeritus, in 1999.

My primary research activities focused on experimental and analytical studies of soil behavior related to geotechnical problems, admixture stabilization of soils, soil improvement and ground reinforcement, physico-chemical phenomena in soils, the stress-strain time behavior of soils, in-situ measurement of soil properties, and mitigation of ground failure risk during earthquakes. I have authored more than 350 publications, including two editions of the graduate level text and reference, "Fundamentals of Soil Behavior," and several state-of-the-art papers and guidance documents on soil stabilization, ground improvement, and earth reinforcement.

Some of my recent and currently active projects include the evaluation of seismic stabilities and design of liquefaction mitigation options for Success Dam in California (U.S. Army Corps of Engineers) and Pineview and Deer Creek Dams in Utah (U.S. Bureau of Reclamation); ground improvement aspects of the Port of Oakland Wharf and Embankment Strengthening Program (Harding Lawson Associates); ground improvement and fill stabilization for the proposed San Francisco Airport Expansion (Fugro West); design review – ground improvement for the I-95/Rt.1 Interchange section of the Woodrow Wilson Bridge replacement project (Haley & Aldrich, Virginia Geotechnical Services, URS, HNTB); and as a member of the Peer Review Panel for the Seismic Vulnerability Study of the Bay Area Rapid Transit System in California.

I am licensed as a Civil Engineer and as a Geotechnical Engineer in California, and as a Professional Engineer in Virginia. I am a Fellow and Honorary Member of the American Society of Civil Engineers, and have served as an officer of the Geotechnical Engineering Division of ASCE; the United States National Committee for the International

Society for Soil Mechanics and Foundation Engineering; the ASCE Committee on Soil Properties, the Committee on Placement and Improvement of Soils; the San Francisco Section of ASCE and the California State Council of ASCE; the Transportation Research Board Committee on Physico-Chemical Phenomena in Soils; the Geotechnical Board of the U.S. National Research Council; the International Society for Soil Mechanics and Foundation Engineering. I recently completed service as Vice Chair of an NRC study committee for development of science needs for remediation of contaminated Department of Energy weapons sites and as a member of an NRC study committee to advise the Department of Energy on Remediation Science and Technology for the Hanford Site. I presently serve as Chair of a National Academies panel to develop recommendations for peer review of U.S. Army Corps of Engineers civil works projects.

Specifically relevant to soil cement are my many years of research on the properties of cement stabilized soils and the use of soil cement in pavement structures, involvement as a consultant on the Koeberg nuclear power plant project in South Africa, and my current work involving deep soil mixing.

Q. 4: Dr. Mitchell, do you consider it necessary to present testimony with another witness?

A. 4: (JKM) Yes. Dr. Steven Bartlett's expertise in native soils in Utah will complement my testimony. In addition, he has had more involvement than I have in the overall review of PFS's analyses relating to soils and the dynamic forces imparted to foundations and soils. Together, we can better inform the Board on PFS's proposed use of soil cement than if we were to testify independently.

Q. 5: What is the purpose of your testimony?

A. 5: (SFB, JKM) The purpose of our testimony is to explain the basis for our professional opinion that (1) PFS's proposal to use soil-cement and cement-treated soil to provide additional seismic sliding resistance and stability to shallowly embedded foundations subjected to intense strong ground motion is a new and unique application of this technology; (2) to our knowledge, there is no prior precedent for PFS's proposed use of this technology; (3) site-specific analyses and testing is required to verify the design at the PFS site to ensure that the soil cement and cement-treated soil will perform their intended functions during earthquake shaking and that target performance requirements are met for cask drop and tipover scenarios; (4) the potential impact of construction and placement of the soil cement and cement-treated soil on the underlying native soils has not been addressed; and (5) PFS's proposal to conduct a soil cement testing program after, rather than before, it obtains a license will not prove the design concept that will form the basis of a licensing decision.

Q. 6: What has been your involvement in reviewing and analyzing PFS's intended use of soil cement and cement-treated soil? NEW

A. 6: (SFB) I have been assisting the State since 1999 and have reviewed PFS's sliding and stability calculation both prior to PFS's intended use of soil cement and also where, through design creep, PFS has expanded its use of soil cement and cement-treated soils. I assisted and gave technical support to the State in filing Contention Utah QQ and the two modifications thereto. I am familiar with sections of PFS's Safety Analysis Report ("SAR") and calculation packages with respect to PFS's characterization of soils, the cone penetrometer testing, PFS's stability analyses and its seismic exemption request. Some of these topics are described in my soils and dynamic analysis testimonies filed concurrently with this testimony.

(JKM) I began assisting the State shortly before the State filed Contention Utah QQ. I provided technical support for filing that contention. My role is generally limited to review of PFS's most recent proposal for use of soil cement and cement-treated soil.

Q. 7: Please describe PFS's intended use of soil cement and cement-treated soil at the proposed Skull Valley ISFSI site?

A. 7: (SFB) PFS states that it intends to use soil cement around the Canister Transfer Building ("CTB") and around the storage pads. Under the storage pads, PFS will use a weaker cement mix, a cement-treated soil.

The placement of soil cement around the perimeter of the foundation for the CTB is intended to provide additional resistance against sliding during the design basis earthquake by acting as a buttress. Without the additional resistance provided by the soil cement around the CTB, the Applicant has calculated that sliding of the CTB is possible (Calc. G(B)-13-4). Thus, the concept of using soil cement as buttress for the CTB has become an integral part of the seismic design of the CTB design.

The placement of cement-treated soil underneath the storage pads is intended to act as an "engineered mechanism" to transfer inertial forces of the casks and pads to the underlying upper Lake Bonneville sediments in order to prevent sliding. SAR, p. 2.6-61. Shear stresses are intended to be transferred through the approximately 2-ft thick cement-treated soil layer to the underlying silty-clay/clayey-silt. The Applicant also implies that additional sliding resistance will be provided by the continuous layer of soil cement between the pads (SAR, p. 2.6-61). Like the CTB, the concept of using cement-treated soil underneath the pads and soil cement between the pads has become an integral part of the seismic design of the storage pads.

The soil cement between the pads is also intended to provide a stabilized base for the support of the cask transport vehicle. SAR, p. 2.6-67d.

Q. 8: Has PFS conducted tests and analyses that are necessary to determine whether soil cement will provide additional resistance against sliding and whether cement-treated soil will act as an "engineered mechanism" in transferring shear stresses to the native soils?

A. 8: (SFB, JKM) No. PFS has conducted a few tests, which we describe later in our testimony. Basically, PFS has decided to wait until after it obtains a license to conduct most of the testing and analyses.

There are only two documents that describe PFS's soil cement program: (1) SAR 2.6-108 through -121 (Rev. 22), included as State's Exhibit 106, and (2) Engineering Services Scope of Work for Laboratory Testing of Soil-Cement Mixes between Stone and Webster and Applied Geotechnical Engineering Consultants, Inc. ("AGEC"), ESSOW No. 05995.02-G010 (Rev. 0), dated January 21, 2001, included as State's Exhibit 107.[1]

Those two documents describe what PFS intends to do in the future. We do not understand how PFS can go forward with its seismic design not knowing whether soil cement and cement-treated will perform its intended seismic function. We see no practical reason why PFS should not perform testing and analyses now rather than at some future date. Some of the questions – but not all of them – we raise here would be resolved through such testing and analyses. Also, if in the future PFS finds that soil cement and cement-treated soil will not support PFS's seismic design, then the licensing basis for approving the PFS facility design will be invalid.

Q. 9: Dr. Mitchell, do you consider there to be any direct precedent for PFS's soil-cement program?

A. 9: (JKM) For pavement structures and as a structural fill – yes; as a restraining buttress and for development of sliding resistance – no.

Q. 10: What is the basis of your opinion?

A. 10: (JKM) Over my 40 year career, I have been involved with or had an academic interest in numerous projects that have used cement to increase certain properties of soils. The use of soil cement for pavement bases and sub-bases goes back to the early 1900s and today it is widely used as a strengthening base for pavement structures. Starting in the late 1950s soil cement has been used for hydraulic structures such as slope protection on dam faces or reservoirs and for canal linings.

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[1] The State obtained a copy of the ESSOW under a PFS confidentiality agreement; PFS claims that the methodology that may be contained in the ESSOW still remains confidential. As a precaution, the State is filing Exhibit 107 as a proprietary filing but in doing so the State does not agree that the document is confidential.

More recently, soil cement has been used as structural fill in seismic areas and for constructing roller-compacted concrete to build dikes and dams. The latest development in the use of soil cement is deep soil mixing.

Q. 11: Does the use of soil cement as a strengthening base for pavements and for hydraulic structures provide a precedent for PFS?

A. 11: (JKM) Not as regards the proposed development of sliding resistance and a buttressing effect.

Q. 12: Are there examples of using soil cement in seismic design?

A. 12: (JKM) Yes. But none of the cases apply to PFS's intended use. The one application I am most familiar with is in Koeberg, South Africa - one of the cases PFS cites in the SAR at 2.6-113 (Rev. 22), State's Exh. 106.

During the late 1970s and early 1980s, I was involved as a consultant on the soil cement issues at the Koeberg nuclear power project located in the coastal area of Cape Town, South Africa. The project required a large excavation, approximately 24 meters deep, to remove an eight meter thick potentially liquefiable layer of saturated loose sand. The sand was mixed with cement, then replaced and recompacted.

Q. 13: Why is the South Africa case not analogous to the PFS case?

A. 13: (JKM) The Koeberg case is not analogous because the soils there were loose, saturated sands. The soils at PFS are plastic, fine grained, cohesive materials. At Koeberg the purpose was to eliminate the potential for liquefaction of the loose sand beneath the reactor building under seismic loading. The fine-grained soils at the PFS site are not liquefiable, and the purposes of the soil cement and cement-treated soil are to provide sliding resistance and buttressing, as stated above.

Q. 14: Are there other examples of soil cement used in seismic design?

A. 14: (JKM) Yes, but again the application is not really relevant to the PFS site. The latest use of soil cement for seismic design is in deep soil mixing. In this application, mix-in-place columns and walls extend down as much as a hundred feet below the ground surface for both support of structures and excavations and for containment of potentially liquefiable soils.

Q. 15: Is deep soil mixing analogous to the PFS case?

A 15: (JKM) No. Deep soil mixing applications are not at all like the proposed PFS use of soil cement.

Q. 16: What is the difference between soil cement and cement-treated soil and why is the difference important?

A. 16: (JKM) Cement-treated soil may contain any amount of cement. To be a soil cement requires that the cement content and compaction conditions be sufficient to attain minimum durability standards as measured by American Society for Testing and Materials ("ASTM") wet-dry and freeze-thaw tests. More cement is needed as the fines content in the soil to be treated increases. The strength of soil cement generally decreases as soil plasticity increases. At treatment levels less than those needed to produce a soil cement, the durability may be inadequate under severe exposure conditions, such as at the PFS site, to prevent degradation of the material over time.

Q. 17: Specific to the PFS site, approximately how much cement is needed to create soil cement and cement-treated soil?

A. 17: (JKM, SFB) The Applicant has not submitted the design of the soil cement and cement-treated soil for the PFS site, so this has not been determined. However, the SAR (p.2.6-67c), State's Exh. 106, implies that about 1 percent cement will be required to create cement-treated soil and about 6 percent will be required to create soil cement in order to meet the target compressive strengths of 40 and 250 psi, respectively. It should be noted that by itself, attainment of a designated compressive strength cannot guarantee a material to be a soil cement. Durability testing is required for this purpose.

Q. 18: The term soil cement seems to imply a fairly strong material. How does the compressive strength of 250 psi soil cement compare with the compressive strength of concrete?

A. 18: (JKM, SFB) Concrete is much stronger. It has typical compressive strengths of at least 3000 to 4000 psi. Also, the concrete that PFS plans to use for the cask storage pads and CTB mat foundation has steel reinforcement so that it can withstand tensile as well as compressive forces.

Q. 19: Why is it important to have reinforcing steel to resist tensile forces in reinforced concrete design?

A. 19: (SFB) Concrete is relatively weak in tension and steel has high tensile capacity. Thus, the reinforcement allows the pad or mat to resist tensile stresses created by bending and torsion of the foundation during the design basis earthquake.

Q. 20: Were the concrete storage pads designed to resist tensile and bending stresses?

A. 20: (SFB) Yes, the storage pads were analyzed and designed for dynamic loading conditions using a soil-structure analysis that was performed by International Civil Engineering Consultants Inc. (Calc. G(PO17)-2).

Q. 21: Does a similar analysis exist to evaluate the dynamic stresses developed in the soil cement and cement-treated soil?

A. 21: (SFB) No.

Q. 22: In your opinion, is a similar calculation necessary to assess the feasibility of the proposed treatment and if so, why?

A. 22: (SFB) Yes. The Applicant has assumed that the soil cement and cement-treated soil will act as an integral mat, thereby keeping each individual pad in place and in-phase with the other adjacent pads during strong ground motion (SAR, pp. 2.6-61 and 62). The Applicant has not considered the potential for out-of-phase motion between pads in the longitudinal direction and the consequences of this out-of-phase motion. However, to act as an integral mat, the soil cement and cement-treated soil mat must resist compressional, shear, bending, torsional and tensile stresses induced by the design basis earthquake both underneath the pads and between the pads. The Applicant has not performed soil-structure interaction analysis to evaluate the magnitude and orientation of these stresses in the mat and how these forces will impact the seismic performance. The magnitude of bending, torsional and tensile stresses developed in the mat could be important because of the very low tensile strength of the soil cement and cement-treated soil. The tensile strength of these materials is typically only about a fifth to a third of the unconfined compressive strength. Thus, even rather low tensile stresses can cause cracking. The Applicant has not calculated the magnitude and orientation of these stresses; thus a rational assessment cannot be made of the seismic performance of the proposed cement treatment.

Q. 23: In your opinion, are there other possible mechanisms that may cause cracking of the soil cement and cement-treated soil beside the dynamic forces?

A. 23: (SFB, JKM) Yes. Other potential mechanisms for cracking of the soil cement and cement-treated soil may include: (1) delamination or debonding along a soil cement lift interface or an interface with the concrete pad or the native soil during a seismic event; (2) shrinkage cracking during curing and drying; (3) settlement cracking resulting from differential settlement at the perimeter of the pads and CTB mat foundation; (4) frost penetration and expansion cracking; and (5) cracking or overstressing due to vehicle loads (e.g., canister transport vehicle).

Q. 24: Of these possible mechanisms, which one would seem to be of most concern?

A. 24: (SFB, JKM) Of most concern is shrinkage cracking of the soil cement between and around the pads and of the soil cement surrounding the CTB. Shrinkage cracks form during the process of curing and aging of soil cement. These are relatively thin generally vertical cracks to subvertical cracks that will develop in the soil cement. From a seismic performance standpoint, the real issue is not thickness of the crack, but its potential for continuity. If these cracks are somewhat continuous, then the tensile resistance has been completely lost along the surface of the crack. This loss of tensile capacity in the mat is extremely deleterious when the mat has to resist dynamic tensile stresses. Lost of tensile capacity will in turn impact the mat's capacity to act as an integral mat and resist out-of-phase motion between individual pads or out-of-phase motion between the CTB concrete mat foundation and the perimeter soil cement mat. Such out-of-phase motion will introduce inertial interaction as discussed in the dynamic analysis testimony by Drs. Farhang Ostadan and Steven Bartlett.

Q. 25: What might be other consequences of cracking and inertial interaction?

A. 25: (SFB) If the cracking or interaction is significant, then there can be a loss of the buttress effect (*i.e.*, passive earth pressure) that is relied upon by the Applicant to resist sliding of the CTB foundation. Also, there can be a reduction or loss the cement-treated soil's ability to transfer shear stresses to the underlying upper Lake Bonneville sediments. These losses, depending on their magnitude, will reduce the factor of safety against sliding, or if large enough, lead to sliding.

In addition, the cracks would provide a pathway for ingress of water through the soil cement between the pads and around the CTB. This water could cause a strength reduction in the underlying Bonneville clay.

Q. 26: In addition to shrinkage cracks, are there other mechanism that may lead to cracking?

A. 26: (SFB, JKM) Differential settlement around the perimeter of the CTB and pads, as well as beneath the pads may be important. The Applicant has estimated about 2 inches of total settlement of the pads (SAR, p. 2.6-50) and 3 inches of total settlement for the CTB. It is anticipated that much of this settlement will be distributed around the perimeter of the pads and CTB due to the abrupt change in vertical static loading conditions between relatively heavily loaded foundations (about 1.5 to 2 kip per square foot) and the adjacent unloaded perimeter area. Also, it is important to keep in mind that the most compressible layer (*i.e.*, the upper Lake Bonneville sediments) lies just below the foundations.

Q. 27: Beyond the target compressive strength of 40 and 250 psi for cement-treated soil and soil cement, respectively, identified by PFS in the earthquake sliding calculations, has PFS identified any other requirements for the cement-treated soil and soil cement?

A. 27: (SFB, JKM) It has. The soil cement between the pads must have a target strength of 250 psi to provide a good subbase for the cask transporter (SAR p. 2.6-67d). The cement-treated soil beneath the pads must have a Young's modulus of 75,000 psi, or less.

Q. 28: What is the purpose of limiting Young's modulus to 75,000 psi?

A. 28: (SFB) In the drop/tipover analysis of the casks (*PFSF Site-Specific HI-STORM Drop/Tipover Analyses*, Rev. 0 and Rev. 1, Holtec Report No. HI-2012653, Apr. 3, and May 7, 2001 respectively), Holtec places constraints on the thickness and modulus of elasticity (*i.e.*, Young's modulus) of the cement-treated soil. The cement-treated soil is limited to a maximum thickness of 2 feet and Young's modulus is limited to a maximum value of 75,000 psi. These constraints are placed on the cement-treated soil in an attempt to limit the decelerations from a hypothetical cask tipover event or end drop accident. The Holtec calculation shows that there is a very small margin against the deceleration limit. If the Young's modulus exceeds 75,000 psi, then the deceleration limit is likely to be exceeded. The Stone and Webster stability analysis of the casks identifies the 75,000 psi as the static Young's modulus of the cement-treated soil. Dr. Ostadan has testified, in the Dynamic Analysis testimony, that the use of the static Young's modulus to analyze dynamic impact is not appropriate for the cask drop/tipover scenario. Furthermore, the Geomatrix calculation for development of ground motion, soil springs and damping effectively assigns a much higher modulus to the cement-treated soil.

(SFB, JKM) The Applicant has not provided any site-specific test data that demonstrate this rather low modulus can be achieved for a cement-treated soil with a minimum compressive strength of 40 psi. There is not very much published test data for these low modulus values. Further, the cement content and the placement conditions are tremendously important in determining the strength and stiffness properties of the cement-treated soil. In sum, whether or not PFS can achieve a Young's modulus of 75,000 psi or less, while meeting the minimum compressive strength requirement of 40 psi, depends on the quantity of cement that is used, the site soil, and the placement conditions (water content and density).

Q. 29: To your knowledge, who is working on the PFS soil-cement program?

A. 29: (SFB) From deposition testimony, it appears that Mr. Paul Trudeau of Stone & Webster was primarily responsible for authoring the description of PFS's soil-cement program in SAR 2.6-108 through -121 (Rev 22). Trudeau Tr.[2] at 18. Mr. Trudeau also developed the ESSOW No. 05995.02-G010 for the Laboratory Testing of Soil-Cement Mixes between Stone & Webster and AGEC. Id. at 54-55. AGEC has conducted a few tests and reported the results to Mr. Trudeau but most of the AGEC testing program is on hold for now. Trudeau Tr. at 67, 72-73.

(SFB, JKM) PFS may retain Dr. Anwar Wissa to assist it with its soil-cement program but as of the date of his deposition on March 15, 2002, there was no formal agreement between Dr. Wissa and PFS. Wissa Tr.[3] at 42-44; Trudeau Tr. at 89, 110, State's Exh. 108.

Q. 30: How will PFS construct the soil cement in its foundation system?

A. 30: (SFB, JKM) From the deposition testimony it appears that PFS has not yet developed a plan for the specific construction techniques that will be employed in excavating the eolian silts and mixing them soil cement and replacing them. State's Exh. 109, Wissa Tr. at 15-34; State's Exh. 108, Trudeau Tr. at 91-92. Irrespective of the methods that are used, it is important that the native soils upon which the soil cement will be placed not be disturbed as this would likely lead to loss of subgrade support and increased post-construction settlement. If PFS chooses to haul eolian silt off site to a central plant for mixing, the time between mixing the water at the central plant and final compaction could affect the properties of the soil cement. Wissa Tr. at 24.

Q. 31: What effect would there be from potential disturbance or remolding of the native clays?

A. 31: (SFB) As I described in my soils testimony, the engineering properties of the native clays – *i.e.*, upper Lake Bonneville sediments – are very important because PFS relies on the shear strength of this layer to provide resistance to sliding. Any disturbance or remolding of these clays could substantially decrease their shear strength.

During his deposition testimony, Mr. Trudeau acknowledged that cohesion available in the upper Lake Bonneville sediments is required as part of the design of the pads and that construction equipment and techniques have the opportunity to destroy the surface of the subgrade if PFS is not careful in protecting those soils. State's Exh. 108, Trudeau Tr. at 96.

The SAR at 2.6-108 (State's Exh. 106) describes the following regarding the

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[2] Excerpts from the deposition transcript ("Tr.") of Mr. Paul Trudeau (March 6, 2002) are included as State's Exhibit 108.

[3] Excerpts from the deposition transcript ("Tr.") of Dr. Anwar Wissa (March 15, 2002) are included as State's Exhibit 109.

construction of the soil cement:

> The layer of soil cement beneath the storage pads will have a minimum thickness of 12 inches and a maximum thickness of 24 inches. In the event the eolian silt layer extends to a depth greater than 2 ft below the elevations of the bottoms of the storage pads, compacted clayey soils will be used to raise the elevation of the subgrade that will support the soil cement layer to an elevation of 2 ft or less below the design elevations of the bottoms of the pads.

Mr. Trudeau estimated that only about two percent of the entire pad area would need to be recompacted with compacted clayey soil. Trudeau Tr. at 33-34, 97-99, State's Exh. 108.

There is insufficient evidence to suggest that only two percent of the site will be affected. In any event, recompacted clay will have a decrease in shear strength from the design values PFS is relying upon for the native soils. PFS is again constrained by Holtec's cask tipover analysis because PFS cannot construct cement-treated soil that is deeper than two feet without exceeding Holtec's bounding conditions on cask tipover. Therefore, PFS must use recompacted and remolded clays.

(SFB, JKM) Another way in which there can be remolding of native clays is from traffic and heavy construction equipment disturbing the crust of the clays. Even small disturbances could cause a decrease in shear strength.

Q. 32: Are there any concerns about the potential changes in moisture content of the clays, and if so, what are they?

A. 32: (SFB, JKM) Yes. When clays gain moisture they soften and there is a decrease in their undrained shear strength. PFS is only testing undrained shear strength of samples at their moisture content as collected from the site. When a cement cap – such as the storage pads – is placed over cement-treated soils and the native soils, there is a potential to increase the moisture content of the native soils.

Experience has shown in conditions such as those at the PFS site you can accumulate water beneath the paved area. This will have a detrimental consequence on the engineering properties of the clay layer.

Changes in moisture content can occur from upward migrating moisture that can no longer evaporate because of the sealed surface above. You do not need to have saturated conditions to cause changes in moisture content of the native soils. By changing the evapo-transpiration environment of the soils, you can actually change the moisture content, and, therefore, the strength of those soils. Moisture that is already present in the soil will likely be redistributed until a new equilibrium is established.

Precipitation, runoff and construction activities could also cause a change in the moisture content of the native soils.

Q. 33: Please describe PFS's soil cement program.

A. 33: (SFB) The PFS soil cement program is described in SAR 2.6-108 through -121 (Rev 22) and the ESSOW between Stone & Webster & AGEC (State's Exhs. 106 and 107, respectively). Trudeau Tr. at 88-89, State's Exh. 108. The ESSOW calls for AGEC to complete the testing program in 13 months. State's Exh. 107 at 5.5. AGEC starting the testing program in about March 2001. Trudeau Tr. at 71-72. To date, AGEC has completed Phase1 (indexing property) and Phase 2 (moisture density) testing. PFS experienced problems with Phase 3 testing for durability and placed the entire testing program on hold. Trudeau Tr. at 72, 110.

Q. 34: Will the tests that PFS has conducted to date prove its design concept?

A. 34: (SFB) No. There are several tests that PFS says it will conduct in the future, most likely after PFS obtains a license from NRC. First, PFS must re-do the failed durability tests. The durability tests are to show that the soil cement around the pads and CTB can withstand freeze/thaw wet/dry cycles and will take approximately two months to complete. The next tests will be the compressive tests to show what mix of Portland cement PFS needs to add to the silts to obtain 250 psi for the soil cement around the pads and around the CTB. Moduli testing of the cement-treated soil to determine whether PFS could achieve a mix that complies with the limitations of the 75,000 psi Young's modulus could be conducted in parallel with the compressive tests. These two phases of testing would take about 2 to 3 months. Trudeau Tr. at 77-81, State's Exh. 108. Thus, there is about 4 to 5 months of testing to be completed before PFS can determine whether it has the correct "recipe" for the soil cement and whether it can concoct a cement-treated soil mix that will not exceed 75,000 psi.

This is not the end of the soil cement program. Next PFS will have to conduct interface strength tests and a bonding study to determine whether there is sufficient adhesion between the cement-treated soil with both the underlying native soils and the bottom of the concrete storage pads. Trudeau Tr. at 80-81. Mr. Trudeau admitted than only then will PFS have proven the design. Trudeau Tr. at 81

Even if PFS does complete all the tests described above, there still will not be proof of the design concept. As described in greater detail in the dynamic analysis testimony that I have presented with Dr. Ostadan, there could be cracking of the cement-treated soil under the pads and separation of the soil cement around the pads and the CTB. In other words, PFS has not shown that the use of soil cement and cement-treated soil will provide an acceptable seismic design for Skull Valley site where up to 4,000 spent nuclear fuel casks will be stored.

Q. 35: Does this conclude your testimony?

A. 34: (SFB, JKM) Yes.

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Consulting Geotechnical Engineer

Dr. James K. Mitchell received his Bachelor of Civil Engineering Degree from Rensselaer Polytechnic Institute in 1951, Master of Science Degree from the Massachusetts Institute of Technology in 1953, and the Doctor of Science Degree, also from M.I.T., in 1956.

He joined the faculty of the University of California, Berkeley in 1958 and held the Edward G. Cahill and John R. Cahill Chair in the Department of Civil Engineering at the time of his retirement from Berkeley in 1993. Concurrently he was Research Engineer in the Institute of Transportation Studies and in the Earthquake Engineering Research Center. He developed and taught graduate courses in soil behavior, soil and site improvement, and foundation engineering as part of the Geotechnical Engineering Program within the Civil Engineering Department. He served as Chairman of the Department of Civil Engineering from 1979 through 1984. He was appointed the first Charles E. Via, Jr. Professor in the Via Department of Civil Engineering at Virginia Tech in 1994, University Distinguished Professor in 1996, and University Distinguished Professor, Emeritus, in 1999.

His primary research activities have focused on experimental and analytical studies of soil behavior related to geotechnical problems, admixture stabilization of soils, soil improvement and ground reinforcement, physico-chemical phenomena in soils, the stress-strain time behavior of soils, in-situ measurement of soil properties, and mitigation of ground failure risk during earthquakes. He supervised the dissertation research of 72 Ph.D. students. He has authored more than 350 publications, including two editions of the graduate level text and reference, "Fundamentals of Soil Behavior," and several state-of-the-art papers and guidance documents on soil stabilization, ground improvement, and earth reinforcement. During the 1960's and early 1970's he served as the NASA Principal Investigator for the Soil Mechanics Experiment, which was a part of Apollo Missions 14-17 to the Moon.

State's
Exhibit 105

Dr. Mitchell serves as a consultant on geotechnical problems and earthwork projects of many types, especially soil stabilization, ground improvement for seismic risk mitigation, earthwork construction, and environmental geotechnology, to numerous governmental and private organizations, both nationally and internationally. Recent and currently active projects include the evaluation of seismic stabilities and design of liquefaction mitigation options for Success Dam in California (U.S. Army Corps of Engineers) and Pineview and Deer Creek Dams in Utah (U.S. Bureau of Reclamation), peer reviewer for geotechnical design and construction issues in the proposed depressed Reno Rail Corridor (Kleinfelder), ground improvement aspects of the Port of Oakland Wharf and Embankment Strengthening Program (Harding Lawson Associates), ground improvement and fill stabilization for the proposed San Francisco Airport Expansion (Fugro West), design review – ground improvement for the I-95/Rt.1 Interchange section of the Woodrow Wilson Bridge replacement project (Haley & Aldrich, Virginia Geotechncial Services, URS, HNTB), and the Embankment Technical Review Board for the Third Runway at Seattle-Tacoma International Airport.

He is licensed as a Civil Engineer and as a Geotechnical Engineer in California, and as a Professional Engineer in Virginia. He is a Fellow and Honorary Member of the American Society of Civil Engineers. He served as Secretary (1966-69), Vice-Chairman (1970), and Chairman (1971) of the Geotechnical Engineering Division of ASCE and as Chairman of the United States National Committee for the International Society for Soil Mechanics and Foundation Engineering. He was Chairman of the ASCE Committee on Soil Properties and Chairman of the Committee on Placement and Improvement of Soils, as well as a member of the Environmental Geotechnics Committee. He served as President of the San Francisco Section of ASCE and Chairman of the California State Council of ASCE during 1986-87. He was Chairman of the Transportation Research Board Committee on Physico-Chemical Phenomena in Soils from 1966-1973, and was a member of the TRB Executive Committee from 1983-1987. He was Chairman of the Geotechnical Board of the U.S. National Research Council from 1990 through 1994. He recently completed service as Vice Chair of a NRC study committee for development of science needs for remediation of contaminated Department of Energy weapons sites. He now is a member of a NRC study committee to advise the Department of Energy on Remediation Science and Technology for the Hanford Site. He was Vice President of the International Society for Soil Mechanics and Foundation Engineering from 1989-1994.

Dr. Mitchell was awarded the Norman Medal in 1972 and 1995, the Thomas A. Middlebrooks Award (three times), the Walter L. Huber Research Prize and the Karl Terzaghi Award, all from the American Society of Civil Engineers; the Distinguished Teaching Award and the Berkeley Citation from the University of California; the Western Electric Fund Award of the American Society for Engineering Education; the Medal for Exceptional Scientific Achievement from the National Aeronautics and Space Administration, and has been selected as the recipient of the 2001 Kevin Nash Gold Medal of the International Society for Soil Mechanics and Geotechnical Engineering. He was elected to the United States National Academy of Engineering in 1976 and to the U. S. National Academy of Sciences in 1998.

Lists of projects and publications are available on request.

April 2001

2.6.4.11 Techniques to Improve Subsurface Conditions

Soil Cement

Discussions presented in Section 2.6.1.12, above, indicate that the soils underlying the eolian silt layer at the surface of the PFSF site are suitable for support of the proposed structures; therefore, no special construction techniques are required for improving the subsurface conditions below the eolian silt. The eolian silt, in its *in situ* loose state, is not suitable for founding the structures at the site. The basemat of the Canister Transfer Building will be founded on the silty clay/clayey silt layer beneath the eolian silt. It was originally intended that the cask storage pads also would be founded on the silty clay/clayey silt layer. However, instead of excavating the eolian silt from the pad emplacement area and replacing it with suitable structural fill, it will be mixed with sufficient portland cement and water and compacted to form a strong soil-cement subgrade to support the cask storage pads. Soil cement will also be utilized around the Canister Transfer Building. The required characteristics of the soil cement will be engineered during detailed design and constructed to meet the necessary strength requirements.

During construction of the storage pads, all of the eolian silt in the quadrant under construction will be excavated. The eolian silt will be mixed with sufficient cement and water and compacted to produce soil cement across the pad area, up to the design elevations of the bottoms of the storage pads. The layer of soil cement beneath the storage pads will have a minimum thickness of 12 inches and a maximum thickness of 24 inches. In the event that the eolian silt layer extends to a depth greater than 2 ft below the elevations of the bottoms of the storage pads, compacted clayey soils will be used to raise the elevation of the subgrade that will support the soil cement layer to an elevation of 2 ft or less below the design elevations of the bottoms of the pads. This will ensure that the layer of soil cement does not exceed a thickness of 2 ft. This is the

State's
Exhibit 106

maximum permissible thickness of the soil cement layer, since the storage cask hypothetical tipover and drop analyses were performed assuming a 2.0-ft thick layer of soil cement underlying the storage pads.

Strength of Soil Cement and Minimum/Maximum Thickness Requirements

The soil cement underlying the pads shall have a minimum unconfined compressive strength of 40 psi to ensure that there is an adequate factor of safety against sliding of an entire column of pads (S&W Calculation 05996.02-G(B)-4, SWEC, 2001b). This layer of soil cement is required to be no greater than 2-ft thick and have a static modulus of elasticity less than or equal to 75,000 psi to ensure that the decelerations from a hypothetical storage cask tipover event or vertical end drop accident do not exceed HI-STORM design criteria (Section 3.2.11.3).

Following construction of the storage pads on top of this layer of soil cement, additional soil cement will be placed around and between the cask storage pads, extending from the bottoms of the pads to a level that is 28 inches above the bottoms of the storage pads. The remaining 8 inches, from the top of the soil cement up to grade, will be filled with coarse aggregate, placed and compacted to be flush with the tops of the pads to permit easy access by the cask transporter. The soil cement placed around the sides of the storage pads is expected to have a minimum unconfined compressive strength of at least 250 psi to satisfy durability requirements within the depth of frost penetration (based on S&W Calculation 05996.02-G(B)-4 (SWEC, 2001b), as discussed in Section 2.6.1.12.1).

The Canister Transfer Building basemat will be founded on the silty clay/clayey silt layer that is below the eolian silt. The design calls for soil cement to be placed around the Canister Transfer Building base mat to make the free-field soil profile for the building consistent with that for the storage pad emplacement area and to help resist sliding forces due to the higher design basis ground motions. Soil cement will surround the foundation mat and will extend outward from the mat to a distance equal to the associated mat dimension; i.e., approximately 240 ft out from the mat in the east and west directions and approximately 280 ft out in the north and south directions. Existing soils (eolian silt and silty clay/clayey silt) will be excavated to a depth of approximately 5 ft 8 inches below grade, mixed with cement, and placed and compacted around the foundation mat.

The soil cement placed around the Canister Transfer Building foundation mat will be 5 ft thick and have a minimum unconfined compressive strength of 250 psi to ensure that there is an adequate factor of safety against sliding of the Canister Transfer Building (based on Calculation 05996.02-G(B)-13 (SWEC, 2001c), as discussed in Section 2.6.1.12.2). The top 8 inches will be filled with compacted coarse aggregate, similar to that used in the pad emplacement area.

PFS is developing the soil-cement mix design using standard industry practice. This effort includes performing laboratory testing of soils obtained from the site. This on-going laboratory testing is being performed in accordance with the requirements of Engineering Services Scope of Work (ESSOW) for Laboratory Testing of Soil-Cement Mixes, ESSOW 05996.02-G010 (SWEC, 2001e). This program includes measuring gradations and Atterberg limits of samples of the near-surface soils obtained from the site. It includes testing of mixtures of these soils with varying amounts of cement and the testing of compacted specimens of soil-cement to determine moisture-density relationships, freeze/thaw and wet/dry characteristics, compressive and tensile strengths, and permeability of compacted soil-cement specimens. The entire laboratory testing program is being conducted in full compliance with the Quality Assurance (QA) Category I requirements of the ESSOW.

As part of this effort, PFS is performing so-called durability testing. These tests are performed in accordance with ASTM D559 and D560 to measure the durability of soil cement specimens exposed to 12 cycles of wet/dry and freeze/thaw conditions. As indicated on p. 16 of PFS Calculation 05996.02-G(B)-4 (SWEC, 2001b):

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

PFS is performing these tests to determine the amounts of cement and water that must be added to the site soils and to determine the compaction requirements to ensure that the soil cement will be durable and will withstand exposure to the elements. As indicated on p. 8 of Portland Cement Association (1971):

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

And on p. 32:

"The principle requirement of a hardened soil-cement mixture is that it withstand exposure to the elements. Thus the primary basis of comparison of soil-cement mixtures is the cement content required to produce a mixture that will withstand the stresses induced by the wet-dry and freeze-thaw tests. The service record of projects in use proves the reliability both of the results based on these tests and of the criteria given below.

The following criteria are based on considerable laboratory test data, on the performance of many projects in service, and on information obtained from the outdoor exposure of several thousand specimens. The use of these criteria will provide the minimum cement content required to produce hard, durable soil-cement, suitable for base-course construction of the highest quality.

1. *Soil-cement losses during 12 cycles of either the wet-dry test or freeze-thaw test shall conform to the following limits:*
   Soil Groups A-1, A-2-4, A-2-5, and A-3, not over 14 percent;
   Soil Groups A-2-6, A-2-7, A-4, and A-5, not over 10 percent;
   Soil Groups A-6 and A-7, not over 7 percent.
2. *Compressive strengths should increase both with age and with increases in cement content in the ranges of cement content producing results that meet requirement 1."*

The on-going laboratory testing program will also include additional tests to confirm that the bond at the interfaces between concrete and soil-cement, soil-cement and soil-cement, and soil-cement and the site soils will exceed the strength of the in situ clayey soils. These tests will include direct shear tests, performed on specimens prepared from the site soils at various cement and moisture contents, in a manner similar to that used by DeGroot in his testing of bond along soil-cement interfaces.

Based on the above, PFS has adequately defined the measures that will be followed in the design and construction of the soil cement to assure that the assumed bonds can be sustained through the period of interest. PFS has committed to performing site-specific testing to confirm that the required interface strengths are available to resist sliding forces due to an earthquake. As indicated above, this testing will include direct shear tests to be performed in the laboratory in the near-term (pre-construction) during the soil-cement mix development to demonstrate that the required interface strengths can be achieved and during construction to demonstrate that the required interface strengths are achieved. In addition, PFS has committed to augmenting this field testing program by performing additional site-specific testing of the strengths achieved at the interface between the bottom of the soil cement and the underlying soils.

The most recent analyses of the PFSF design basis ground motions assumed the incorporation of a 5 ft thick soil cement layer over the entire pad emplacement area and also surrounding the Canister Transfer Building. The 5 ft soil cement layer around the Canister Transfer Building extends to the free field boundary from the edge of the building basemat. This soil cement layer is assumed to have a minimum shear wave velocity greater than 1,500 fps (Geomatrix 2001a and 2001b). As indicated in Section 2.6.1.2.2, soil cement around the Canister Transfer Building should have a minimum unconfined compressive strength of 250 psi to ensure a factor of safety greater than 1.1 for seismic sliding stability. The design requirements for the 5 ft thick soil cement layer around the Canister Transfer Building will be based on the results of laboratory and field testing to be conducted during the final design stage.

The surficial layer of eolian silt, existing across the entire site as shown in the pad emplacement area foundation profiles (Figure 2.6-5, Sheets 1 through 14), is a major factor in the earthwork required for construction of the facility. This layer consists of a nonplastic to slightly plastic silt, and it has an average thickness of approximately 2 feet across the pad emplacement area. This layer was expected to be removed prior to construction of the storage pads. However, based on evaluation of the earthwork associated with site grading requirements for flood protection and the environmental impacts of truck trips required to import fill to replace this material, PFS will stabilize this soil with cement and use it as base material beneath the storage pads and adjacent driveways.

Section 2.6.1.12 indicates that there is ample margin in the factor of safety against a bearing capacity failure of the silty clay/clayey silt underlying the site and that the settlements are acceptable for these structures. They indicate that the critical design factor with respect to stability of these structures is the resistance to sliding due to loadings from the design basis ground motion. As discussed in that section, the silty clay/clayey silt layer has sufficient strength to resist these dynamic loadings; therefore, adequate sliding resistance can be provided by constructing the structures directly on the silty clay/clayey silt layer. The soil cement around the storage pads and Canister Transfer Building will be designed and constructed to have a minimum unconfined compressive strength of 250 psi and quality assurance testing will be performed during construction to demonstrate that this minimum strength is achieved. The soil cement directly beneath the storage pads will be designed and constructed to have an unconfined compressive strength of at least 40 psi with static elastic modulus of less than ~75,000 psi. Therefore, the resistance to sliding due to loadings from the design basis ground motion will be enhanced by constructing the cask storage pads on a properly designed and constructed soil-cement subgrade. See the section titled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement" in 2.6.1.12.1 for additional details.

Using soil cement to stabilize the eolian silt will reduce the amount of spoil materials generated, create a stable and level base for pad construction, and substantially improve the sliding resistance of the storage pads. The soil cement will be placed above the *in situ* silty clay/clayey silt layer and will be designed to improve the strength of the eolian silt so that it will be stronger than the clayey soils that were originally intended for use as the founding medium for the pads. The soil cement will also be used to replace the compacted structural fill that the original plan included between the rows of pads. This continuous layer of soil cement, existing under and between the pads, will spread the loads from the pads beyond the footprint of the pads, resulting in decreased total and differential settlements of the pads. The layer of soil cement above the base of the pads and the bond and friction of the pad foundation with the underlying soil-cement layer will greatly increase the sliding resistance of the pad.

Soil cement has been used extensively in the United States and around the world since the 1940's. It was first used in the United States in 1915 for constructing roads. It also has been used at nuclear power plants in the United States and in South Africa. The largest soil-cement project worldwide involved construction of soil-cement slope protection for a 7,000-acre cooling-water reservoir at the South Texas Nuclear Power Plant near Houston, TX. Soil cement also was used to replace an ~18-ft thick layer of potentially liquefiable sandy soils under the foundations of two 900-MW nuclear power plants in Koeberg, South Africa (Dupas and Pecker, 1979). The strength of soils can be improved markedly by the addition of cement. The eolian silt at the site is similar to the soils identified as Soil A-4 in Nussbaum and Colley (1971), Soils 7 and 8 in Balmer

(1958), and Soil 4 in Felt and Abrams (1957). As indicated for Soil A-4 in Table 5 of Nussbaum and Colley (1971), the addition of just 2.5% cement by weight to the silt increased the cohesion from 5 psi (720 psf) to 30 psi (4,320 psf). The cohesion for Soils 7 and 8 also were increased significantly by the addition of low percentages of cement, as shown on Tables VI and VII of Balmer (1958). Figure 10 in Felt and Abrams (1957) illustrates the continued strength increase over time for these soil-cement mixtures. Other examples of soil-cement strength increases over time are presented in Figure 4.3 of ACI (1998), Table 6 of Nussbaum and Colley (1971), and Figures 6 and 7 of Dupas and Pecker (1979). Therefore, the soil cement will be much stronger than the underlying silty clay/clayey silt and the strength will increase with time, providing an improved foundation material. This will provide additional margin against sliding compared to the original plan to construct the pads directly on the silty clay/clayey silt layer.

As shown in the section titled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement" in Section 2.6.1.12.1 above, the shear resistance required at the base of the pads can be provided easily by the passive resistance of the soil cement acting against the vertical side of the foundation and by bond between the pad foundation and soil-cement contact and the cohesive strength of the soil cement. Shear resistance will be transferred through the approximately 2-ft thick soil-cement layer and into the underlying silty clay/clayey silt subgrade. Additional resistance will be provided by the continuous layer of soil cement under and between the pads; therefore, shear resistance requirements within the silty clay/clayey silt layer will be less with the soil-cement layer compared to the original plan to construct the pads directly on the silty clay/clayey silt without the proposed soil-cement layer.

DeGroot (1976) indicates that this bond strength can be easily obtained between layers of soil cement. He performed nearly 300 laboratory direct shear tests to determine the effect of numerous variables on the bond between layers of soil cement. These variables included the length of time between placement of successive layers of soil cement, the frequency of watering while curing soil cement, the surface moisture condition prior to construction of the next lift, the surface texture prior to construction of the next lift, and various surface treatments and additives.

His results demonstrated that, with the exception of treating the surface of the lifts with asphalt emulsion, asphalt cutback, and chlorinated rubber compounds, the bond strength always exceeded 6.6 psi, the minimum required value of cohesion if the passive resistance acting on the sides of the pads is ignored. The minimum bond strength he reports, other than for the asphalt and chlorinated rubber surface treatments identified above, is 8.7 psi. This value applied for two tests that were performed on samples that had time delays of 24 hours and did not have a cement surface treatment along the lift line. He reports that nearly all of the specimens that used a cement surface treatment broke along planes other than along the lift lines, indicating that the bond between the layers of soil cement was stronger than the remainder of the specimens. Excluding the specimens that had 24-hr delays between lift placements and which did not use the cement surface treatment, the minimum bond strength was 10.7 psi and there were only two others that had bond strengths that were less than 20 psi. Even these minimum values for the group of specimens that did not use a cement surface treatment exceeded the cohesive strength (6.6 psi) required to obtain an adequate factor of safety against sliding without including the passive resistance acting on the sides of the pads, and all of the rest were much greater, generally more than an order of magnitude greater.

DeGroot reached the following conclusions:

1. Increasing the time delay between lifts decreases bond.
2. High frequency of watering the lift line decreases the bond.

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

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

DeGroot's direct shear test results demonstrate that the specimens having a cement surface treatment all had bond strengths that ranged from 47.7 psi to 198.5 psi, with the average bond strength of 132.5 psi. Even the minimum value of this range is nearly an order of magnitude greater than the cohesion (6.6 psi) required to obtain a factor of safety against sliding of 1.1, conservatively ignoring the passive resistance available on the sides of the pads. Therefore, when required due to unavoidable time delays, the techniques DeGroot describes for enhancing bond strength will be used between the top of the soil cement and succeeding lifts or the concrete cask storage pads, to assure that the bond at the interfaces are greater than the minimum required value. These techniques will include roughening and cleaning the surface of the underlying soil cement, proper moisture conditioning, and using a cement surface treatment.

A fundamental assumption in the PFS approach is that sufficient bonding and shear transfer between clay and soil cement interfaces can be achieved using various construction techniques. As indicated above, DeGroot has demonstrated that techniques are available that will enhance the bond between lifts of soil cement. These techniques should be equally effective when applied to the soils at the PFSF site. PFS has committed to perform direct shear tests of the interface strengths during the design phase of the soil cement to demonstrate that the required interface strength can be achieved, as well as during construction, to demonstrate that they are achieved.

PFS has discussed the change to use soil cement beneath the storage pads with the project consultants who have analyses in-place that are based on the storage pads resting on the silty clay/clayey silt. The consultants contacted were Geomatrix (development of seismic criteria and soil dynamic properties), Holtec International (cask stability analysis), and International Civil engineering Consultants (pad design). Each has indicated their analyses would not be adversely affected by this proposed change.

The design, placement, testing, and performance of soil cement is a well-established technology. The "State-of-the-Art Report on Soil Cement" (ACI, 1998) provides information about soil cement, including applications, materials, properties, mix proportioning, design, construction, and quality-control inspection and testing techniques. PFS will develop site-specific procedures to implement the recommendations presented in ACI (1998) regarding mix proportioning, testing, construction, and quality control. The following describes the processes that will be used to develop a proper soil-cement mix design and establish adequate sliding resistance at each material interface in the storage pad and soil system:

- Soil-Cement Mix and Procedure Development – The sliding forces due to the design basis ground motion will be resisted by bond between the base and sides of the foundation and the soil cement and by passive resistance of the soil cement acting against the vertical side of the foundation. The soil-cement mix will be designed and constructed to exceed the minimum shear resistance requirements. During the soil-cement design phase, direct shear testing will be conducted along manufactured soil-cement lift contacts and concrete contacts that represent anticipated field conditions. The direct shear testing, along with other standard soil-cement testing, will be used to confirm that adequate shear resistance and other strength requirements will be provided by the final soil-cement mix design. Procedures required for placement and treatment of the soil cement, lift surfaces, and foundation contact will be established in accordance with the recommendations of ACI (1998) during the mix design and testing process. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during this detailed design phase of the project.

- Soil-Cement Lift and Concrete Interface – The soil cement will be constructed in lifts approximately 6-in. thick (compacted thickness) as described in ACI (1998). Construction techniques will be used to ensure that the interface between the soil-cement layers will be adequately bonded to transmit shear stresses. As described in Section 6.2.2.5 of ACI (1998), these techniques will include, but will not be limited to: minimizing the time between placement of successive layers of soil cement, moisture conditioning required for proper curing of the soil cement, producing a roughened surface on the soil cement prior to placement of additional lifts or concrete foundations, and using a dry cement or cement slurry to enhance the bonding of concrete or new soil cement layers to underlying layers that have already set. In addition to conventional quality control testing performed for soil-cement projects, direct shear testing will be performed on representative samples obtained from placed lift contacts to confirm design requirements are obtained. Sacrificial soil-cement lifts may be used to protect the soil-cement subgrade in the pad foundation areas.

- Soil Cement and *In Situ* Clay Interface – The soil cement and *in situ* clay interface will be constructed such that a good bond will be established between the two materials. Construction techniques will be utilized that will ensure that the integrity of the upper surface of the clay is maintained and that a good interface bond between the two materials is obtained. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during the detailed design phase of the project.

An additional benefit of incorporating the soil cement into the design is that it will minimize the environmental impacts of constructing the facility. Using on-site materials to construct the soil cement, rather than excavating and spoiling those materials, will reduce environmental impacts of the project. In addition, replacement of some of the structural fill layer between the rows of pads with soil cement, as shown in Figure 4.2-7, will result in reduced trucking requirements associated with transporting those materials to the site.

Adequacy of the Soil Cement Design

The adequacy of the design of the soil cement surrounding and underlying the pads to ensure the sliding stability of the pads under seismic conditions is demonstrated by S&W Calculation 05996.02-G(B)-04 (SWEC, 2001b). This calculation determined that there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement and between that soil cement layer and the underlying clayey soils that the factor of safety against sliding exceeds the minimum required value, with no credit for the soil cement placed between storage pads above the bottom of the pads. The underlying layer of soil cement is also required to have a static modulus of elasticity less than or equal to 75,000 psi to ensure that decelerations of a cask resulting from a hypothetical storage cask tipover event or vertical end drop accident do not exceed design criteria (Sections 4.2.1.5.1.E and 8.2.6).

The large extent of soil cement in the storage pad emplacement area allows the soil cement layer to be considered as part of the free field soil profile for the site response analyses. The properties of the soil cement, higher shear wave velocity and higher density than the existing soils in the area, help to minimize the response at the surface of the site caused by the design basis ground motions. Soil cement was added around the Canister Transfer Building foundation mat to make the free field soil profile for the building consistent with that for the storage pad emplacement area (as discussed in Section 2.6.4.11), and to help resist sliding forces, in conjunction with the building's perimeter key, due to the revised design basis ground motions. The adequacy of this design feature is demonstrated in Calculation No. 05996.02–G(B)-13 (SWEC, 2001c), which determined that the design of the soil cement surrounding the Canister Transfer Building (in conjunction with the building's perimeter key) is adequate to ensure the stability of the Canister Transfer Building under seismic conditions.

2.6.4.12 Criteria and Design Methods

The allowable bearing capacity of footings is limited by shear failure of the underlying soil and by footing settlement. The minimum factor of safety against a bearing capacity failure from static loads (dead load plus maximum live loads) is 3.0 and from static loads plus loads due to extreme environmental conditions, such as design basis ground motion, is 1.1. Allowable settlements are determined based on Table 14.1, "Allowable Settlement," of Lambe & Whitman (1969) and assume that the differential settlement will be 3/4 of the maximum settlement. Section 2.6.1.12 provides more details.

In order to comply with the requirements of NUREG-75/087, Section 3.8.5, "Foundations," Section II.5, "Structural Acceptance Criteria," the recommended minimum factor of safety against overturning or sliding failure from static loads (dead load plus maximum live loads) is 1.5 and from static loads plus loads due to extreme environmental conditions, such as design basis ground motion, is 1.1. Where the factor of safety against sliding is less than 1 due to the design basis ground motion, the displacements the structure may experience are calculated using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes. The magnitude of these displacements are evaluated to assess the impact on the performance of the structure. See Section 2.6.1.12 for details about these analyses.

2.6.5 Slope Stability

There are no slopes close enough to the proposed Important to Safety facilities that their failure could adversely affect the operation of these facilities.

STATE'S EXHIBIT 107

ENGINEERING SERVICES SCOPE OF WORK

FOR

LABORATORY TESTING OF SOIL-CEMENT MIXES

ESSOW No. 05996.02-G010 (Rev. 0)

Dated January 31, 2001

This exhibit is being filed as a proprietary filing document but in so doing, the State makes no claim as to its confidentiality

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	)	**PAUL J. TRUDEAU**
Facility)	)	
	)	**(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

State's
Exhibit 108

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A. That's response spectra, I believe.
MR. TRAVIESO-DIAZ: Excuse me for interrupting. Do you mean 1160?
THE WITNESS: It's 1.165.
Q. (By Ms. Chancellor) No, you said 1.60?
A. 60, yeah. It might be 1. -- I don't know. I don't know whether --
MR. TRAVIESO-DIAZ: That was the basis of my objection before. You know, it is very hard for the witness to remember without being presented a document, Are you familiar with it?
MS. CHANCELLOR: That's fine. If he's given me the name of the document and given me his best recollection of the reg guide. I'm not going to challenge if he relies on a document that he's got in his filing cabinet.
Q. I'm just trying to get a sense of what reg guides and what regulations you work with, in general, with respect to your geotechnical investigation. So we've got 1.567, 0800 and reg guide dealing with response spectra.
Anything else you'd like to add to the list?
A. No.
MS. CHANCELLOR: Okay. If I could have

this document marked as Exhibit 12.
(A discussion was held off the record.)
(Exhibit-12 was marked.)
Q. (By Ms. Chancellor) Mr. Trudeau, I've handed you a copy of PFS -- an excerpt from PFS's SAR, Revision 22, Section 2.6.4.11, Techniques to Improve Subsurface Conditions. Are you familiar with this section of the SAR?
A. Yes.
Q. Are you primarily responsible for authoring this section of the SAR?
A. Yes.
Q. And does this section, in general, deal with PFS's application of soil cement in its foundation design?
A. Yes.
Q. And what experience have you had in applying soil cement in foundation design in any other project?
A. I have none.
Q. Are you responsible for any other sections of the SAR where you've been basically the primary author?
A. Chapter --
Q. I've got a copy of Chapter 2 here and a

part of -- if you'd like to take a look at it. I don't have all of Chapter 2 but the first part of Chapter 2.
A. But not the table of contents?
Q. Oh, doesn't it have -- at the beginning of the chapter, doesn't it have the table of contents?
A. Sorry. Found it.
Q. I think that was a document control argument.
You can take the clip out.
A. How detailed a list do you want here?
Q. Oh, just the main general areas --
A. 2.6.1.5, Facility Plot Plan and Geologic Investigations, I co-authored or authored most of that, I would say.
Same with .6, Relationship of Major Foundations to Subsurface Materials, I authored that.
2.6.1.7, Excavations and Backfill, likely I wrote that --
Q. Okay.
A. -- back in '97.
I probably had input to the Site Groundwater Conditions in 2.6.1.9, but that may have been authored by someone else. Same with 2.6.1.10, Geophysical Surveys.
2.1.1.11, Static and Dynamic Rock

Properties at the Site, is largely going to be my work.
And 2.6.1.12, Stability of Foundations for Structures and Embankments, will be largely my work.
2.6.4, Stability of Subsurface Materials, was probably authored by me as well.
2.6.4.7, Response of Soil and Rock to Dynamic Loading.
2.6.4.8, Liquefaction Potential.
2.6.4.9, Design Basis Ground Motion, I probably authored, but it just refers to Geomatrix's work earlier in the SAR.
2.6.4.10, Static Analyses.
Q. Going back to the design basis ground motion, would that be the way in which you reviewed and used -- an example of the way in which you used and reviewed the Geomatrix calculation to write up the --
A. This section of the -- this section of the SAR just simply just defines what the design basis ground motion is, and it references back to Geomatrix's complete description in early sections of the SAR.
Q. Okay.
A. So this just gets that it's .117 g horizontal, .695 g vertical, and it refers to the Geomatrix reports.
Q. Okay. I understand. Thank you.

Q. Now, looking at SAR on page 1.6-108, towards the bottom of the page, it says that -- one, two, three, four lines from the bottom, it says that, Compacted clay soils will be used to raise the elevation of the subgrade.
Will that be -- will the soils be compacted on-site, those clay soils?
A. Correct.
Q. And what consideration have you given to the remolding of those clay soils from compaction?
A. Well, they will be remolded as part of the compaction, but we'll -- we'll have to demonstrate by testing that we've got adequate strength in those compacted clay soils.
Q. And how will you demonstrate that?
A. By testing.
Q. When?
A. As the project moves ahead.
Q. And how --
A. These -- these areas represent a very minor portion of that entire pad emplacement area. I'm -- to hazard a guess, I would say it's probably less than 2 percent of the entire area. It's just mentioned here in case we hit that eventuality. We understand that we've got a 2-foot limitation. If we've got a

2-and-a-half-foot-deep hole, we've got to put something else in there. And there may be 2 percent of the entire area where we're going to find that the in situ subgrade with the design grades are such that we need to fill it a little thicker than the 2-foot limitation of the soil cement below the pad. So this statement is what we're planning to do to get that piece of the subgrade filled in.
Q. And what's your basis for assuming that you'll only find about 2 percent of --
A. That's based on a review of the data that we've got, the profiles that are shown in the SAR, Figures 2.6-5 --
Q. The pallet --
A. Yeah. -- sheets 1 through 14. If you take a look at where the pads are shown on those figures, you'll see that almost all of them are within the 2-foot limitation.
(A discussion was held off the record.)
Q. (By Ms. Chancellor) Do you plan to develop a grading plan to show these clay -- clay areas -- just a moment.
(A discussion was held off the record.)
Q. (By Ms. Chancellor) I was way off.
Do you plan to develop a grading plan to

show these silty areas where you'll need to have the compacted soils?
A. I would expect that we'll have some sort of an excavation plan that will be part of the construction drawings that will be produced. I don't know that we'll actually go out and do any additional work at this point to try to identify where this bottom is that -- that we're discussing right now prior to getting out and excavating, but those discussions will be held as part of the normal process of getting the construction specs set up for this -- for this project.
Q. On page 3.6-113 of the SAR, if you'd turn to that page, it states that --
A. You mean 2.6?
Q. What did I say? Yeah, 2.6.113. In the middle of the first full paragraph, the sentence that starts, This continuous layer of soil cement existing under and between the pads will spread the loads from the pads beyond the footprint of the pads resulting in decreased total differential settlement of the pads.
In -- in the settlement calculations you -- it showed the settlement of the pads was 3 inches, and now it's 1.7 inches. Is this statement the reason for that decrease in the settlement of the pads?
MR. TRAVIESO-DIAZ: Do you understand the

question?
THE WITNESS: That's not the reason for this decrease, no.
Q. (By Ms. Chancellor) What's the reason --
A. I mean this here text in the SAR is not the reason for the decrease in the settlement numbers that you just cited. I don't recall exactly what's in the calcs that you've cited, but if you've got them, I'll take a look and --
Q. Which ones do you need?
A. The one that cites the 1.7.
Q. I've got the 1.7 in the SAR, but I didn't bring the -- I didn't bring the settlement calcs with me. I can get those.
On page 2.6.5, Revision 22, of the SAR, which I'm handing you now, it has a -- it shows the settlement of the pads as 1.7, and in Revision 17 the elastic settlement was 0.5. The next number, which I can't read upside down, consolidated settlement, changed from 1.7 to 0.8, and a secondary compression from 1.1 to 0.4.
What is the reason -- if we need to get the calculations, we can pick this up later, but what is the reason for the change in settlement from 3.3 inches to 1.7 inches?

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find it. If you're happy with what I've given you so far, we can go move on.

Q. No. You take as much time as you like.

MR. TRAVIESO-DIAZ: Can we go off the record for a second?

MS. CHANCELLOR: Sure.

(A discussion was held off the record.)

MS. CHANCELLOR: Back on the record.

THE WITNESS: Commitments that I can find stated in this section of the SAR at this point in time are on page 2.6-111. The second sentence in the second paragraph reads, PFS has committed to performing site-specific testing to confirm that the required interface strengths are available to resist sliding forces due to an earthquake.

It continues on, a sentence following the next one, In addition, PFS is committed to augmenting this field testing program by performing additional site-specific testing of the strengths achieved at the interface between the bottom of the soil cement and the underlying soils.

So those are the commitments I was referring to in my response to the interrogatory.

Q. (By Ms. Chancellor) So on page 109, 117 and on page 111 is what you've testified to at the

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moment?

MR. TRAVIESO-DIAZ: I think he said 117 not 107, 117.

MS. CHANCELLOR: Did I say --

MR. TRAVIESO-DIAZ: I thought you said 107.

MS. CHANCELLOR: I meant 117. I beg your pardon.

THE WITNESS: Yes.

Q. (By Ms. Chancellor) Okay. And is it true that PFS will implement a document called State-of-the-Art on Soil Cement, a document by American Concrete Institute? If we look on page 2.6-117, in the last paragraph of the design placement testing, PFS will development site-specific procedures to implement the recommendations presented in State-Of-the-Art Report on Soil Cement, ACI 1998?

A. Correct.

Q. I'm handing you a document, State-of-the-Art Report on Soil Cement, ACI 230.1 R-90. Is this the document that is referred to on page 2.6.117 of the SAR?

A. I do not think so. I think this is an earlier version of it.

Q. Okay. Thank you.

Have you produced to the State a copy of

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this document, State-Of-the-Art Report on Soil Cement, 1998, that you're using? If not, we'd like to request a copy. It's a document referred to on 2.6-117.

Can we go off the record a moment?

(A discussion was held off the record.)

Q. (By Ms. Chancellor) Mr. Trudeau, Mr. O'Neill from NRC during the break handed me a copy of a document entitled State-Of-the-Art Report on Soil Cement, ACI 230.1R-90, Reapproved 1997. If you'd take a look at that document, is that the document that is referred to on 2.6.117 of the SAR?

A. Yes, I believe it is.

Q. Thank you.

Could you describe the PFS soil cement test program?

A. Yes.

Q. Would you?

A. The purpose of the ongoing program is to develop design mix, a soil cement design mix with the site soils. Essentially it's to determine how much cement we need to mix with the various types of soils that we've encountered in the test pits that we took at the site to produce a durable soil cement mix, one that will meet the requirements of the ASTM tests for wet/dry cycles and freeze/thaw cycles.

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The program included digging 16 test pits at the site where we sampled -- took bulk samples of the soils on a 2-foot interval, going down below ground in each of these 16 locations. For the southeast quadrant of the site, the Phase 1 area of the pad emplacement area, for each of the 2-foot depths we took a bucket every 6 inches, essentially, so we ended up with four buckets for the zero-to-2-foot depth and four buckets for the 2-to-4-foot department and four buckets for the 4-to-6-foot depth in each of test pits 1 through 4. The other three quadrants, we only took one bucket for each of the 2-foot depths.

So we collected quite a number of buckets of soil from the site -- these are 5-gallon buckets -- for testing for the soil cement mix design process.

The first phase of the laboratory testing included index property testing, measuring water contents of all of these samples that we tested, Atterberg limits for most of them -- each of the depth ranges we measured Atterberg limits. We didn't test all four buckets from each of the four test pits in the Phase 1 area to this date, but we've gotten gradations performed on those as well, including both sieve analyses and hydrometer analyses.

Based on that -- the results of that

Q. And is that maximum strength approximately a hundred psi?
A. Yes.
Q. And is the strength a factor on how much portland cement you mix with the silt?
A. Yes.
Q. And in your test program are you mixing various percentages of cement to determine what the recipe should be?
A. Yes.
Q. And what are those percentages?
A. The ESSOW identifies some in that Section 1.0, Scope of Work - General, in the third paragraph.
Q. Oh, I knew I saw it somewhere. Okay.
A. Now, this says the expected cement contents to be used in the testing process of 6, 9 and 12 percent. These are representative of what we expected for the soil cement, not the cement-treated soil.
Q. Okay.
A. So we expect that we'll be using less cement than these for the cement-treated soil. But the cement-treated soil is located below the pad, which is 36 inches thick, so it does not have to withstand

freeze/thaw cycles, so it will not need to comply with the freeze/thaw durability test. It's below the frost zone in Skull Valley, which is only 30 inches below grade.
Q. So the soil cement program, is that limited to true soil cement which you will use around the CTB and around the pads?
A. That's -- that may be what this ESSOW says, but we realize that we need to have testing of the cement-treated soil as well. So I don't -- I don't recall that we have any specific discussion of the cement-treated soil in here, but we have to do the testing on the cement-treated soil. So it will be tested as part of this program, eventually.
Q. But the cement-treated soil will not be tested on the freeze/thaw ASTM test --
A. Correct. It will be tested for compressive strength and modules because those are the required parameters for design.
Q. Will it be tested for durability or is that only the freeze/thaw --
A. The freeze/thaw and the wet/dry tests are the durability tests.
Q. Well, will the cement-treated soil be "treated" for wet/dry tests?

A. No.
Q. Even though you claim it's below the frost line, won't it still be subject to wet/dry conditions?
A. Not really.
Q. Why not?
A. It's 3 feet down, below the soil cement, below the concrete pad -- actually, the concrete pad is the critical area.
Q. The testing program for the cement-treated soil, has any work started on that?
A. It's the same soils as are being tested in this program, so all of the Phase 1 work is still applicable for those soils.
Q. And the Phase 1 is the collection of the samples?
A. It's the index property testing that's been done. The Phase 2 testing I would say is the moisture density testing that's been done, although I'm not sure I've got final results on that testing. But I think I might have.
So those test results are applicable to the materials that would be used also for the cement treated soil. The follow-on testing hasn't been done yet, the strength testing that's necessary to be done, the moduli testing hadn't been done yet.

Q. So Phase 3 will include, for the cement-treated soil, strength testing and moduli testing?
A. For the cement-treated soil, that's correct.
Q. You waved your hand when we mentioned strength. Was that a qualification?
A. Well, the strength testing will be done on the soil cement specimens as well, but I consider that part of Phase 4. The durability testing is Phase 3, in my estimation.
Q. Oh, I see. So Phase 3 of the testing program is not applicable to the cement-treated soil --
A. Correct.
Q. -- but Phase 4, the strength and modulus testing, is applicable to both the cement-treated -- no? You tell me, then.
A. Okay. The Phase 4 testing for the soil cement will include the compressive strength testing to demonstrate that we've got at least 250 psi. We're expecting that it's going to be higher than that, more like -- more likely 400 psi, but our design is based on 250 because we felt we could comfortably achieve the 250 based on the data that's presented in the State-of-the-Art Report on Soil Cement.

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Q. So we're talking about true soil cement now?
A. That's correct.
Q. Okay.
A. So the Phase 4 testing of the true soil cement is the stuff around the Canister Transfer Building. That, we need to show the compressive strength exceeds 250 psi. So that's the Phase 4 testing for that material.
The testing of the cement-treated soil, in addition to the compressive strength requirement of 11.1 psi, which is insignificant for the cement-treated soil -- we're basing our design on 40 psi for that value that -- as the lower bound of the value. So -- for the cement-treated soil. So we need to demonstrate that our compressive strength is at least 40 psi to comply with what we state in the SAR for the cement-treated soil. But in addition to that strength requirement for the cement-treated soil, we have modulus limitation. So those specimens, we will measure the modulus of elasticity during compression --
Q. And that's only applicable to the cement-treated soil, the modulus limits?
A. Because of the cask tipover problem --
Q. Okay.

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A. -- right.
Q. In the ESSOW, Exhibit 14, if you would look on page 3, has any information been redacted or blacked out here?
A. I don't know.
MR. TRAVIESO-DIAZ: You're not suggesting he can tell you that from memory, are you?
MS. CHANCELLOR: Well, this is our copy, and it's just got one line and two words on it and --
THE WITNESS: This does not look like my copy, so I don't -- I don't know what happened on that page.
MS. CHANCELLOR: Can I request that you review to see whether we've got a complete copy of this? If there's been any redacted material, I'd like to know the basis upon which it was redacted.
THE WITNESS: Yeah, you could.
MS. CHANCELLOR: That was directed at Mr. Travieso-Diaz.
THE WITNESS: Oh. Excuse me.
Q. (By Ms. Chancellor) If you look at 5.5 of the ESSOW, which is on page 12 under Schedule --
A. 5.5?
Q. 5.5 on page "4."
A. Oh, my God.

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Q. When did AGEC -- let me just read 5.5. "On the premise that notification to proceed will be received by the Contractor not later than February 1, 2000, the laboratory work shall be completed and the draft laboratory testing report shall be delivered on or before March 30, 2001."
A. Oh, your copy doesn't say in the best of all possible worlds? Sorry. That hasn't happened.
Q. When has AGEC received a notice to proceed -- notification to proceed?
A. I don't recall the exact date that they were told to get started, but we've had problems getting that program moving because of the need to update all of our calculations and our SAR documents and the licensing litigation. This program has lower priority than those other items have required, so that's why it's hung up so long.
Q. To the best of your recollection, when do you think Stone & Webster gave the notification to start to AGEC? When did they -- when do you think they --
A. I think it was last spring sometime, but I don't know exactly when.
Q. So the best you can come up with is the spring of 2001?

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A. It might have been March.
Q. About a year ago?
A. Right.
Q. And do you expect the program to be completed in the 13-month time period that is suggested here by the schedule in the ESSOW, from February 1 to March 30?
A. No.
Q. How long do you expect the program to take?
A. Well, it's on hold right now, so it's going to take until we can get it moving ahead again.
Q. Now, why is it on hold?
A. Because we've received some results that have indicated that they didn't compact the test specimens properly. We've brought on board Dr. Anwar Wissa as an expert in soil cement to assist us in evaluating why this could have -- how this could have happened, what did they do wrong that would have caused the densities to be so low?
They're supposed to be within 2 percent of the maximum density from the moisture density tests that are performed in accordance with ASTM D558, the standard test method for moisture density relations of soil cement mixtures. They were off by 8 percent or more in some of these specimens. So clearly specimens

not compacted to sufficient density would not be expected to pass this durability test regimen.
So that's where we are today. We've, as I said, brought Anwar Wissa on board to assist us in moving ahead. And we're currently involved in this litigation so we're not moving ahead on the lab testing, but we will sooner -- as soon as time permits.
Q. Do you have concerns about the ability of AGEC to conduct the test program to Stone & Webster's satisfaction?
A. No, I don't. The AGEC is in the business of performing geotechnical testing services. I'm sure they've been audited by the -- I don't know the correct name of the group that does the auditing of geotechnical labs, but I know there is one that does that in accordance with ASTMs for that purpose. And I -- I expect that AGEC complies with all those requirements and can follow procedures to get these tests done.
So I think they can get there, I just think that they had a bad day, you know? I mean, you know, one of the possibilities could be that they didn't -- they did not compact the specimens quickly enough to get the density that they needed, so this is some -- one of the things that we'll be looking at when we get

moving ahead again with this program.
Q. When the program does move, how long do you anticipate it will take to complete?
A. It's going to take a while yet because it involves another round of durability testing that's 12 cycles of 48 hours per cycle, minimum, so that's -- that's at least a month's worth of testing there, not counting weekends. Could be six weeks to get that done.
The compression test specimens have to be compacted with the right recipes and then cured. I don't recall right now what the cure times are, but they're at least 7 days. They may be 28 days.
Q. So this is Phase 2 of the testing; is that correct?
A. That will be Phase 3, the durability is Phase 3, the compression tests --
Q. The moisture density is Phase 2, right?
A. Right.
Q. And --
A. That we're comfortable with. That's been done.
Q. And have you received results from the moisture density --
A. Yes.

Q. -- and indexing?
A. And -- yes, the Phase 1 property index testing I have results for.
MS. CHANCELLOR: And could we obtain copies of those results?
MR. TRAVIESO-DIAZ: Well, the testing program, as such, is not complete until you get results that reflect the various tests that are being run. I don't believe that either the Phase 1 or any of the other phases have now been reviewed and approved by QA or it has been formally submitted to Stone & Webster. It is a just ongoing, in-process work.
MS. CHANCELLOR: Could you check -- Mr. Trudeau testified that he is satisfied with the indexing, Phase 1 and Phase 2 of moisture density parts of the test program. I would like to request copies of whatever Mr. Trudeau is relying upon to make that statement, to support that statement.
MR. TRAVIESO-DIAZ: Well, if you are asking for the materials that Mr. Trudeau has reviewed as such, those materials can be provided. If you're asking on the representation that these are formal test results that have been reviewed by everybody else including but not limited to Mr. Trudeau that has to approve the results of the program, that I cannot

supply because I don't believe it exists. I think I explained that.
MS. CHANCELLOR: I would like the former, anything that Mr. Trudeau is relying upon to say that he is satisfied with Phase 1 and Phase 2 of the cement test program.
MR. TRAVIESO-DIAZ: Okay. So we are clear, you're asking for the material that Mr. Trudeau has reviewed that has led him to believe that he's satisfied with the results of Phase 1 and Phase 2. Is that what you're asking for?
MS. CHANCELLOR: That's what I'm asking for.
MR. TRAVIESO-DIAZ: All right.
MS. CHANCELLOR: If and when it has been QA'd and it has gone through all the formal review, if it is at that stage, I'd like a copy of that too.
THE WITNESS: I expected to assemble all of these phases' results into a complete report that would be issued to the NRC and the world, but --
Q. (By Ms. Chancellor) That would be post-license, correct?
A. I don't know.
Q. At the rate it's going, do you anticipate that it will be by April 1 when prefiled testimony is

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due?
Okay. So --
A. She's got a mean sense of humor, doesn't she?
Q. So Phase 1 and 2 you're satisfied with.
Phase 3, because of the -- of failure to compress the samples or whatever, part of Phase 3 or all of Phase 3 has to be redone?
A. Correct.
Q. And can you give me a ballpark estimate of how long that will take?
A. It will take at least four weeks from the day we start to maybe as much as six weeks because of the 12 cycles at 48 hours per cycle for the test, plus probably a week to create the specimens. So we're talking between four and seven weeks, it seems to me, for the durability tests to be repeated.
Q. Okay. And then Phase 4, from when you start that or when you start writing the specs for that, how long do you anticipate that that will take?
A. I would guess about a month, depending on the cure requirements, again. There may be a 28-day cure requirement which would delay it another month. But the actual testing itself is not that -- doesn't take that much time. It's -- the samples can be set up

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rather quickly, but they've got to be cured for a period of time. And then once they've cured, it doesn't take long for the tests to be performed and the data to be presented.
Q. Does the one-month time period take into account --
A. The curing?
Q. -- any curing that may be required?
A. No.
Q. Okay. So go to whoa, from the beginning of Phase 1, including the curing, about how long is that going to take?
A. The compression testing phase will probably take two months, one month for the setup and curing and another month to get the testing done and the results produced.
MR. O'NEILL: Can I ask a question just quick?
With respect to the four to seven weeks, you had mentioned that was concerning which phase?
THE WITNESS: During the durability testing phase, Phase 3 I'm calling that.
MR. O'NEILL: Phase 3, durability? Okay.
Q. (By Ms. Chancellor) Is there any other type of strength test planned besides compression?

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A. Yes.
Q. And what is that?
A. Some direct shears testing.
Q. I've heard that terminology before. And when will that be done?
A. After we get the recipe ready.
Q. So that will be at the end of the soil cement testing program?
A. It will follow Phase 3, definitely. It may be able to be done in parallel with the compression testing.
Q. Okay. So for the compression testing, we have two months.
And what about the modulus testing, isn't that part of Phase 4?
A. It's the -- for the cement-treated soil testing, right.
What's the question?
Q. How long is that going to take?
A. How long? That will also require curing, which I think will be a 28-day period. It may be another month -- you know, it's a couple months to three months kind of time frame, would be my guess.
Q. And --
A. But that can be done in parallel too.

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Q. That was my question. So you can do the compression and the modulus testing at the same time?
A. In parallel.
Q. Okay. So all told, including the modulus testing, we're looking at about three months for Phase 4?
A. Sounds about right, yes.
Q. And about almost two months for Phase 3, four to seven weeks?
A. Yes.
Q. And is there a Phase 5?
A. I don't remember right now.
Q. What happens at the end of Phase 4? Are you done?
A. At the end of Phase 4, we'll know that we've got a soil cement recipe that meets the 250 psi requirement for strength and the durability requirements. So for the Canister Transfer Building soil cement, yes, we'll be done. For the cement-treated soil, we need the modulus limitation met, and we need the bottom end of the 40 psi strength met. So --
Q. It will be done after Phase 4?
A. Perhaps. The direct shear testing will be to test the interface strengths between these various

materials.
Q. Is that where you talk about the test similar to DeGroot?
A. Correct, the bonding study. And --
Q. And is that part of this ESSOW?
A. Not part of this ESSOW yet, but it's part of the work that needs to be done.
Q. Phase 5?
A. I guess.
Q. And how will that study be conducted?
A. We will get samples of the dirt from the site and mix it to the recipe that we've identified and bond concrete to the top of that soil cement -- I mean, cement-treated soil mixture and cure it and then test it for strength to confirm that we've got the strength we needed and do the same thing for that cement-treated soil mixture cured on top of undisturbed samples of this clay that we'll have to obtain from the site. We're planning to get some block samples to do that.
Q. Do you consider this proving your design through all these testing?
A. It will -- it will prove the design.
(A discussion was held off the record.)
Q. (By Ms. Chancellor) Getting back to the ESSOW, the Scope of Work, paragraph -- second paragraph

where it talks about samples will be obtained by others, are they the bucket samples --
A. Correct.
Q. -- that you referred to?
A. That is correct.
Q. Gradations will be performed. By whom?
A. AGEC.
Q. Okay. Same with Atterberg limits shall be performed?
A. Correct. That's the Phase 1 testing.
Q. Moisture density freeze/thaw, wet/dry compressive strength, that's AGEC, correct?
A. This whole ESSOW is AGEC.
Q. But it's not -- maybe I'm worrying this to death, but it doesn't say who's doing it.
A. This is the scope of work for this ESSOW so --
Q. It doesn't say AGEC shall conduct Atterberg limits.
A. It says AGEC, on the cover, is doing this work.
Q. Tensile strength -- tensile strength -- I can't say that word -- is that going to be performed by AGEC?
A. That was intended at the time, yes.

Q. And it's no longer intended?
A. Well, I don't know. That's part of what we've got Wissa on board to help with. You know, at the time I thought that -- based on the previous depositions, that it would be worthwhile to get some tensile measurements, but as I've indicated today, I don't believe that it's important to the -- to the -- our design that we have tensile measurements of this material. We're not relying on the tensile strength of this stuff.
Q. So tensile strength is on hold, you don't know whether you'll do that or not under this?
A. Correct.
Q. Permeability tests?
A. Same.
Q. On hold?
A. Yes. The whole program's on hold, but, yes --
Q. I mean -- I mean --
A. -- yes.
Q. -- in terms of whether it will be included in the program.
A. Correct.
Q. And the compressive strength relates to both soil cement and cement-treated soil, correct?

A. Correct.
Q. If you do tensile strength and permeability tests, if you do decide to do those, will that be for both the cement-treated soil and the soil cement or would it be for one or the other of them?
A. Yes. I would think that we might be doing them only for the soil cement if we -- if we do them.
Q. In the third paragraph it states, The engineers shall specify the testing process, including the percentages of cement to be tested. What does this mean, specify the testing process?
A. Well, it means which samples of the test pit buckets we want to have tested, how much cement we want put into these, what types of tests we want performed on each of these different buckets.
Q. And you testified that Dr. Wissa is involved in this testing program --
A. He is --
Q. -- or assisting in the testing program?
A. Correct. He's been retained as a soil cement expert.
Q. And is he being retained by -- to assist Stone & Webster?
A. Correct.
Q. And --

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A. He, by the way, is the same Anwar Wissa that's on the committee that issued the state-of-the-art report that we talked about earlier, the ACI 230.1R-90.
Q. And how have you used Dr. Wissa to date?
A. We've had discussions of the Utah QQ --
MR. TRAVIESO-DIAZ: Excuse me. You are instructed not to refer to any conversations with or for counsel. So to the extent you describe what Dr. Wissa has done, his work on behalf of performance of the test program, as opposed to any litigation-related activities.
MS. CHANCELLOR: Unless you're relying on litigation-related activities as part of his soil cement testing program.
THE WITNESS: You know, I think I might have misspoken. Isn't Wissa retained through Shaw Pittman?
MR. TRAVIESO-DIAZ: I do not recall how, but, again, bearing clearly the distinction in mind that to the extent Dr. Wissa has provided support on behalf of litigation or for litigation-related activities, you are instructed not to refer to those. To the extent Dr. Wissa has provided help with the definition of performance of future work in the program

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itself, you can speak to that.
MS. CHANCELLOR: And also whether he has critiqued the work that has been done to date.
Q. What technical assistance has Dr. Wissa provided to you?
A. I'm a little confused as to what I can say --
Q. Why don't you start, and if you get into an area that you -- that Mat is uncomfortable with, I'm sure he will object.
A. Okay. He's reviewed what we propose to do. It's my understanding that he has no problems with what we've proposed to do, that clearly this is going to work. This is not some esoteric application of soil cement, that it will, indeed, provide and we will, indeed, be able to demonstrate the bonding that we're saying we'll be able to get between the concrete pad and the soil cement and that we'll be able to get the interface strength within the layers of soil cement or cement-treated site to be greater than the strength of the in situ clays and that we will be able to demonstrate the strength of the bond between the cement-treated soil and the underlying clayey soils.
Q. This is the DeGroot-type --
A. Correct.

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Q. -- issues?
A. The bonding study stuff.
Q. What about the -- has Dr. Wissa commented or had any involvement in the AGEC testing aspects of the soil cement?
A. I've shown him the results that we've received to date, and he agrees that these durability tests likely failed because the densities weren't correct. And he suggested that perhaps the densities weren't correct because there was a delay time between mixing the specimens and getting them compacted during the operation at AGEC. So that's one of the things that we need to confirm doesn't happen in the -- in the rerun of the -- retest of those durability tests.
Q. And have you used or will you use Dr. Wissa to refine the various phases of the soil testing program under AGEC? You have four phases --
A. That's what I expect to happen, yes.
Q. Has he refined any of those phases to date?
A. No.
Q. Is there any -- other than this ESSOW, is there anything -- any one document that comprehensively describes the various phases and total extent of the soil testing program?
A. Not clearly identified as phases that we've

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been talking about here, but the SAR describes all of the testing that we're planning to do.
Q. Okay. So in terms of a comprehensive description of the soil cement program, we would look to Section 2.6.4.11 of the SAR?
A. Correct.
MR. TRAVIESO-DIAZ: In the last question you went beyond what is in the ESSOW.
MS. CHANCELLOR: I beg your pardon?
MR. TRAVIESO-DIAZ: In your last question you went beyond what is in the ESSOW.
MS. CHANCELLOR: I'm sorry. I didn't understand --
THE WITNESS: Beyond.
MR. TRAVIESO-DIAZ: Beyond what is in the ESSOW. Your question, if I recall, was is there a comprehensive document that describes what will be done, right?
MS. CHANCELLOR: My question was is there a comprehensive document that describes PFS's soil cement program. I don't think I limited it to testing, just the soil cement program.
MR. TRAVIESO-DIAZ: Oh, okay.
Do you understand the question now?
THE WITNESS: The best description of the

soil cement testing and construction program is in the SAR.

Q. (By Ms. Chancellor) And to --

A. Chapter 2.6. There may -- I think there's another section as well that discusses soil cement but --

Q. Certainly.

(A discussion was held off the record.)

THE WITNESS: Certain aspects of the soil cement are also discussed in Section 2.6.1.12, Stability of Foundations for Structures.

Q. (By Ms. Chancellor) Could you give me that cite again?

A. 2.6.1.12. But the best description is this 2.6.4.11.

Q. In response to Interrogatory No. 3, you state that you've retained Dr. Wissa as a consultant to assist in the soil cement program. Is there an engineering services scope of work for Dr. Wissa?

A. Not at this point, but we expect that his firm will be doing some of the -- like the interface strength tests for us, so there will be an ESSOW to lay out that program. And we're -- at this point we're expecting that his company is going to be doing that testing.

Q. But is it correct that the testing that Dr. Wissa will do would follow Phase 4 of the AGEC's soil cement test program?

A. That's -- that's correct. He may do the Phase 3 work on the cement-treated soil. I don't know yet. That was the modulus testing, you know, the --

Q. We called that Phase 4, but it's really Phase 3.

A. For the cement-treated soil. It's the next phase for the cement-treated soil.

Q. Cement-treated soil?

A. If you're more comfortable with Phase 4 --

Q. No, that's fine. I just didn't want the record to be unclear.

So that's the modulus and the --

A. Compression --

Q. Compression --

A. -- testing of the cement-treated soil, because that's the same material that we're going to be running these interface strength tests on that we're anticipating he will be doing for us.

Q. Will Dr. Wissa also be doing direct shear tests?

A. It remains to be determined what the interface strength test is going to look like, but I

think it wants to be a direct shear test because we want to force failure along that plane. So I think, yes, they will be direct shear tests.

Q. So is it correct to say that the direct shear test and this DeGroot-type testing, we're only talking about the cement-treated soil under the pads?

A. Correct.

Q. Once you go through all this testing, the way in which the construction is done of the soil cement, will that have an effect on whether the soil cement will perform as intended or the cement-treated --

A. Well, construction techniques can have effects that would be detrimental to the performance of soil cement, but those need to be controlled during construction so that we produce the interface strengths that we're looking for, that we're relying on.

Q. And do you anticipate that you'll use Dr. Wissa to develop any construction procedures or QA/QC measures?

A. I expect he will participate in the development of those.

Q. And when do you anticipate that those procedures will be written up?

A. Following this laboratory testing work.

It's further down the road.

Q. And are any of these -- any of the general outlines of the construction procedures and QA/QC measures for the placement and construction of the soil cement, are any of these found in the SAR? Is there any discussion at all of construction procedures or QA/QC measures for construction?

A. I suspect there is in 2.6.4.11, but I don't know. I will check.

Construction techniques are described somewhere in here. Whether the QA aspects of it are clearly delineated, I'm not sure.

It says on page 12.6-118, for instance, Procedures required for placement and treatment of the soil cement lift surfaces and foundation contact will be established in accordance with the recommendations of ACI 1998 during the mix design and testing process. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during this detailed design phase of the project.

Q. And on page 2.6-113 of the SAR, the last paragraph, it mentions that soil cement has been used extensively. Is this true soil cement or are we talking about cement-treated soil, do you know, in

this --
A. **It's true for both, but this, I think, is referring to soil -- true soil cement.**
Q. And the examples given here, the South Texas Nuclear Power Plant near Houston and the nuclear power plant in Koeberg, South Africa, was soil -- if you know, was soil cement there used because of liquefaction?
A. **In South Africa, that's correct.**
Q. In Texas was it used to provide additional -- you objected to the way in which I rephrased it -- to provide sliding resistance?
A. **I do not believe it was used to provide sliding resistance at the Texas plant.**
It says in the SAR here that at the south Texas plant it was used as slope protection for a 7,000-acre cooling water reservoir.
Q. So are these examples of soil cement providing -- do you know of any examples of soil cement used to provide sliding resistance?
A. **No.**
MS. CHANCELLOR: Can we go off the record for a moment?
(Lunch recess was taken.)
Q. (By Ms. Chancellor) Okay. I'd like to now

turn to the native soils underlying the soil-treated cement under the pads. Now, you've testified earlier today that the top layer of soil in the pad emplacement area are eolian soils, correct?
A. **Correct.**
Q. And that PFS is going to remove those eolian soils and mix these soils with portland cement?
A. **Yes.**
Q. And then the cement-treated soil will then be directly beneath the pads?
A. **Correct.**
Q. Do you agree that the soils directly below the cement-treated soil are partially saturated silty clay/clayey silt?
A. **Yes.**
Q. For purposes of this discussion, can we call the silty clay/clayey silt upper Lake Bonneville deposits?
A. **Certainly. That's so much easier.**
Q. Especially for the court reporter.
What role, if any, does adhesion and cohesion of upper Bonneville clay play in providing the slide stability of the pads and the CTB foundations, according to the calculations you've performed?
A. **It provides the resistance we need to keep**

the pads in place.
Q. Is adhesion and cohesion important, then?
A. **Yes.**
Q. Do you believe that the upper Lake Bonneville deposits are partially saturated?
A. **Yes.**
Q. Do you have an opinion on whether there will be any change in the moisture content of the upper Bonneville deposits when the cement-treated soil is placed on top of them?
A. **Yes.**
Q. And what is that opinion?
A. **I understand that there's a concern that the soil cement to be placed at the site may serve as an impermeable barrier that will permit moisture changes in these soils, but I have a hard time believing that that's going to be a big problem for these soils because of the great depth to the groundwater table at the site -- it's down 125 feet -- and because of the semiarid conditions out in Skull Valley. I think we're talking like less than 8 inches of rainfall per year, most of which will not be able to permeate through the soil cement cap. So I just have a hard time understanding the proposition that we're going to have a moisture change problem in those soils.**

Q. Now, do you agree that Skull Valley is in the basin and range?
A. **Yes.**
Q. And have you worked in -- have you done any geotechnical work in the basin and range area?
A. **Not prior to this project.**
Q. Do you have an opinion, and, if so, what is it, on whether the construction processes will impact the Bonneville deposits?
A. **I understand and expect that the construction techniques to be used have the opportunity to destroy the surface of the subgrade if we're not careful in protecting those. There are -- there are a variety of construction equipment available that can, indeed, destroy the cohesion that's inherent in these soils. But clearly, where the cohesion available in these soils is required as a design -- part of the design of these pads, we need to protect those soils during construction, and we need to demonstrate at the start of construction that the techniques that we're using will not have an adverse impact on the strength of these soils.**
Q. So is it the equipment or the techniques or both that can destroy the cohesion?
A. **It's both.**

Q. And I think you testified earlier that any sort of construction procedures and QA/QC methods will not be developed until --

A. Later in the design process. But -- but it's not -- I mean we're talking about the pads at this point where we need the cohesive strength of this clay as -- for the soil cement on top of the -- cement-treated soil, actually to be bonded to this layer, so it's that subgrade -- the top of that subgrade at the end of the excavation directly under the pads that's the concern.

These pads are not that big. They're 30 feet wide. There is construction equipment that can sit on either side of these pads and reach out to make a cut to the final subgrade surface. And all other construction equipment can be -- all construction equipment, period, can be kept off of the exposed subgrade. So I'm convinced that we can get that subgrade protected sufficiently so that we're not destroying the strength of that material when we're building this.

The exposed subgrade doesn't want to stay exposed either, so the construction procedures will require that that final excavation doesn't take place until they're ready to put that first lift of

cement-treated soil down to protect it. And that lift of cement-treated soil can be pushed out onto the surface of the subgrade with low ground pressure equipment that won't have an impact, an adverse impact on the underlying clay. And in that manner we can ensure that we don't destroy the cohesion that we need and that we can develop the bond that we need.

Q. . But if the eolian silts -- if the clay layer doesn't come to the grade level that you anticipate, you'll need to put construction equipment in the pad emplacement area to compact the silts that are there, correct?

A. For the -- for the few minor areas on the site where we might require more than 2 feet of cement-treated soil under the pad, in that area we would have to put in a compacted clay material, a low plasticity clay material, which we will have to demonstrate by laboratory testing that that compacted clay will have the cohesion that we need underneath the cement-treated soil.

And that will have to be done by equipment placed in the hole where the pad will be constructed, yes, but that -- that process will not result -- I mean the clays that we're talking about using will be the same materials that we're trying to protect in the

other areas. Those -- those are stiff clays now that we're expecting we will be able to use -- we'll be able to test some of those in the lab to show that we can compact those and get the strengths that we need so that the compacted clay surface will provide the cohesion that we need under the cement-treated soil. So if they -- if the equipment that we're using to put this new clay fill in damages the surrounding area, the surrounding area will end up being compacted along with this other clay area.

Q. How --

A. It can be -- you know, the compacted clay is going to have sufficient strength to resist the sliding forces that --

Q. How will you know whether the surrounding clays to those that are being compacted will be affected by the equipment?

A. Well, it will be obvious that they've been destroyed by the -- just by looking at the stuff. I mean it's -- the material is a very stiff clay right now, and if you work it enough, you can remold it to a point where you can't -- let me rephrase that. If it gets remolded or worked up by the equipment, it would be obvious that it's in a condition that's not suitable.

Q. Okay.

A. Okay?

Q. Do you agree that a change in water content of the Bonneville clays will affect the settlement strength and adhesion between the soil and the cement-treated soil?

A. I do not believe the water content change would affect the settlements of these materials. We have performed consolidation tests dry on these specimens -- not really dry but, in the in situ moisture content, and we've performed tests on comparable samples of this soil with complete inundation and not noted any marked change in the settlement for those inundated samples with respect to the non-inundated samples. So I don't believe it will affect the settlements at all. It's possible that a moisture change could affect the strength of the soils.

Was there more to that question that I don't recall?

Q. Adhesion.

A. Adhesion? As the strength might be affected, the adhesion might be affected.

Q. And will the strength be less?

A. Less, yes.

Q. And the adhesion will be less?

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that you're going to excavate from the top of the pad emplacement areas?
A. **The eolian silts, yes. The material that had the higher sulfate is not that material, it's the upper Bonneville --**
Q. Oh, the upper Bonneville.
A. **-- clay material that we won't be using --**
Q. I thought you said both.
A. **-- that we won't be using --**
Q. Okay.
A. **-- in making soil cement or cement-treated soil.**
Q. Okay.
A. **That's the material that we would likely use as the compacted clay soil in those few areas where we might be low.**
(A discussion was held off the record.)
Q. (By Ms. Chancellor) Have you performed or are you going to perform any testing regarding the potential interaction of the cement-treated soils with the native soils?
A. **Yes.**
Q. And when and to what extent?
A. **That will be part of the interface strength testing program that Wissa will be doing for us, as I**

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said earlier. We're expecting to go to the site, get some block samples of the -- these upper Bonneville clay soil subgrade to take to Wissa's lab, and he would make the cement-treated soil mix and place it, compacted, on top of this block sample and cure it and then run the direct shear test, I think, to measure the interface strength available.
That testing is -- I described in the SAR. It's not in the ESSOW yet, as we said earlier, but it is in the SAR.
Q. When do you anticipate you'll develop an ESSOW for Wissa?
A. **I don't know for sure but within the next month or two would be my guess. I don't know because I don't know how much of my time is going to be dedicated to getting ready for the hearings and my other commitments. But I've got to get together with Wissa at a time convenient for him and me and -- when the project's ready to move ahead with that activity. These other items are obviously higher priority.**
(A discussion was held off the record.)
Q. (By Ms. Chancellor) Moving on to a different area, just so you're not wondering if it has anything to do with native soils, what's your understanding for the regulatory basis for the factor

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of safety against sliding and overturning, first, for the pads and then for the CTB?
MR. TRAVIESO-DIAZ: What do you mean by the regulatory basis? I believe the question is vague.
Q. (By Ms. Chancellor) In the SAR, for example, on 2.6.120, you state that, The minimum factor of safety against a bearing capacity failure from static loads is 3.0, from static loads plus loads due to extreme environmental conditions such as design basis ground motion is 1.1.
What is your understanding of the regulatory requirement relating to the minimum factor of safety against sliding in extreme environmental conditions as being 1.1? Where does that come from?
A. **I believe that comes from NUREG-0800, which is applicable for nuclear power plants. As I discussed earlier, nuclear power plants, they're concerned that the structures don't slide typically because there are Category 1 piping systems that need to be protected between the structure and the yard area. So they're anxious for the nuclear power plant structures to make sure that the structures don't slide. And for the earthquake loads they accept a number like 1.1 as evidence that the building won't slide during the event.**

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Now, those -- NUREG-0800 does not apply to these ISFSIs. NUREG-1567, I believe, does.
Q. And when you mentioned NUREG-0800 having the 1.1 factor of safety, were you referring to the CTB or to the -- realizing that --
A. **Well, that's for structures -- that's for structures at a nuclear power plant.**
Q. Do you consider the pads to be a structure?
A. **It is a reinforced concrete pad --**
Q. For purposes of meeting a 1.1 factor of safety against sliding, do you consider it to be a structure?
MR. TRAVIESO-DIAZ: Objection. He has not testified that the 1.1 factor for sliding applies to the pads.
MS. CHANCELLOR: He says that he looked to NUREG-0800, realizing that it was the nuclear power plants, but that's where the 1.1 factor of safety comes from. And I'm asking him was he referring to the CTB only or the CTB and the pads, and I'm trying to figure out how he categorizes the pads.
THE WITNESS: We -- we use the 1.1 as the target factor of safety for sliding for this facility, realizing that the 1.1 applies to structures at a nuclear power plant, understanding that that number

In The Matter Of:

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State's
Exhibit 109

clarify your question?

BY MS. CURRAN:

Q: Do you agree with the statement that that's made —

MR. TRAVIESO-DIAZ: Do you mean the entirety of the statement?

MS. CURRAN: Yes.

BY MS. CURRAN:

Q: You can break it down, if you

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want.

A: Let me read it, please.

Q: Sure.

A: No, I don't necessarily agree with this.

Q: Could you go through and explain? Maybe you want to break it up be sub parts.

The applicant has not considered the impact to native soil caused by construction and placement of the cement-treated soil?

A: Well, I think there's been some discussion addressed about how they're going to possibly construct it, and not disturbing the soils, and things like that. So they have be considering that aspect of it.

Q: If we inserted the word "adequately" after "not," would you still agree with that first part of the statement that I just read?

A: No. I wouldn't agree with that.

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Q: Why not?

A: I think for the stage of development of this project, I think it's been adequately addressed.

Q: But for purposes of actually building the facility, it's not adequate?

A: For actual construction, that's correct.

Q: If you look at the second phrase, which says that the applicant has not analyzed the impact to settlement, is your opinion similar, that some information has been gathered, but not enough to approve the construction of the facility?

A: Repeat that.

Q: If we look at the second phrase here, whether the applicant has analyzed the impact to settlement, would you agree that some information has been collected?

A: Yes.

Q: Do you consider that the amount of information that has been collected is

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adequate for purposes of going ahead with construction?

A: No. It's not adequate.

Q: And I have the same question with respect to the last part of that sentence, which refers to adhesion properties.

A: Yes. It's the same answers.

Q: Is there any aspect of the issue of the design of soil cement or cement-treated soil for which you feel or you believe that the applicant has obtained sufficient information in order to proceed with construction?

A: No. I don't think it's enough to proceed with construction, no.

Q: Dr. Wissa, is there a standard formula for soil cement?

A: A standard formula?

Q: Yes.

A: Can you explain what you mean by formula?

Q: Well, you know exact proportions

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of every ingredient that goes into it and what they are?

A: Well, we know what ingredients go into it. But the proportions, we do not know.

Q: And there's a difference between soil cement and cement-treated soil; is that correct?

A: It's a degree of stabilization and durability. Its the same concept. But it's just a degree of stabilization.

Q: So that cement-treated soil does not have the same degree of stabilization and durability as —

A: Well, that's the way you are trying to interpret it. I think the nomenclature is vague. But I think that's generally accepted today as not being as durable.

Q: I want to ask you a little bit about your understanding about the way that soil cement and cement-treated soil are to

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be used at the PFS facility.

Am I correct in understanding that the cement-treated soil is going to be directly underneath the concrete pads for storage of the casts?

A: Yes.

Q: Will the cement-treated soil extend beyond the perimeter of casts laterally at all?

A: I'm not sure. I don't think so. I think that beyond that, they're going to use what you call cement stabilized soil. But I couldn't swear to that. I'm a bit vague about it. But I believe it's primarily under the — I don't know the answer exactly. I can't recall. It's there somewhere in the —

Q: And do you know, taking the soil cement that's going to be around the edge of the pads, how far out will it extend beyond the edge of the pads? Do you know?

A: Well, the pads — now speaking of

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the stabilized, or the soil treated?

Q: The soil cement?

A: The soil cement?

Q: Yes.

A: It connects one pad to the next one. So it is within the distance between the pads. And I don't recall the exact clearance between them. But it extends from one pad to the next pad.

Q: And at the outer perimeter, how far does it go out?

A: I don't recall. But I assume it goes out to some distance. I don't know.

Q: And do you know how far it extends beyond the perimeter of the canister transfer building?

A: I know it's quite some distance. It's not speaking tens of feet, but probably a hundred or more.

Q: What is your understanding of how construction will be carried out with respect to the soil cement and

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cement-treated soil?

A: How it will be carried out I think will have to be left to the contractor and the availability of his equipment and his experience. I think, to me, is how it will not be done. By that, is that certain things should be in the specifications of construction that you would not allow him to do.

Q: And what are they?

A: Well, for example, you will minimize disturbance of the subgrade of the excavation. You will minimize it from getting exposed to the elements. You will not allow it to be reworked. Things like that, things which — it's more of a preventative than telling him how he is to do his job.

And he will come back, as I would see it, with his concept. And then one would agree with it or say it doesn't meet with the objects of — and I'm just giving

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you one example in the case of the subgrade of the excavation. And what he's going to do is going to cause disturbance and damage those subgrade.

I'm calling it the subgrade. But it's the bottom layer, say, the way you're going to start placing your soil cement, for example.

Q: You're talking about the clay silt, silty clay? That's the subgrade?

A: That's correct.

Q: Why would you want to minimize disturbance to the subgrade?

A: Because you don't want remolding and the possible loss of strength will increase compressibility.

Q: And what affect, if you were to lose stress and compressibility, what would that affect?

A: Well, I don't know at this time. Because we don't know how sensitive these soils are to disturbance. Okay. This is

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hypothetical. I think that once we know this, we will be in a better position to either be flexible or more rigid on what he can or cannot do.

Q: But in terms of why you would worry about this, is it because if you were to disturb the subgrade, that it might be less resistant in an earthquake?

A: I think to answer you, first of all, I'm not as much concerned about settlements as about loss in strength and, therefore, its ability to have the shearer resistance for this lateral movement which we're relying on.

Q: And you also mentioned exposure to the elements.

Why would that be a concern?

A: Well, in a similar way. If you got a lot of rain and the whole site was open, you would have it flooded, maybe if it was a heavy rainfall for a long time. Then it's probably more a problem of efficiency.

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Because then you'd have to let it dry out substantially before you'd want to start construction again. So there is a practical problem of it, too, of exposing it to the elements.

Q: So during construction, what will be done here is, equipment will be set up for mixing soil and cement; is that correct?

A: Yes.

Q: And it will be mixed right on site?

A: Yes.

Q: And will it be mixed in place or done off to one side? Or can you give me a picture of how that's going to happen?

A: Well, I think here it's going to be a function of a contractor, his ability, his experience, and so on. There are two approaches to it. One is mix in place. And the other is plant-mixing it; in other words, you hold material away. You put it into a central plant, mix it, and hold it

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back, and place it.

Q: And you don't know which one will be used?

A: Not at this time, no.

Q: Does it matter which one you use, in terms of the impact on the subgrade?

A: If you can achieve the quality control, no, it wouldn't. Everyone has their preferences.

Q: Which one do you prefer, and why?

A: I prefer the central plant mixing. You have better quality control on the amount of cement, the amount of water, the mixing, than mixing in place.

Q: So you —

A: But you could — a good contractor with the right equipment could achieve the same by mixing in place.

Q: Why do you say a good contractor? It's harder to do, to mix in place?

A: I would say it takes more experience for a contractor to mix in place

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than to haul it away and have a plant there which does it. There's less human influence.

Q: I would think that, to just say it another way, that there's more of an impact on the site if you're mixing it in place, because you have more heavy equipment that's right? There is that fair to say?

A: No. We are talking about an interesting situation, unlike a highway where you have miles of it. These pads are fairly small. The quantity of soil cement is not large per pad. And, therefore, you could do one pad at a time. And you wouldn't need a lot amount of equipment moving around in place. So I don't think that's a main issue.

Q: Have you done this before? Have you supervised this process of mixing soil and cement and making soil cement?

What do you do if the soil is too wet?

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A: You have several options. Obviously, one option which is usually done is you work it, pulverize it, and have it dry out. Another thing, in some instances you may want to add quick line or something to dry out the soil. But then you change its properties. But that is a method of improving the soil, making it easier to work. That's two which come to mind. I think those are probably the most common ones.

Q: If you used the first method, you dry it out first and then you pulverize it, where do you do that?

A: You are taking the — I'm sorry — you're taking the soil and excavating it, stockpiling it. And now, if it's wet, you will work it, spread it, out, let it dry out. That's not in the location where you're going to be compacting it. It's not in the location of the pad itself. Because if you did that, you would disturb the whole

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area. You would haul it away or spread it somewhere, and then put it back in after it reaches the right moisture content, and mixed in with the cement.

Q: So we're talking about a process where you have a backhoe that's digging up the eolian silt, I suppose. And then you are maybe drying it in the pile somewhere on the site, or maybe putting it right on a truck and trucking it out. This is if we go with option A of processing it off site.

Then it gets taken to another plant, and portland cement is added and its put into a cement truck?

A: No.

Q: What happens then?

A: Well —

Q: I'm showing my ignorance.

A: No. The cement truck, you wouldn't be able to pour it. If you used a cement truck, I think you would have too wet a mix to be able to pour it back in. What

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you do is — you're right, to some extent, that you take it to the central plant. You'd probably stockpile it there, have moisture equilibrium, so you don't have a bucket of wet, bucket of dry.

Then you put it into the mixing — let's say tank if you want. It could be a continuous process, or it may be a batch process. You would add the cement, and the water, mix that up, and then put it in trucks, and haul it back to where you want to place it.

Q: You don't have to keep spinning it around to keep it from hardening?

A: You don't — well, you do work — if you're going to delay, it depends on the time between mixing the water and final compaction. If it's going to take along time — by "long time," I'm saying a couple of hours — and if it's hot water, you'd probably want to work it during that period. But preferably, you'd want to place it as

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soon as possible and not have to rework it.

Q: When you do the mixing in place, what kind of equipment is used in that case?

A: A pulver mixer.

Q: A "powder" mixer?

A: No. Pulver, P-U-L-V-E-R M-I-X-E-R pulverization mixer. They call it a pulver mixer, which is a high-speed Harrow rotating blades which take the soil and break it up first. You have to do this at the right moisture content, so if it's too wet, it gums up. The drier it is, the better you are that way. But if it's too dry, it could get too hard.

But for the right moisture content, you break it up. And then you, at the same time, could be adding the cement, and conceivably also could be adding the water in this pulver mixer. Or you can do it in several passes. You first break it up. Then you add the cement, mix that in. And then you come again, add the water, mix

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all that in, and then come back.

Q: And you're using a Harrow, like an agricultural machine?

A: Well, it's a little more — it's high-speed blades which break up the material and mix it. So it's not a Harrow. Harrow is the wrong word. Harrow is more just rotating it. It's breaking it up by high-speed rotation of cutters. Or they're high-speed meaning, yeah, spinning.

Q: And this machine, let's call it the high-speed Harrow.

A: Okay. Let's call it that.

Q: We'll just call it that.

A: I call it the pulver mixer.

Q: The pulver mixer?

A: Yeah.

Q: Is it a heavy piece of equipment?

A: Not essentially, no.

Q: How heavy is it?

A: Depends on the size and so on. In this case, these are a lot smaller areas.

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It wouldn't be very heavy equipment.

Q: Would you foresee it having any kind of an impact on the subgrade by sitting on top of it?

A: Well, let me back off a bit. I have a hard time seeing that you could take two feet of material and in situ mix two feet and recompact it in one layer and go efficiently. I think you'd have to move it beside where you're going to place it, mix it up, and then put it in. So I don't see us really being able to take two feet. I don't know of any equipment which could cut two feet, mix it up well, and put it back in.

Q: Because you would need to be able to cut less, or more?

A: Less.

Q: It's much less?

A: I think the depth of two feet is excessive.

Q: In other words, you don't think

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that's a reason that it's not advisable to do the in situ mixing?

A: I didn't say that. I think the in situ mixing — let me define in situ mixing a little further. In this context, in situ mixing means using the soils close or located in place, and blending it with that type of equipment, the pulver mixer, versus hauling it away, taking it to a central plant, and mixing it. That's what I call in situ mixing.

It doesn't necessarily have to be literally in situ. And you just take it like you would when we say in situ mixing of these deep foundations, where you would mix in place and you put a cement grout and mix in there. I think even in the case of in situ mixing, you move the soil around.

In the highway, they would wind row it, mix it up, and then spread it out again. So it isn't literally just staying there. You do move it around, even in

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highways, when you have what you call in situ mixing.

Q: So just so I understand it, using the pulver mixing, it wouldn't necessarily be that you would mix everything right in the exact same place where it was going to be in the end; the mixing might be done off to one side of the ultimate destination?

A: That's correct.

Q: Would you take out Exhibit 21, which is the SAR chapter two?

A: Yes.

Q: And turn to page 2.6118.

A: Yes.

Q: Can you tell me, looking at the second bullet there, what does it mean when it says, The soil cement will be constructed in lifts approximately six inches thick?

A: When you compact soils, if you have too thick a layer, you end up having inadequate density in the bottom of the layer. So you have to limit the thickness

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of the layer to get adequate compaction. So to achieve two feet, it would be very difficult, if at all possible, to compact it all in one layer. You would have to compact it in several layers. Usually six- to eight-inch is about the maximum you would want to do the compacted layer.

Q: So you do six-inch layers at a time when you —

A: Compacted, yes.

Q: So when you put the material back in the hole, you compact it with some kind of machine?

A: Correct.

Q: What kind of machine is used for that?

A: Well, it depends. It could be a rubber tire compact. It could be a steel drum, smooth tar. Several sheets of — it depends on what the soil is or the soil cement is, and what equipment is available and so on.

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Q: It says here, in the same section as described in section 6.2.2.5 of ACI 1998, These techniques will include, but will not be limited to, minimizing the time between placement of successive layers of soil cement.

Can you explain what is the minimal time between placement of successive layers of soil cement?

A: Well, I think this, you have to be a little careful of what you mean by that. You want to obviously prevent the surface drying out. Okay. If it does, you have to scarify it. And then what you're interested in is achieving a good bond between each layer. So surface drying out is one thing.

Also, if it — if the first layer, let's say — let's say you prevent it drying out by humid curing it or putting a spray on — well, you wouldn't put a asphaltic seal coat, because you want good bonding. You may want to use a plastic, a

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geomembrane, to prevent evaporation losses. But then you do get it curing.

So let's say a week later you come

back and want to put the next layer on, you would have a discontinuity. And, therefore, you would have to pretreat your soil to improve the bond. But you don't want to wait a week. So what we're saying here is you try to do it within a reasonable amount of time.

But let's say the equipment breaks down and you have delays. Then you'd have to do something with that surface to make sure you have good bonding again. What it's saying here, basically, you don't want to wait a week between layers, if you can help it.

Q: Turning to page 2.6-119. If you look at the first full paragraph there, entitled, Soil cement and in situ clay interface, the first statement says, The soil cement and in situ clay interface will

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be constructed such that a good bond will be established between the materials.

Can you explain what is the purpose of that bond?

A: This is a important — well, it's important throughout. The soil cement, it would be under the pads. Because under the building, you have five feet of concrete that we — five feet of concrete, and no soil cement under the building.

What it is, is you're trying the whole objective here of a soil — modified soil or cement-treated soil, is to transfer the shear stresses due to an earthquake down to the clay below. So you want a good bond between the soil cement and clay interface.

Q: And how is that done?

A: Well, what you do want is — most likely, we would add a coating of cement or a cement slurry, a thin — thick slurry. And this is going to be established by a test, what's the best way of achieving a

Page 34

good bond.

And that's where these shear tests plan to determine what's the best way of achieving a good bond between the soil cement and the underlying clay subgrade.

Q: And you used the term "good bond." Is that something that you define quantitatively?

A: No. It's measured. You would measure the — you would cause them to fail. And you would measure the shear strength, or the force required to cause them to slip. And from that, you can say anything — we know what we need as minimum.

Q: What's the minimum that you need?

A: I don't recall what the minimum was. But there is — they have worked it out from the analysis what's the minimum required. I don't know it offhand, minimum shear strength required at these interfaces.

Q: How do you perform that test?

A: There is a — it's a direct shear

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box, you call it. And usually for this type of test, you'd use one which is probably a one-foot-by-one-foot instead of a — you could use a small one. A small one's usually for size two-inch or four-inch-by-four-inch.

But I think in this case you would probably use one which is maybe a foot square. But it could be a four-inch one. And it has two boxes, two boxes, halves. And you pull one with respect to the other. And you measure the resist — or the force required to cause them to slip. So half of the box would slip in one direction, the other half in the other direction.

Q: That seems lick a pretty simple thing to do. You could do that today, right? You could perform that test today?

A: Yes.

Q: To your knowledge, has that test been performed?

A: On this specific job?

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Q: Yes.

A: No. To my knowledge it has not been performed.

Q: Do you know why not?

A: No.

Q: And what are the variables that go into meeting that requirement, that shear strength? Is it the nature of the concrete slurry? Is it the weight of the pad on top of the clay?

What are the things that go into if you change it, it changes the shear strength?

A: Well, obviously, if you change the loads, you change shear strength. But in this case, we know what the loads are going to be. So we're not going to apply much higher loads than that of the slab and the overburden above it, or whatever's above it, the soil cement above it and the concrete slab. And so then you wouldn't use anything above that.

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The other factors are the moisture content; the type of treatment, surface treatment, whether it's dry cement, or is it a cement slurry, or a moist slurry.

Q: Let me just interrupt you there and clarify. When you say the type of treatment, you're talking about the interface between the subgrade and the cement-treated soil?

A: That's correct.

Q: Okay. What else? Does it have to do with characteristics of the cement-treated soil, also?

A: Yes.

Q: What aspects of the cement-treated soil affect the resistance to stress?

A: Well, probably the controlling — obviously, if the soil — the cement-treated soil is the weakest link. It's going to fail through the cement-treated soil. If the clay is the weakest ling, it's going to fail through clay. If the bond is the

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weakest link it will fail through that think layer that we're talking about.

And the idea would be to make sure that the thin layer between the two, or the interface, is not the weak link. That's really the objective of all we're doing here, is make sure it fails either through either the underlying clay or the cement-treated soil. And I suspect it's probably going to be through the clay rather than the cement-treated soil.

Q: It would be possible, wouldn't it, to design the pads so that their thickness was the thickness of the eolian silt; so that, in other words, they would entirely displace the layer of eolian silt and touch the subgrade below?

A: I can't answer that question. Because that's outside my area.

Q: You don't do concrete?

A: Yes, I do concrete. But I don't get involved with canisters tipping over and

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things like that, which control the thicknesses.

Q: Oh, I see. But there isn't any reason, from the standpoint of the stability of concrete by itself, that would prevent PFS from building a pad that was four or five feet thick, as opposed to two-foot thick?

A: I need to understand what you mean by "stability."

Q: Well, disregarding the issue that they're holding casts on top of them, if you were just building a pad out in the desert, would there be any reason that you couldn't design the pad to be five feet thick and go down as far as to touch the subgrade layer?

MR. TRAVIESO-DIAZ: I'm going to

object to the form of the question. Because it assumes something for which there is no foundation, which is that there is a uniformed distance from the surface to the layer underneath. And that hasn't been

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established. What I'm saying is that your question assumes that there is four to five feet uniform distance between the top and the bottom.

MS. CURRAN: Okay.

BY MS. CURRAN:

Q: I'd like to ask you about a statement here also on page 2.6-119.

In the second full paragraph, the first sentence reads, An additional benefit of incorporating the soil cement into the design is that will minimize the environmental impacts of constructing the facility.

This represents that minimizing environmental impacts is an additional benefit of incorporating soil cement into the design.

What's the first benefit of incorporating the soil cement into the design?

A: I can only see what's — state

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what's said here. From what I gather, you're saying, if you read the next sentence, is use of on-site materials to construct soil cement rather than excavating and spoiling these materials is an environmental benefit.

Q: Right.

A: That's what they're stating here.

Q: Right. But it says it's an additional benefit.

So I'm just wondering: Is it a benefit in some other way to incorporate soil cement into this design?

A: I don't know. I did not write this paragraph. So I don't know. I'd have to read back over and see what other benefit was involved in it. This was not my wording.

MS. CURRAN: I'd like to take a ten-minute break.

(Recess)

BY MS. CURRAN:

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Q: I'm going to pass you kind of a bulky item, Doctor. This is a set of some of the exhibits. And I just want to look at one of them, which is Number 13. These happen to be stapled together. And I'd like you to turn the Exhibit 13, which has already been marked: Applicants objections and responses to the State of Utah's 14 set of discovery requests directed to the applicant, dated February 19, 2002.

I believe earlier in the deposition you stated that you had been retained by Shaw Pittman, and not by PFS; is that correct?

A: That's correct.

Q: Well, I'd like you to turn to page 20 of this discovery response. You'll see at the top of the page, this is an answer to interrogatory number three.

It states, PFS has retained Dr. Anwar E.Z. Wissa as a consultant to assist in the soil cement program.

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Is that incorrect?

A: Well, I haven't received any formal contract or information that I have been retained.

Q: Do you have a handshake?

A: An insinuation or a handshake may be the case, but no formal agreement of any kind exists. And as of today, I have not spent any time or billed them or done anything with them to confirm that this is the case. As I said, I would hope it would be the case. But from where I'm speaking to you, and I expect they will retain me, but there is no formal agreement as of this date.

Q: So the phrase "has retained" is somewhat hopeful language?

A: I didn't write this. So whoever wrote this — maybe I should have read this and assumed that I have been retained.

Q: Now, it also says here, PFS anticipates that Ardaman & Associates will

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be performing additional relevant soil cement testing.

Have you been retained to represent soil cement testing?

A: I think it's the same context, where we've discussed it; and they told us can we do this work; and are we willing to, and so on. I've agreed yes. But the physical — or the documentation that we have been retained, I do not have yet. It may be in the mail, for all I know.

Q: Have you had any involvement with PFS's other consultants in the soil testing that has been done?

A: I had a meeting with the lawyers where other consultants were present.

Q: Have you had any involvement with AGEC?

A: No. I don't think so.

Q: Did you participate at all in the engineering services scope of work that we've all looked at as Exhibit 14?

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A: No.

MS. CHANCELLOR: Can I ask a question?

MS. CURRAN: Yes. You're breaking up, Denise. I don't know why.

MS. CHANCELLOR: Dr. Wissa, have you had any conversations with Paul Trudeau at Stone & Webster?

MR. TRAVIESO-DIAZ: I am going to object to having two counsel examine my witness at the same time.

MS. CHANCELLOR: Okay. That's fine. We'll do it at a break. And we'll just go back and Diane can ask the questions. That's just fine.

MR. TRAVIESO-DIAZ: Just one at a time, please.

MS. CURRAN: Well, we were doing it one at a time.

MS. CHANCELLOR: I mean, I was trying to be efficient. And I haven't broken in before. I was trying to be

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efficient so that we could get Dr. Wissa out of there as quickly as possible. If you want to delay this, we have will have phone conversations. We'll go back. We'll cover the same ground. And we'll re-ask the question.

MR. TRAVIESO-DIAZ: I'm sorry, Denise. Rule number one in depositions is only one lawyer is allowed to ask questions of a witness at a point in time. If you want to ask questions later, after Diane finishes, then we can talk about it. But no double-teaming, please.

MS. CHANCELLOR: Okay. That's fine. I was trying to be efficient.

MR. TURK: Denise, I personally don't blame you. I think this is very exciting. And I understand the impulse to break in.

BY MS. CURRAN:

Q: Dr. Wissa, have you had any conversations with Paul Trudeau of Stone &

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Webster regarding the PFS design issues?

A: Other than with attorneys present? Other than that?

Q: Yes.

A: Yes. I've had one.

Q: Can you describe it for me, please?

A: Paul Trudeau delivered some documents to me at — some plans or — without the attorneys present, just

STATE OF UTAH'S PREFACE TO TESTIMONY OF STEVEN F. BARTLETT AND FARHANG OSTADAN ON CONTENTION UTAH L/QQ - Dynamic Analysis

I. **PFS's design has no margin for error.**
   A. Unprecedented: One of a kind design with untested concepts inherent in its design.
   B. Unique features: 4,000 unanchored casks, shallowly embedded foundations, soil cement to provide resistance to sliding.
   C. Unproven: "controlled" and in-phase cask sliding, untested cement treated soil under pads to strengthen soil; soil cement around pads/CTB to provide resistance to sliding.
   D. Conflicting requirements: the storage pads need to act rigidly for controlled cask sliding but not be too rigidly for cask tipover
   E. Strong ground motions at PFS site: For DSHA pga 1.1.5 g (horiz); 1.17 g (vertical). For PSHA (2000 years return event) pga 0.711g (horiz); 0.695 g (vertical).
   F. Ground motions could potentially be >1 g at some resonance frequencies.

II. **Soils at PFS site are compresibity, deformable and of relatively low strength**
   A. PFS has introduced soil cement and cement treated soil as an "engineering mechanism" in a attempt to improve poor soil conditions.
   B. Foundations overlying compressible soils will settle with time - may crack soil cement which will affect their abiltiy to resist dynamic sliding
   C. Dynamic response of structure is affected by the response and deformation of soil and interaction of the foundations and supported structures - *ie.*, soil structure interaction (SSI)
      1. SSI components: kinematic interaction and inertial interaction. PFS SSI analysis deficient
      2. Need to characterize soils to determine their strength/deformation at levels of strian from DBE – done with strain-controlled cyclic tests. PFS has not done these tests.

III. **PFS's calculations do not demonstarte it has an adequate margin of safety in its design**
   A. HI 2012640 (cask performance) assumes pads will behave relatively rigidly - no deformation, casks slide smoothly.
      1. Cask tipover analysis requires pads to ≤3 ft. thick, Young's modulus ≤75,000 psi; cement-treated soil is expected to exceed 75,000 psi Young's modulus.
      2. Conflicting use of static vs. dynamic Young's modulus of cement treated soil: Holtec used 75,000 psi, which Stone & Webster identify as a static modulus, yet Geomatrix calculation of soil properties and desing moiton used a much higher modulus value.
   B. The adequacy of foundation design is a function of the dynamic forces that will be imparted to them. Critical shortcomings in Holtec's analysis: Seismic loads and assumptions made in calculation those loads - sensitive to input parameters (see Altran report); sesimic loads underestimated, appropriate soil springs and damping not selected.
   C. ICEC obtained the main input parameter, dynamic forces acting on the pad, from Holtec. Dynamic loads given to ICEC do not represent total dynamic load.
   D. ICEC calculation shows the pad behaves flexibly under seismic loads
      a. If the pads behaves flexibly, less radiation damping – underestimation of seismic loads
   E. PFS's (SWEC) dynamic analysis of pads did not analyze pad-to-pad interaction and incorrectly calculated dynamic forces for stabiltiy.
   F. SWEC assmes adhesion of cement treated soil to natives soils and soil cement around pads moving in unision with the pads provides resistance to sliding of a longitudinal column of pads. Not realistic: pads are not locked together and the whole quadrant will not move together.
      a. Separation/gaping of soil-cement from pads during earthquake cycling forces.

b. Gaping most likely to occur along preexisting shrinkage or settlement cracks or from tensile cracks resulting from bending and torsional forces from DBE.
c. Separation and lack of tensile stength will introduce out-of-phase motion; additional dynamic earthquake cyclic forces will act alternatively on pads and soil cement
d. Strain incompatibility/stress concentration in soil cement but if soil cement does not fail in compression it may act as a strut and transfer intertial forces from pad to pad.
e. Wave energy created from simultaneously vibrating pads - this type of pad-to-pad interaction creates addition source of energy at the natural frequency of the pads.
f. In computing dynamic forces, SWEC used pga in its structural analysis of the pads - pga has nothing to do with cask/pad response; contradicted by SWEC analysis of CTB where seismic load were obtained from a dynamic response analysis of the CTB.
g. Correct acceleration should be obtained from Holtec report; Sandia Lab Report for NRC shows pad response accelerations several time larger than pga.
h. SWEC significantly underestimated the seismic load on the pads
i. Pad analysis does not meet 1.1 factor of safety - Newmark sliding block analysis, a deformation analyses, cannot be used to demonstrate adequacy of pad foundations.
j. In any event, SWEC simplified Newmark block analysis is invalid

G. CTB stability analysis suffers some of the same impedimens as the pad analysis.
1. Separation and cracking of soil cement buttress by out-of-phase motion of CTB foundation mat and soil cement - buttress will be ineffective during seismic event
2. Stiff soil cement perimeter around CTB impacts soil spring and damping parameters and kinematic motion of mat foundation - reduce factor of safety to < 1.1.
3. No valid determination that foundation mat is rigid - if the mat is not rigid, soil damping used in dynamic analysis is excessive and CTB seismic loads are underestimated.

H. PFS has not considered cold bonding, potential variations in the motion of the pad and the casks, and the sensitivity of Holtec's nonlinear analysis to input motion.

I. PFS seismic analysis has not complied with ASCE 4-98 – PFS has not considered nonvertically propogating waves, accidental torsion and multiple set of time histories.

IV. Conclusion: Slight margin for error in PFS's design; PFS has used erronerous assumptions and PFS has not demonstrated that unique features of its design will perform safety.

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	)	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel	)	
Storage Installation)	)	April 1, 2002

STATE OF UTAH TESTIMONY OF DR. STEVEN F. BARTLETT AND DR. FARHANG OSTADAN ON UNIFIED CONTENTION UTAH L/QQ (Dynamic Analyses)

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

A. 1: (SFB) My name is Dr. Steven F. Bartlett. I am an Assistant Professor in the Civil and Environmental Engineering Department of the University of Utah, where I teach undergraduate and graduate courses in geotechnical engineering and conduct research. I hold a B.S. degree in Geology from Brigham Young University and a Ph.D. in Civil Engineering from Brigham Young University. I am a licensed professional engineer in the State of Utah.

Prior to this University of Utah faculty position, I worked for the Utah Department of Transportation ("UDOT") as a research project manager and have held a number of other positions with UDOT and other employers where I have applied my expertise in geotechnical engineering, earthquake engineering, geoenvironmental engineering, geotechnical design, applied statistics, and project management.

My position as principal geotechnical investigator for DOE contractor, Westinghouse, on a multi-disciplinary team overseeing the seismic qualification high-level radioactive waste storage tank farm at the DOE Savannah River Site is relevant to this testimony. In that position I reviewed the Safety Analysis Report for the facility and used NRC regulatory guidance documents for my review of that and other projects at the Savannah River site.

My curriculum vitae is included as State's Exhibit 92 and is filed concurrently with my Soils Testimony.

Q. 2: Do you consider it necessary to present testimony with another witness?

A. 2: (SFB) Yes. My testimony is interlinked with the testimony of Dr. Farhang Ostadan. I have worked closely with Dr. Ostadan in our review and analysis of PFS's presentation of the seismic design for its facility. It would be expedient and advantageous for the Board to hear our testimony together.

Q. 3: Please state your name, affiliation, and qualifications.

A. 3: (FO): My name is Dr. Farhang Ostadan. I hold a Ph.D. in civil engineering from the University of California at Berkeley. I am a consultant in the field of soil dynamics and geotechnical earthquake engineering. I am also a visiting lecturer at the University of California at Berkeley and teach a graduate course on soil dynamics and soil-structure interaction. My curriculum vitae is included as State's Exhibit 110.

I have more than 20 years' experience in dynamic analysis and seismic safety evaluation of above and underground structures and subsurface materials. I co-developed and implemented SASSI, a computer program for seismic soil-structure interaction analysis currently in use by the industry worldwide. I am also the technical sponsor of this program in collaboration with the University of California at Berkeley.

I have participated in seismic studies and review of numerous nuclear structures, among them Diablo Canyon Nuclear Station; the NRC/EPRI large scale seismic experiment in Lotung, Taiwan; the large underground circular tunnel for Super Magnetic Energy Storage; General Electric ABWR and SBWR standard nuclear plants; Westinghouse AP600 standard nuclear plant; Tennessee Valley Authority nuclear structures (Browns Ferry, Sequoyah, Watts Bar); and the ITP, RTF, and K-facilities in the Savannah River Site for the Department of Energy. I have published numerous papers in the area of soil structure interaction and seismic design for nuclear and other structures.

Q. 4: Dr. Ostadan, do you consider it necessary to present testimony with Dr. Bartlett?

A. 4: (FO) Yes. Dr. Bartlett and I have worked closely together in our analyses and review of PFS's design concept, the dynamic loading and the effects the loading will have on the casks, pad and building foundations, and soils. To present this testimony separately would create a very disjointed and confusing record.

Q. 5: What is the purpose of your testimony?

A. 5: (SFB, FO)[1] The purpose of our joint testimony is to explain the basis of our individual professional opinions that PFS's design is unique, unprecedented and unproved and that if these unique features fail, PFS has no backup system upon which it can rely. Another purpose of our testimony is to describe PFS's major seismic calculations, show how they have not been integrated with each other and discuss the significant concerns we have with those calculations as well as other concerns we have with PFS's dynamic seismic analysis.

Q. 6: What has been your role in assisting the State in the PFS proceeding?

A. 6: (SB/FO) Commencing in 1999, we have both given technical assistance to the State in the review of PFS's dynamic analysis of the proposed ISFSI site. We provided technical analysis in support of contention Utah QQ and in responding to PFS's Motions for Summary Disposition of Utah L (the original geotechnical contention) and Utah L, Part B (seismic exemption). During the course of our work, we have reviewed the sections of the Applicant's Safety Analysis Report, and updates thereof, relating to its geotechnical investigation and analysis of the proposed site, and relevant calculations, reports, and other documents prepared by the Applicant or its contractors and submitted to the NRC or produced to the State in discovery. We reviewed documents produced by the Staff (*e.g.*, the Safety Evaluation Report ("SER")) and are familiar with and have applied NRC regulations and guidance documents as they relate to geotechnical review.

Q. 7: Please describe the structure of your testimony.

A. 7: (FO/SB) First, we will provide an overview of PFS's design concept and how the unique features in the PFS facility design and that of the cask manufacturer, Holtec International, are unproved and unprecedented. Second, we discuss how those unique features influence our review and note some general but significant failings in PFS's seismic analysis. Next, we will discuss the shortcomings in five major calculations that relate to the seismic analysis and the dynamic loading for the casks, the storage pads, Canister Transfer Building ("CTB") and the pad and CTB foundation soils. Finally, we turn our attention to other concerns we have with PFS's seismic analysis.

Parenthetically, we would add that the State is filing concurrently separate testimony by Dr. Ostadan and Dr. Mohsin R. Khan relating to the specifics of Holtec's cask stability analysis.

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[1] Unless designated otherwise, the initials at the beginning of an answer will designate whether one or both witnesses are responding to the entire question.

Q. 8: Please give an overview of PFS's computation of ground motions at the PFS site.

A. 8: (FO/SB) As part of its original license application, PFS estimated ground motion at the site had a peak ground acceleration of 0.72g in the horizontal direction and 0.80g in the vertical direction using a deterministic seismic hazard analysis ("DSHA"). PFS later revised and significantly increased the strong ground motion estimates for the DSHA, which now have peak ground acceleration (pga) values of 1.15g in the horizontal direction and 1.17 g in the vertical direction. For the 2,000 year return period event, PFS first estimated pga values of 0.53g (horizontal) and 0.52g (vertical), using a probabilistic seismic hazard analysis. But, after a further seismic investigation, pga values were significantly increased to 0.711 g (horizontal) and 0.695 g (vertical), which are the latest peak ground accelerations for the design. The proposed 2000-year return period strong ground motion is obviously less conservative than the DSHA motion.

Q. 9: Please give an overview of the PFS design.

A. 9: (FO/SB) PFS's design contains many unique features. One unique feature of the PFS design is that there will be thousands of unanchored casks sitting in groups of 2 x 4 casks on concrete pads that are 30 feet wide, 67 feet long and three feet thick. SAR Fig. 1.2-1 (Rev. 21). There will be up to 500 pads in the pad emplacement area and the pads will be surrounded by an approximate 2-foot layer of soil cement and underlain by a 1 to 2-foot thick layer of cement treated soil. The soil cement and cement treated soil consist of Portland cement mixed with the surficial eolian silt layer. The cement treated soil will have less Portland cement added to it than the soil cement. The amount of cement and the properties of the treated soil are still undetermined because PFS plans to delay the requisite testing to complete the design until after it obtains a Part 72 license from the NRC. It is surprising to us that the Staff has found this to be acceptable, because the use of soil-cement and cement treated soil has become an integral part of the seismic design of the pad foundations and the proposed application at the PFS site is unprecedented and unproven. As we subsequently discuss, we have several issues with the seismic design and use of soil cement and cement treated soil that PFS expects to provide seismic stability at the PFS site.

Another unique design feature at the proposed PFS facility is the use of soil-cement around the Canister Transfer Building to provide resistance to earthquake sliding. The CTB is the building in which the canister containing spent fuel rods will be transferred from a transportation cask (HI-STAR) to a storage cask (HI-STORM). The concrete building is founded on a 5-foot thick reinforced concrete mat with plan dimensions that are approximately five feet thick, 240 feet wide and 280 feet long. SAR at 4.7.-3 (Rev. 22). Soil-cement will extend out from the foundation mat approximately 240 feet in both the east and west directions and 280 feet in both the north and south directions. SAR at 2.6-108b (Rev 22). PFS intends to use the perimeter soil cement as a buttress to improve the sliding resistance of the CTB. Without the soil cement buttress, PFS's calculations show that the factor of safety of 1.1 against sliding cannot be met. Thus, the use and performance of the soil cement is crucial to the seismic design. However, the use of soil cement to provide sliding resistance to shallowly embedded, heavily loaded foundations subjected to intense strong ground motion is unprecedented and unproven.

Yet another unique feature is the "controlled sliding" design concept for the Holtec storage casks. Holtec puts forward the proposition that during strong ground motions, the casks will be allowed to slide and such sliding will occur in a uniform and controlled manner without collision or tipping. Such a concept defies observations from major earthquakes and engineering logic. It is unprecedented to design unanchored dry storage casks for a seismically active area with such intense strong ground motions similar to those at the PFS facility. The unconservatism in the design is further compounded when PFS uses its claim of "controlled" cask sliding as a mechanism to reduce the seismic loading to the pad foundations.

There are also conflicting requirements in PFS's design. Holtec's cask tip over analysis is bounded by requirements that the concrete pad and underlying cement treated soil not be too stiff. (Stiffness in this case has been measured by the modulus of elasticity, commonly called Young's modulus.) If this system is too stiff, then it will be unable to meet maximum deceleration requirements put forth by Holtec in the cask drop/ tipover analyses. Thus, the cement treated soil mix under the pads cannot be too stiff, otherwise it will exceed Holtec's bounding condition. However, there are counter-balancing requirements that the cement treated soil be strong enough, so as to resist cracking or damage caused by earthquake strong ground motion or other loading and environmental factors. PFS has not demonstrated that these counter-balancing requirements can be met for the pad design.

Q. 10: Does the design of the PFS facility factor into your review of PFS's seismic analyses, and if so, how?

A. 10: (FO/SB) Yes. First, PFS has presented a one-of-a kind design. We know of no similar design that uses untested concepts that are inherent in this design. What this amounts to is a very unconservative design - 4,000 unanchored casks, each 20 feet high, 11 feet in diameter and weighing about 175 tons,[2] resting on a shallow pad foundation, and buttressed by an unproved soil-cement structural design element over clayey soils which are known to lose strength due to any disturbance caused by construction. The site is subject to strong ground motions, with the peak ground acceleration estimated to be approximately 0.7g. As Dr. Marvin Resnikoff explains in his concurrently filed testimony, the bounding ground motions in the Certificate of Compliance for the HI-STORM cask for the purpose of determining the maximum zero point acceleration that will not cause incipient tipping are

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[2] *See* PFS SAR, Table 4.2-2, Rev. 12; State's Exh. 142 to Dr. Resnikoff's Testimony (Seismic Exemption - Dose Exposure).

bounded by accelerations of 0.445 g (horizontal) and 0.16 g (vertical). The seismic analysis PFS has presented – and the Staff has endorsed in the SER – without taking into account any of the shortcoming we raise here, contains essentially no margin for error.

Second, there is no redundancy in PFS's design. It is usual in seismic design to have redundancy so that if a particular component fails, there is a backup system or mechanism to ensure satisfactory performance. But this is not the case here. There is no redundancy in PFS's design For PFS's seismic design to perform adequately during strong ground motions, the Holtec cask must slide in a controlled manner and in-phase; the pads must act rigidly to allow such controlled sliding; but they must not be too rigid in the event of cask tipover. Also, the cement treated soil under the pads must not be too rigid in the event of cask tipover, but it must have adequate strength and bonding to adhere to the pad foundation and the native soils to create shear resistance to foundation sliding. Further, the soil-cement around the pads must lock the longitudinal rows of pads together as an integral mat and the large areal mass of soil-cement around the CTB must act as a buttress to prevent foundation sliding.

As the above illustrates, the lack of safety elements in PFS's design means that deviations or missteps in estimating the material properties and dynamic response of the casks, foundation structures, soil-cement, cement treated soil and native soils at the proposed facility may be sufficient to create unintended consequences or to result in design failure. This point should be kept in mind when we raise specific challenges to the way in which PFS has conducted its seismic analysis.

Q. 11: As a general matter, how do you analyze the effect of earthquake ground motions on foundations and soils.

A. 11: (FO/SB) When analyzing the ability of a foundation system to resist earthquake ground motions, there are two aspects to consider: "capacity" and "demand." Capacity is the ability of the soil and foundation to resist the demand. The capacity of the soil to provide dynamic stability has been discussed by Dr. Bartlett in his testimony regarding soils and soil cement. The demand is the dynamic and static forces applied to the foundation and soils by the weight of the structures and the earthquake inertial forces. The DBE ground motion of about 0.7 pga is considered a large demand and places considerable inertial loadings on the unanchored casks and on the foundations of the storage pads and Canister Transfer Building.

Q. 12: Based on your experience, what are the notable elements of seismic setting for design of the PFS facility?

A. 12: (FO/SB) The PFS site has close proximity to major seismically active faults, with one newly discovered fault essentially dipping under the site. There is a high potential for the facility to experience very high levels of strong ground shaking with accelerations exceeding 1 g at some frequencies of response.

Q. 13: Previously you stated that the DBE ground motion has a peak ground acceleration of about 0.7 g, but you state that there is a possibility that the structures and foundations may see accelerations in excess of 1 g at some frequencies. Could you please explain this?

A. 13: (SB) All structures and foundations can resonate during earthquake shaking. Earthquake waves arrive with differing wavelengths (or frequencies) and amplitudes. Resonance of a structure is possible when the frequency of vibration, or harmonics, of the structure approximately matches the frequency of the predominate earthquake waves. This resonance can cause accelerations in the structure that well exceed peak ground acceleration. Thus, in seismic design it is important to consider the natural frequency of vibration or period of the structure, because the frequency of resonance controls that amplitude of ground motion experienced by the structure.

Q. 14: Is resonance an important consideration in the PFS design?

A. 14: (FO/SB) Yes, it is important to the PFS design. All seismic design must consider the potential for resonance. The magnitude of resonance is particularly important in the design of the pads, because resonance in the vertical direction of the pads can cause the transfer of large inertial forces to the casks. As we discuss later, the pads have a natural frequency of vibration in the vertical direction and will have a resonance effect. Sufficiently large resonance can potentially cause uplift of the cask, so that there may be brief moments during the earthquake when the base of the casks are uplifting or not uniformly contacting the top of the pads. Such uplift can have very deleterious effects and can greatly increase the sliding motion and potential tipping of the casks.

Q. 15: Do you consider any aspect of PFS's design concept to be unprecedented?

A. 15: (FO/SB) What is unprecedented in PFS's design concept is designing structures with foundations effectively placed on top of the soil and expecting such structures to remain stable under the high level seismic loads. For casks not tied down to the pads, the design concept that the casks will be able to slide freely on the pads when subject to design ground motions is also unprecedented.

Q. 16: Are you aware of any heavily-loaded, shallowly-embedded critical structures that have been placed over clayey soils in an area with high level of seismic motions?

A. 16: (FO) I do not know of any.

Q. 17: Based on your past experience, what general elements of the overall design are of concern to you?

A. 17:. (FO) My concern originates from observation of earthquake damage to structures from many earthquakes in the past. I have seen many pictures of heavy objects such as rocks, trains, tanks moved around, toppled and some thrown in the air when subjected to ground motion, some even subjected to ground motions less than the design motion at PFS site. State's Exhibit 111, earthquake pictures and explanations.

To start with, I have wondered how unanchored casks simply resting on the pads without any structural connection could remain stable during the design motion. I also noted that PFS has not built any redundancy into the design. What if the parameters used for design were not accurate and the system has not accounted for variability? What if key mechanisms are not properly accounted for in the design? What if the ground motion experienced is higher than the proposed DBE motion? This is extremely important because the design presumes controlled sliding, which is generally considered an uncontrolled condition. What if the sliding is larger than anticipated? Will the casks collide, or slide off the pad areas, or tip over? Also, will the CTB slide?

All seismic designs have an engineered redundancy built into their system. The redundancy exists as a backup system and should be robust. For example, if a column in a building fails, it does not necessarily cause collapse of the building because there are other columns or mechanisms to take the redistributed load. Or, if a pier of a bridge fails, the bridge will be standing due to the support of the other piers. Introducing redundancy is a common and necessary feature in seismic design. The less redundancy in the design, the more demand is placed on the system and the more chance for uncontrolled or unintended consequences.

Q. 18: How would you characterize the analysis and design for the stability of the casks in the PFS facility?

A. 18: (FO) The analysis of the casks sliding on the pads is based on a nonlinear time history analysis. Such analyses are very complex and are not common for critical facilities. Such complex analyses are very sensitive to input parameters and the results can change significantly if the input parameters change. Therefore, it is prudent to confirm the results of analysis by performing experiments on models or prototypes to ensure the adequacy of the design. At a minimum, a wide range of expected input parameters should be used in such a complex and sensitive analysis, especially for such parameters that are not commonly used and little is known about their behavior under seismic loading.

Q. 19: What kind of experiment would you consider appropriate for design of the casks on the pad?

A. 19: (FO) Shaking table tests. A model of the system can be built on the shaking table and the table can be excited with the design motion and the response can be recorded and evaluated.

Q. 20: Has PFS performed such tests?

A. 20: (FO) Not to my knowledge.

Q. 21: Are soil properties at the PFS site important in you review of PFS's seismic analysis?

A. 21: (SB) Yes. Soil properties are important for all five calculations that we are discuss in this testimony. The dynamic properties of the soil become especially important when seismic loads are high as they are at the PFS site.

Q. 22: What aspect of the soil condition and properties at the PFS site becomes important when seismic loads are high?

A. 22: (SB) In general, more competent soils have a higher capacity to carry seismic loads without failure or excessive deformation. At the PFS site, the soil layers directly under the foundation are of silty and clayey soils, which are generally considered less desirable soils due to their compressibility, deformability and relatively low strength. PFS has recently noted the weakness of the supporting soil and has introduced soil cement and cement treated soil as an "engineering mechanism" in an attempt to improve generally poor soil conditions.

Q. 23: Why are compressibility and deformability of the soil important? Isn't only shear strength required to resist seismic loading?

A. 23: (SB) The compressibility of the soil is important at the PFS site because foundations overlying compressible soils will settle with time. Such settlement, if large enough, may crack the soil cement and cement treated soil around the pads and CTB and affect their ability to resist dynamic sliding. The Applicant has estimated about 2 inches of total settlement of the pads (SAR, p. 2.6-50) and 3 inches of total settlement for the CTB. This condition invalidates the assumption of an integrated foundation for ten rows of pads and also potentially negates the validity of the passive pressure used in the stability analysis of the individual pad and the CTB.

Even more important is the consideration of soil deformation. Soil by its nature is a deformable body, which will strain or deform during the earthquake. Deformation of the soil can have many consequences to the dynamic response and interaction with the foundation and supported structures. The dynamic response of the structure is affected by the response and deformation of the soil and the interaction of the masses of the foundations and supported structures. This type of interaction is called soil-structure interaction and is a very important consideration for this site. Soil-structure interaction has two components: 1) kinematic interaction that results from differences in stiffness of the foundations and the soils, and 2) inertial interaction that results because the foundations and their supported structures have different masses than the supporting soils.

In characterizing the soils for such analyses, it is important to determine their strength and deformation characteristics at the levels of strain introduced in the foundation soils by the design basis earthquake. These determinations are generally done with strain-controlled cyclic tests, so that potential degradation of shear strength and modulus is accounted for during cyclic strains. Significantly, PFS has not done these types of tests for the native soils, soil cement and cement treated soils for this site. PFS has not indicated any intention to perform such testing (pre or post-licensing). *See* my soils testimony at A.30.

Q. 24: **There have been numerous calculation packages presented as part of PFS's seismic analysis. Please describe the major calculations, whether and how they interrelate, and the concerns you have with these calculations.**

A. 24: **(FO/SB)** Apart from the calculations by Geomatrix Consultants to develop the design motion and the dynamic soil properties, there are five sets of PFS calculations that we will describe here: two have been performed by Holtec, one by International Civil Engineering Consultants ("ICEC"), and two by Stone and Webster ("SWEC").

First we will comment upon two calculations conducted by Holtec: one was for the cask tipover, HI-2012653, Rev. 2 (10/31/01), *PFSF Site Specific HI-STORM Drop/Tipover Analyses*, and the other relates to the performance of the casks, HI-2012640 (Rev. 2), Rev.1 (8/20/01), *Multi Cask Response at the PFS ISFSI from 2000-Yr Seismic Event*. The Holtec Report, HI-2012653, was done to determine whether the maximum deceleration of the storage cask at the top of the active fuel region could achieve the design basis value of 45g's in a non-mechanistic tipover event. SAR at 4.2-9 and 10 (Rev. 22). The purpose of other report, HI-2012640, is to show that the casks sliding on the pads have limited displacement, would not impact each other and would not tip over due to seismic excitation.

The conditions Holtec assumed in its cask tipover analysis, HI-2012653, is that the concrete pad is no more than three foot thick and that the modulus of elasticity of the cement-treated soil does not exceed 75,000 psi. One critical issue in this calculation is whether the modulus of elasticity assumed by Holtec is dynamic or static. Another is that the cement-treated soil mix under the pads cannot be too stiff – *i.e.*, it should not have a modulus of elasticity that exceeds 75,000 psi. Yet, the dynamic stability analysis of the pads requires that the cement treated soil improve the shear strength of the native soil to a sufficient degree in order to transfer the seismic loads to the underlying soils. The improvement of the shear strength by adding cement will commensurately increase the soil's stiffness. These contrasting goals of flexibility but improved shear strength may create conflicting design requirements. The achievement of these goals requires a detailed knowledge of the properties of cement treated soil under both static and seismic conditions for the duration of the design life of the facility. This specific knowledge is not available for these design goals, nor is there design precedence or site-specific testing to support the proposed design. PFS has stated its intent to conduct testing some time after it obtains a licence from NRC to determine whether it can achieve these goals, but because this is a critical and novel part of the seismic design, we believe it is critical that soil cement testing be presented prior to licensing. We do not consider that any decision-maker can make an informed or rational judgment on PFS's intended use of soil cement or cement-treated soil without these data.

Q. 25: What is your general impression of Holtec's evaluation?

A. 25: (FO/SB) Based on the increase in ground motions and on PFS's adoption of soil-cement in design, Holtec had to revise its earlier analysis of the cask-pad to include the increase in ground motions and the use of soil cement. We have significant concerns with Holtec's analysis and the assumptions upon which Holtec relied. This is important because evaluating the adequacy of the foundation design is a function of the dynamic forces that will be imparted to them. In order to evaluate the response of the foundation or the soil cement to resist seismic loads and to evaluate the stability of the casks, it is critical to understand the seismic loads and the assumptions made in calculating the seismic loads. The independent analysis by Altran Corporation has confirmed how sensitive such complex analyses are to the input parameters. Altran's study[3] has shown that the results can change significantly when input parameters are changed within the acceptable bounds. This is described in more detail in testimony (cask stability) Dr. Ostadan presents with Dr. Mohsin Khan.

(FO) In the cask performance analysis, HI-2012640, Holtec assumed that the pad would behave relatively rigidly. In this case, Holtec assumed that there would be no deformation within the pad and that the casks will slide smoothly over the surface of the pad. This creates a conflicting philosophy about whether the pads will behave flexibly or rigidly under earthquake conditions. On one hand Holtec's analysis requires a flexible pad of limited stiffness (*i.e.*, rigidity) for cask tipover. On the other hand, Holtec's design assumes that the pads will be sufficiently rigid to allow the casks to slide smoothly.

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[3] Analytical Study of HI-STORM 100 Cask System Under High Seismic Condition, Technical Report No. 01141-TR-000, Revision 0 (Dec. 11, 2001), State's Exhibit 122 to concurrently filed Testimony of Drs. Ostadan and Khan.

Q. 26: Please elaborate on your concerns about treating the pads rigidly.

A. 26: (FO) The misconception that the pads will behave rigidly will overestimate foundation (*i.e.*, radiation) damping. During radiation damping, the foundation vibrates and a signification portion of the seismic energy coming down from the structure to the pads is dissipated into the soil. Uniform motion and dissipation of energy from a rigid pad are vastly different from those of a flexible pad. A flexible pad cannot produce as much radiation damping. In my opinion, the assumption of pad rigidity will incorrectly reduce the dynamic load from the seismic analysis potentially leading to an underestimation of the actual seismic loading.

Q. 27: Are there other concerns you have with Holtec's analysis?

A. 27: (FO) Yes. In the drop/tipover analysis of the casks (*PFSF Site-Specific HI-STORM Drop/Tipover Analyses*, Holtec Report No. HI-2012653, Holtec has assumed a lower stiffness of the soil cement cement-treated soil under the pad and limits the concrete pad thickness to 3 feet to meet the requirement of drop/tipover condition. In doing so, it has failed to recognize the difference between the static and dynamic modulus of the soil cement and the effect of significant temporal and spatial change in bearing pressure acting on the soil cement. The expected large difference between the static and dynamic modulus invalidates the assumption made for the design. The Young's modulus of 75,000 psi for cement treated soil used by Holtec is identified as a static modulus in the Stone & Webster stability analysis of the casks. Yet, in the Geomatrix calculation of the soil properties and design motion, a much higher value of modulus was assigned to the cement treated soil. Even though PFS has not yet performed any test on soil cement cement-treated soil, it is likely that the dynamic modulus of the soil cement will exceeds the value of 75,000 psi and the requirement for the drop/tipover condition can will not be met.

Q. 28: Please describe the ICEC calculation for the structural design of the storage pads?

A. 28: (FO) The ICEC calculation, *Storage Pad Analysis and Design*, Calc. No. 0599602-G(PO17)-2, Rev. 3, 4/5/01 was performed to design the concrete storage pads. This is a fairly complete calculation that develops the seismic loads for the structural design of the pads. Two methods of analysis are used to ensure the results are consistent. The input for this analysis including soil properties, soil springs and damping are obtained from other calculations. A critical shortcoming in the ICEC calculation is that ICEC obtained the dynamic forces acting on the pad from the Holtec Report HI-2012640. Dynamic forces are the main input parameters for design of the pads. As discussed later, we have several concerns with the computation of these forces. Nonetheless, a significant point that can be seen from the ICEC calculation using the forces provided by Holtec is that some vertical displacement or deformation of the pad is occurring. As I discussed, at length, in my deposition testimony, the relative displacement between the different nodal points on the pad in the ICEC calculation show pad deformation. In that same testimony, I discuss the transfer function for the pads which shows the frequency response of the cask-pad-soil system and from this I conclude that the fundamental frequency of vibration of the pads is between approximately 5 to 11 hz. If the pads were behaving rigidly, the natural frequency of vibration would be much greater. *See* Ostadan Tr. at 91-118 and Tr. Exh. 31, included as State's Exhibit 112.[4] Thus, the ICEC results support the proposition that the pads will behave flexibly under seismic loads.

Q. 29: Do you have any major concerns with the ICEC calculation?

A. 29: (FO) The calculation is fairly complete except that the long-term settlement of the pads has not been considered in design of the pads. The long-term settlement of the pads could have been obtained from the Stone and Webster calculation, G(B)-21 *Supplement to Estimated Static Settlement of Cask Storage Pads* (May 21, 2001), and used by ICEC for the pad design. However, the ICEC calculation reveals other useful information about the behavior of the cask-pad system which were not reported in the Holtec report.

Q. 30: From which calculation did ICEC obtain the dynamic forces acting on the pad?

A. 30: (FO) From the Holtec calculation report, HI-2012640 (Rev. 2), Rev.1 (8/20/01), *Multi Cask Response at the PFS ISFSI from 2000-Yr Seismic Event.* This very short report provides the results of the movement of the casks on the pad. It is based on a very complex nonlinear time history analysis and includes the pad, the soil spring and damping and the design motion. It does not provide any results or discussion about the effect of soil-structure interaction on the pad response. There is more information presented in the ICEC calculation about the pad response than in the parent calculation report by Holtec.

Q. 31: What can you see in the ICEC calculation that is not presented in the Holtec Report, HI-2012640?

A. 31: (FO) The ICEC calculation clearly shows that the effects of soil-structure interaction is very important. The fundamental frequency of vibration for the foundation system changes significantly when the soil properties are changed from the lower bound, to best estimate and to the upper bound cases. These frequencies can be checked by simple stiffness over mass calculation, which confirms the frequencies shown from SASSI analysis. Because the effect of soil-structure interaction is important, it is likely that the effect of pad-to-pad interaction will be very important, particularly with respect to the sliding motion

[4] Excerpts from deposition transcript of Dr. Farhang Ostadan (March 8, 2002), and Exh. 31 to deposition, 04/05/01 ICEC (International Civil Engineering Consultants, Inc.), excerpts from calc. no. G(PO17)-2, Rev. 3, *Storage Pad Analysis and Design.*

of the unanchored cask on the pads. Holtec and Stone and Webster in their calculations have not considered the effect of pad-to-pad interaction for pads spaced only 5 feet apart in the longitudinal direction in analysis. There are 500 identical pads vibrating effectively at the same frequency. The resonance caused by such identical systems has not been considered by PFS in any of its analyses.

Since Holtec HI-2012640 has not reported the magnitude of the dynamic loads in their report, some information may be obtained from the ICEC report calculation. The horizontal reaction forces are reported in the ICEC calculation. By dividing the reaction forces by the weight of the casks and the pad, one can clearly observe the effective acceleration experienced by the cask and the pad system. This acceleration is less than 0.60 g. This is for the case where 8 casks are placed on a pad with a coefficient of friction of 0.8. The effective acceleration is less than the peak ground acceleration and is clearly much less than the design motion at the natural frequency of the system. This simple calculation shows that the dynamic loads given to ICEC for the design of the pad are deficient and do not represent the total dynamic load of the cask and the pad.

Q. 32: You mentioned that for part of its calculation, ICEC obtained soil springs and damping from Holtec. Please explain soil springs and damping.

A. 32: (FO) Soil springs and damping are typically considered for dynamic analysis of structures such as the pad and the casks to represent the effect of the supporting soil layers as well as the foundation size in the response. These properties are frequency dependent. If the pads are assumed to be rigid, the damping will be larger. If the pads are indeed flexible the damping will be less. The less the damping, the higher the motion of the pads and the seismic loads on the pads. It is important to use the soil spring and damping values at appropriate frequencies corresponding to the foundation frequencies and check the pad rigidity assumption based on the final design.

Q. 33: What effect, if any, do soil springs and damping have on Holtec's analysis?

A. 33: (FO) To be able to predict the motion of the pad and cask movement, it is important to select the appropriate soil spring and damping values. The Holtec analysis did not properly consider the frequency dependency of these parameters with respect to important frequencies of the vibration. Holtec has provided no check to compare the parameters used by other available rigorous solutions to ensure the foundation parameters are reasonably accurate. Soil springs and damping change significantly with frequency of vibration.

In the calculation of soil spring and damping for the Canister Transfer Building ("CTB") (SC-4, Rev. 2, *Development of Soil Impedance Functions for Canister Transfer Building*, March 21, 2001, SWEC), the soil spring and damping are plotted as a function of frequency, showing that these parameters are highly dependent on the frequency due to soil layering at the site.

It is therefore necessary to ensure the selected spring and damping values represent the appropriate foundation behavior under dynamic loading, particularly as it relates to prediction of cask movement on the pads. This Holtec has failed to do.

Q. 34: Please describe the other two major calculations, the ones performed by Stone & Webster.

A. 34: (FO/SB) The other two calculations deal with the dynamic stability of the storage pads and CTB foundations, respectively, Stone & Webster in (1) *Stability Analyses of Cask Storage Pads*, Cal. No. G(B)-04, Rev. 9 (July 26, 2001), and (2) Calc. No. G(B)-13, *Stability Analyses of Canister Transfer Building*, Rev. 6 (July 26, 2001). These calculations investigate the seismic stability of the storage pad foundations to determine the overturning stability, sliding stability and bearing capacity for static and dynamic loads resulting from the DBE.

The first calculation evaluates the static and seismic stability of the cask storage pad foundation. The potential failure modes investigated include overturning, sliding and bearing capacity for static and dynamic loads due to the design basis earthquake (2000 year return period). The second calculation calculates seismic stability against overturning, sliding and dynamic bearing capacity of the Canister Transfer Building. It also calculates the static bearing capacity of the Canister Transfer Building.

Q. 35: What, if any, are your concerns with the pad sliding stability calculation, G(B)-04, Rev. 9?

A. 35: (FO/SB) In calculation G(B)-04, Rev. 9, PFS describes its intentions to use cement-treated soil as a structural element under the pads and a stiffer soil-cement mix around the perimeter of the pads. There are two overriding concerns with this calculation: pad-to-pad interaction, and calculation of the dynamic forces for pad stability.

Q. 36: Please describe your concerns about pad-to-pad interaction.

A. 36: (FO/SB) Stone & Webster assumes that for a longitudinal column of pads, resistance to sliding is provided by adhesion of the cement treated soil to the native soils beneath the pad and that the soil cement around the pads is moving in unison with the pads. This may be true if each quadrant of pads were locked together by a reinforced concrete mat foundation. But when you have soil cement between the pads, assuming that the whole quadrant moves together under a unified (in-phase) sliding conditions is not realistic. Moreover, we have significant concerns about the separation or gaping of soil-cement from the pads during the cycling of earthquake forces. This gaping will most likely occur along preexisting shrinkage or settlement cracks or will be introduced as tensile cracks in the soil cement resulting from the bending and torsional forces introduced by the design basis earthquake. This separation and lack of tensile strength will not allow the pads and soil cement to act as an integral unit, thereby introducing out-of-phase motion and additional dynamic forces that will act alternately on the pads and on the soil-cement during earthquake cycling. This will create inertial pad-to-pad interaction. Because the pad and the soil cement are not structurally integrated, PFS's assumption of having a group of the pads act as an integral foundation is not correct.

This concern was borne out in deposition testimony given by the PFS witness, Dr. Wen-Shou Tseng[5]:

> Q. So do you believe the soil cement will not have an impact in integrating the motion of the different pads together?
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> A. It stiffens up the soil, certainly, and that effect has been included in this. But structurally you don't have really positive connections. Eventually I don't think they would behave as an integrated structure.

Tseng Tr. at 69-70.

(SB) In deposition testimony, PFS's witness responsible for both the pad and CTB stability calculations, Mr. Paul Trudeau[6], stated that cracking and openings may occur during the seismic event and that the cracking will most likely occur at pre-existing shrinkage cracks in the soil cement. In response to a question whether there will bending and/or torsional stresses in the soil cement around the CTB, Mr. Trudeau offered the following opinion:

> 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

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[5] Excerpts from deposition transcript ("Tr.") of Dr. Wen-Shou Tseng (March 12, 2002) included as State's Exhibit 113.

[6] Excerpts from deposition transcript of Mr. Paul Trudeau (March 6, 2002) included as State's Exhibit 114.

> 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.

Trudeau Tr. at 40-41. State's Exh. 114.

I disagree with Mr. Trudeau's opinion that the cracking will be vertical and passive resistance is not diminished. This has not been demonstrated by the Applicant. However, the more important point here is that such cracking will allow out-of-phase motion between the soil cement and the adjacent structure. Once cracked, the soil cement can no longer provide resistance to tensile forces and can no longer prevent out-of-phase motion. This interaction must exist due to the different masses involved which will produce differences in the frequencies of vibration in the horizontal and vertical directions. In the case of the pads, pad-to-pad interaction appears to be particularly acute in the longitudinal direction of the pads due to their close (*i.e.*, 5-foot) spacing.

Further, sliding failure of the pads is not a requisite condition to produce pad-to-pad interactions. Significant gaping and pounding (*i.e.*, inertial interaction) can occur without initiating sliding failure. This is because the pads and soil cement are resting atop a deformable soil body (*e.g.*, Bonneville clay and deeper cohesionless Bonneville soils). The consequences of these interactions can be considerable. For example, the Young's modulus of soil cement between the pads is about 30 times greater than the Young's modulus of the Bonneville clay. This creates strain incompatibility and stress concentration in the soil cement between the pads as the gap attempts to close, due to the higher modulus of the soil cement. If the soil cement does not fail in compression, it may act as a "strut," thus introducing significant pad-to-pad interaction and transfer of inertial forces from pad to pad. Thus, the presence of competent soil cement between the pads may actually be detrimental to the design, when one considers the potential for cracking, separation and pad-to-pad interaction.

(FO/SB) The primary concern with pad-to-pad interactions pertains to the potential transfer of cask and pad inertial loads from one set of pads and casks to adjacent pads and casks. The transfer mechanism occurs via the relatively stiff soil cement that is placed between the pads. The transferred inertial load will act as a driving force on the adjacent pad, which is only 5 feet away. The consequences of this transfer have been completely neglected in the sliding and stability calculations of the casks and the pads.

(FO) In addition to the transfer of inertial forces resulting from pad-to-pad interaction, there will be another consequence of pad-to-pad interaction. When there are two or more nearby bodies that are simultaneously vibrating, this creates a condition where additional wave energy is created from the interaction. For example, if the cask-pad-foundation system is vibrating at a natural frequency of about 8 hertz and hundreds of nearby pads are doing the same, there will be significant amplification of the motion. This type of pad-to-pad interaction creates an additional source of energy at the natural frequency of the pads. This is a well known fact based on my experience when working on nuclear projects that have adjacent or nearby structures.

Q. 37: What are your concerns about PFS's calculation of the dynamic forces acting on the pad?

A. 37: (FO) The second overriding concern with G(B)-04, Rev 9, is the method that Stone & Webster has used to calculate the dynamic forces for pad stability. In calculation G(B)-04, Rev. 9, Stone & Webster has used peak ground acceleration in its structural analysis of the pads. In this calculation, the mass of the casks and concrete pads was calculated and the inertial force was incorrectly calculated by multiplying the these masses times peak ground acceleration. Peak ground acceleration has nothing to do with the cask and pad response, nor does it consider resonance. This approach is contrary to that used in the stability analysis of the CTB where the seismic loads were obtained from a dynamic response analysis of the building.

In calculation G(B)-04, Rev 9, the seismic loads for the pad and the casks are erroneously estimated using the accelerations from the design motion and not from the acceleration response of the pad and the casks. The acceleration of the pad and the casks in the vertical direction is chosen to be 0.659g which is the acceleration of the design motion and not the response of the pad or the cask. Similarly, the horizontal acceleration of the pad is chosen to be 0.711g. This is not correct because the accelerations used do not represent the response of the casks and the pads and are likely to be less than the actual response accelerations.

The correct acceleration values should have been obtained from the Holtec calculation Report HI-2012640. This point can be made clear by looking at the recent report prepared for the NRC staff by Sandia National Laboratory (March, 2002).[7] *See* State's Exhibit 115, Figures 16, 17, 20, 21 and 22 from the Sandia Report. This is an independent report but similar to the Holtec report for dynamic analysis of the pads and the casks. The inputted soil design motions used are the same as those used in the Holtec report. The Sandia analysis clearly shows that the pad response accelerations are several times larger then the peak ground acceleration used by Stone and Webster in its stability analysis. Therefore, the seismic loads used in the stability analysis by Stone and Webster are not correct and significantly underestimate the seismic loads actually occurring on the pads.

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[7] *Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage (PFS) Facility*, (March 8, 2002), Luk, Vincent K., et al, Sandia National Laboratory, excerpts included as State's Exh. 115.

Q. 38: What additional concerns do you have regarding the stability calculations of the pads?

A. 38: Another concern is PFS' s failure to meet a factor of safety against sliding of 1.1. In Calc. G(B) 04, Rev 9, PFS states that the cask storage pads have an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion because the factor of safety "is greater than the criterion of 1.1." Calc. G(B) 04, Rev 9 at 14[8]. The factor of safety, FS, equals the resisting force divided by the driving force. Id. at 15. *See* State's Exh. 116.

The simplified Newmark sliding block analysis presented in the revised calculation G(B) 04, Rev. 9 does not meet the 1.1 factor of safety against siding. In the revised calculation G(B) 04-9, pp. 46 - 51, the Applicant has included a Newmark sliding block analysis in an attempt to show "acceptable" deformation for the case of sliding on a deeper, sandy/silty layer. This layer is approximately 8 to10 feet deep and has a factor of safety against sliding of less than 1. *See* State's Exh.116, p. 46.

From this case the Applicant states:

> 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, should they occur, they will not adversely impact the performance of the pads.

Calc. G(B) 04, Rev 9 at 46.

The Applicant further states, "[t]he 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." Id.

I disagree with the approach that deformation analyses can be used to demonstrate the adequacy of the foundations for the storage pads. NUREG-0800, Section 3.8.5, "Foundation," Section II.5, "Structural Acceptance Criteria" requires a minimum factor of safety against sliding and overturning of 1.1 for earthquake events." State's Exh. 93 included with my soil characterization testimony.

Also, the simplified Newmark (1965) sliding block analysis presented in the revised calculation G(B) 04, Rev. 9 has errors and unconservative assumptions which invalidate the

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[8] Excerpts from Calc. G(B) 04, Rev 9, included as State's Exhibit 116.

conclusions obtained from the analysis.

First, the vertical earthquake forces are incorrectly calculated. The Applicant has used the peak vertical ground acceleration when calculating N (maximum resistant coefficient). Use of vertical pga assumes rigid behavior. This is an incorrect assumption as discussed previously.

Second, the Applicant has not considered unsymmetrical sliding and potential for pad-to-pad interaction. Newmark (1965) gave solutions for unsymmetrical sliding in the case when the motion takes place with different resistance to sliding in one direction. Unsymmetrical sliding may take place at pads located at the end of the columns or rows and also where there is pad-to-pad interaction. PFS did not consider these cases in its simplified sliding analysis. Newmark (1965) charts show much larger displacements for the case of unsymmetrical sliding.

Third, the charts presented in Newmark (1965) have been normalized to pga = 0.5. The design basis earthquake peak ground acceleration is about 0.7 g. The Applicant has not explained the applicability of these normalized charts to the larger design basis ground motion expected at the PFS site.

Fourth, Newmark (1965) charts are based on very limited earthquake data. The charts were developed from only 4 western U.S. earthquakes. The Applicant has not compared the amplitude, frequency, phasing and velocity pluses in these records to that used for the design basis ground motion at the PFS site. These charts may not be robust enough for design until these uncertainties are understood and the applicability of these charts to the design basis ground motion.

Q. 39: Please describe the Stability Analysis of the CTB.

A. 39: (FO) This analysis is contained in Stone & Webster calculation, *Stability Analyses of Canister Transfer Building*, Cal. No. G(B)-13, Rev. 6 (7/26/01). The purpose of Calc. G(B)-13 is to determine the stability against overturning, sliding, and bearing capacity failure of the CTB. In its analysis, PFS claims that the factor of safety against sliding is greater than 1.1. Cal G(B) 13, Rev 6 at 23.[9] This calculation suffers from some of the same issues raised in our review of Calc. No. G(B)-4 regarding the use of soil cement to improve the sliding capacity of the CTB mat foundation.

[9] Excerpts from Calc. G9B)-13, Rev. 6 included as State's Exhibit 117.

Q. 40: What are the main concerns with PFS's analysis of the dynamic stability of the CTB?

A. 40: (SB) One concern is that PFS has not supported the presumed passive resistance provided by the soil cement with the requisite engineering calculations and testing. PFS has failed to demonstrate that the proposed soil cement buttress will not simply crack and be rendered ineffective during a seismic event. For the case of the CTB, PFS has not considered the deleterious effects of separation and cracking of the cement-treated soil buttress caused by out-of-phase motion of the CTB mat foundation and the cement-treated soil buttress. PFS has not calculated the bending and tensile stresses that will develop in the soil cement and how these stresses will affect the ability of the cement-treated soil buttress to resist these forces without cracking or separation.

(FO) Another concern is that PFS has failed to analyze the dynamic interaction of the soil cement with the CTB mat foundation. In the case of the CTB foundation, the influence of the large soil cement mass around the building has been ignored.[10] Also, the presence of a stiff, soil cement perimeter around the CTB of about one building dimension impacts the soil spring and damping parameters and kinematic motion of the mat foundation. Therefore, such shortcomings in the calculation can easily reduce the 1.1 factor of safety against sliding to values of less than 1, indicating instability of the CTB for sliding.

(FO) Finally, based on my past experience dealing with analysis and design of large mats, such as the CTB mat foundation, in my opinion, the concern on the assumption that the mat is rigid has not been addressed. This is particularly important due to the slim margin in PFS's design (for example, the factor of safety against sliding would be less than 1 if PFS were not to use soil cement). Stone & Webster should have all the necessary data from the structural analysis and design of the mat to make a determination on the validity of the assumption for rigidity of the mat. If the mat is not rigid, the soil damping used in the dynamic analysis is excessive and the seismic loads for the CTB are underestimated.

Q. 41: In addition to the concerns that you have raised regarding the cask, pad and CTB stability calculations, do you have other concerns that have not been discussed or that you wish to further explain?

A. 41: (FO) Yes. Because Holtec's analysis of the cask-pad-soil cement is a nonlinear analysis, it is very important to consider all potential variations in the motion of the pad and the casks. The stability of the cask is dependent, in part, upon the response of the pad foundation to resist seismic loads and assumptions in calculating the seismic loads.

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[10] *See* Calc. No. SC-4, Rev. 2, *Development of Soil Impedance Functions for Canister Transfer Building*, 3/21/01 (SWEC); (2) Calc. No. SC-5, Rev. 2, *Seismic Analysis of Canister Transfer Building*, 4/4/01 (SWEC).

PFS has not considered the range of applicable phasing of the foundation pad motion and the cask motions, the actual interface conditions between the casks and the pad on cement-treated soil, the applicable wide range of phasing relationship in input time histories and types of earthquake waves striking the pads, and the effect of pad-to-pad interaction with pads only 5 ft apart in the longitudinal direction. If the pads and the casks move out of phase, significant instability conditions arise.

Q. 42: Are there any factors PFS has not considered that may result from the PFS site being located close major faults?

A. 42: (FO) Yes. Since the PFS site is located close to a set of major faults dipping under the site (*see Development of Design Basis Ground Motions for the Private Fuel Storage Facility*, Rev. 1, March 2001, Geomatrix), seismic waves arriving at foundation structures are not necessarily vertically propagating waves. This is contrary to Holtec's assumption. The waves striking at angles will cause additional rocking and torsional motion of the foundation above and beyond the motion caused by vertically propagating waves. ASCE 4-98 requires consideration of non-vertically propagating waves in the dynamic analysis to ensure the effect of such waves are properly included in the design. While the effect of non-vertically propagating waves is less for smaller foundation such as pads, its consequence on the sliding movement of the casks may not be small. On the other hand, the effect on the large foundation of the CTB is expected to be significantly more.

Q. 43: What is ASCE 4-98?

A. 43: (FO) The full title of ASCE 4-98 is *Seismic Analysis for Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures*, published by the American Society of Civil Engineers in 1998. The purpose of ASCE 4-98 is to provide "minimum requirements and acceptable methods for the seismic analyses of safety-related structures of a nuclear facility." ASCE 4-98 §1.1; excerpts from ASCE 4-98 included as State's Exhibit 118.

Q. 44: Are there other requirements of ASCE 4-98 that PFS should follow?

A. 44: (FO) Yes. PFS's failure to consider non-vertically propagating waves or the alternative option to use accidental torsion is not in compliance with ASCE 4-98 requirements. ASCE 4-98 at § 3.3.1.2, State's Exh. 118. Another requirement of ASCE 4098 not considered by PFS is the use of multiple time histories in the non-linear analysis ASCE 4-98 § 3.2.2.3.2(d). *See* id.

Q. 45: Is Holtec's analysis sensitive to phasing of the input motion and, if so, what effect does this have?

A. 45: (FO) Yes. The nonlinear analysis in HI-2012640 can be very sensitive to phasing of the input motion and thus multiple time histories should be used. Only one set of time histories, developed by Geomatrix (Calc. No. F(PO18)-3, Rev. 1, *Development of Time Histories for 2000-year return period design spectra*, Mar. 21, 2001), has been used by Holtec. ASCE 4-98 requires multiple sets of time histories to be used in the nonlinear analysis in order to include the effect of time history variation on the design. ASCE 4-98 at § 3.2.2.3.2(d). State's Exh. 118. Also, based on my experience, the common industry practice for nonlinear calculations is to use at least three sets of time histories because the nonlinear analysis is sensitive to phasing. In order to cover the variation of the phasing in the design, a minimum of three (or sometimes four) time histories are used. This is an important safety consideration that PFS has failed to address.

Q. 46: What other aspects of the cask sliding on the pads have not been considered by Holtec?

A. 46: (FO) The potential for cold bonding and how it may influence the sliding of the casks were not considered by Holtec. Cold bonding is a phenomenon that occurs when two bodies (cask and pad) with such a large load (the cask) are in contact. Some local deformation and redistribution of stresses may occur at the points of contact, which would create a bond. For example, years pass, and the cask is applying stress on the pad. Over many years there is a local deformation that takes place at the contact points. It creates what is commonly called in the industry cold bonding. Even though you started with the concept of a smooth surface with limited friction, because of the stress on the contact points over time, however, there may be a bonding, a welding taking place and you may no longer have this smooth, ready to slide condition such as the one Holtec relied upon in its analysis.

Q. 47: Does this conclude your testimony?

A. 47: (FO/SB) Yes.

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EDUCATION:

Ph.D., Civil Engineering
University of California, Berkeley, California, 1983.

SUMMARY:

<u>15 Years</u>**:** Extensive experience in dynamic analysis and seismic safety evaluation of above and underground structures and subsurface materials. Co-developed and implemented SASSI, a system for seismic soil-structure interaction analysis currently in use by the industry worldwide. Developed a method for liquefaction hazard analysis currently in use for critical facilities in the United States.

EXPERIENCE:

As Chief Soils Engineer with Bechtel, San Francisco office, Mr. Ostadan was responsible for providing guidance and support to all projects in the areas of earthquake resistant design, dynamic analysis of structures, soil-structure interaction (SSI) analysis, and seismic stability evaluation of subsurface materials. He has participated in seismic studies and reviews of numerous nuclear structures, offshore structures, underground structures and transportation structures; conducted technology transfer and training courses for engineers of various companies and institutes including Bechtel Corporation, Impell Corporation, General Electric Company, SEAONC, Westinghouse Corporation, Lawrence Livermore Laboratory, and Tennessee Valley Authority (TVA) in USA; Kraftwerk Union, AG West Germany; Tractional Inc., Belgium, Nuclear Data Corporation, Japan; Atomic Energy Organization, Iran.

Major project work includes seismic analysis and evaluation of responses for: the Diablo Canyon Nuclear Station as part of the Long-Term Seismic Program (LTSP); NRC/EPRI large scale seismic experiment in Lotung, Taiwan; large underground circular tunnel for Super Magnetic Energy Storage (SMES); General Electric ABWR and SBWR standard nuclear plants; Westinghouse AP600 standard nuclear plant; Tennessee Valley Authority (TVA) nuclear structures (Browns Ferry, Sequoyah, Watts Bar); several facilities involving liquid gas storage tanks; Heerma TTP offshore structure in the North Sea; seismic stability and liquefaction study at the ITP, RTF, and K-facilities in the Savannah River Site for the Department of Energy; several transportation projects including numerous Caltrans bridges in California; BART extension lines including tunnel and aerial structures along the Dublin and San Francisco airport lines, Muni

State's
Exhibit 110

Metro Project, Downtown San Francisco; and Richmond Parkway Project in the San Francisco Bay area.

EXPERIENCE (cont'd)

1983 – 1985: Earthquake Engineering Technology Inc., San Ramon, California, As Project Engineer was responsible for development of a method for nonlinear seismic soil-structure interaction analysis in time domain.

1979 – 1983: University of California, Berkeley. As Research Assistant in the Civil Engineering Department, duties included development of the flexible volume method for dynamic SSI analysis of soil-pile-structure systems; member of SASSI development team.

PROFESSIONAL REGISTRATION:

Registered Civil Engineer, California

PROFESSIONAL ASSOCIATIONS:

Member of American Society of Civil Engineers
Member of EERI, Earthquake Engineering Research Institute
Member of Sigma Xi, The Scientific Honor Society, University of California, Berkeley

PUBLICATIONS

Technical Papers:

Lysmer, J., Tabatabaie-Raissi, Tajirian, F., Vahdani, S., Ostadan, F., SASSI - A System for Analysis of Soil-Structure Interaction, Report No. UCB/GT/81-02, Geotechnical Engineering Department of Civil Engineering, University of California, Berkeley, April 1981.

Ostadan, F., Dynamic Analysis of Soil-Pile-Structure Systems, Ph.D. Dissertation, University of California, Berkeley, 1983.

Ostadan, F., Udaka, T., Okumura, M., One Dimensional Seismic Response Study Using Different Soil Models, 8th SMIRT Conference, Brussels, Belgium, 1985.

Ostadan, F., Lysmer, J., Dynamic Analysis of Directly Loaded Structures on Pile Foundations,

8th SMIRT Conference, Brussels, Belgium, 1985.

Ostadan, F., Lysmer, J., Simplified Dynamic Analysis of Soil-Pile-Structure Systems, 5th International Symposium & Exhibition on Offshore Mechanics and Arctic Engineering, Tokyo, Japan, 1986.

Technical Papers (cont'd):

Ostadan, F., Tseng, Wen S., Lilhanand, K., Application of Flexible Volume Method to Soil-Structure Interaction Analysis of Flexible and Embedded Foundations, 9th SMIRT Conference, Lausanne, Switzerland, 1987.

Ostadan, F., Tseng, Wen S., Effect of Foundation Flexibility and Embedment on the Soil-Structure Interaction Response, 9th World Conference on Earthquake Engineering, Tokyo, Japan, August 1988.

Ostadan, F., Tseng, Wen S., Effect of Site Soil Properties on Seismic SSI Response of Deeply Embedded Structures, ASCE Foundations Engineering Congress, Evanston, Illinois, June 1989.

Ostadan, F., Tseng, W. S., Sawhney, P. S., Liu, A. S., The Effect of Embedment Depth on Seismic Response of a Nuclear Reactor Building Design, 10th SMIRT Conference, Los Angeles, California, August 1989.

Ostadan, F., Arango, I., Oberholtzer, G., Hsiu, F., Radially Loaded Circular Tunnel Structure, IX Panamerican Conference on Soil Mechanics and Foundation Engineering, Vina del Mar, Chile, August 1991.

Ostadan, F., Marrone, J., Arango, I., Litehiser, J., Liquefaction Hazard Evaluation: Methodology and Application, 3rd U.S. Conference on Lifeline Earthquake Engineering, Los Angeles, California, August 1991.

Ostadan, F., Hadjian, A. H., Tseng, W. S., Tang, Y. K., Tang, H. K., Parametric Evaluation of Intermediate SSI Solutions on Final Response, 11th SMIRT Conference, Tokyo, Japan, August 1991.

Ostadan, F., Arango, I., Litehiser, J., Marrone J., Liquefaction Hazard Evaluation, 11th SMIRT Conference, Tokyo, Japan, August 1991.

Ai-Shen Liu, G. W. Ehlert, R. S. Rajagopal, P. S. Sawhney, F. Ostadan, Seismic Design of ABWR and SBWR Standard Plants, ICONE2, San Francisco, California, March 1993.

R. S. Rajagopal, S. Sawhney, F. Ostadan, Seismic Considerations for the Standardized Advanced Light Water Reactor (ALWR) Plant Design, American Power Conference, Chicago, Illinois, April 1993.

I. Arango, F. Ostadan, Qualification of Liquefaction Hazard and Its Application to Risk Assessment and Urban Zoning, 5th International Conference on Seismic Zonation, Nice, France, October 1995.

F. Ostadan, S. Mamoon, I. Arango, Effect of Input Motion Characteristics on Seismic Ground Responses, 11th World Conference on Earthquake Engineering, Acapulco, Mexico, June 23-28, 1996

Technical Papers (cont'd):

I. Arango, F. Ostadan, M. Lewis, B. Gutierrez, Quantification of Seismic Liquefaction Risk, ASME PVP & ICVT Pressure Vessel and Piping Conference, Montreal, Quebec, Canada, July 21-26, 1996.

F. Ostadan, T. Liu, K. Gross, R. Orr, Design Soil Profiles for Seismic Analyses of AP600 Plant Standard Design, ASME PVP & ICVT Pressure Vessel and Piping Conference, Montreal, Quebec, Canada, July 21-26, 1996.

Computer Programs:

User's Manual, Theoretical Manual, and Verification Manual for Computer Program SASSI.

Installation and Validation Reports for Computer Program SASSI prepared for: EDS Nuclear Incorporated, California; Kraftwerk Union, AG, West Germany; Tractional Incorporated, Brussel, Belgium; Bechtel Corporation; General Electric Company; Westinghouse Corporation; Lawrence Livermore Laboratory.

User's Manual, Verification Manual, and Application Manual for Computer Program NANSSI (nonlinear analysis of soil-structure systems), Kozo Keikaku Engineering, Japan.

User's and Theoretical Manuals for Computer Program ASHLE (Advanced Seismic Hazard/Liquefaction Evaluation), Bechtel Corporation.

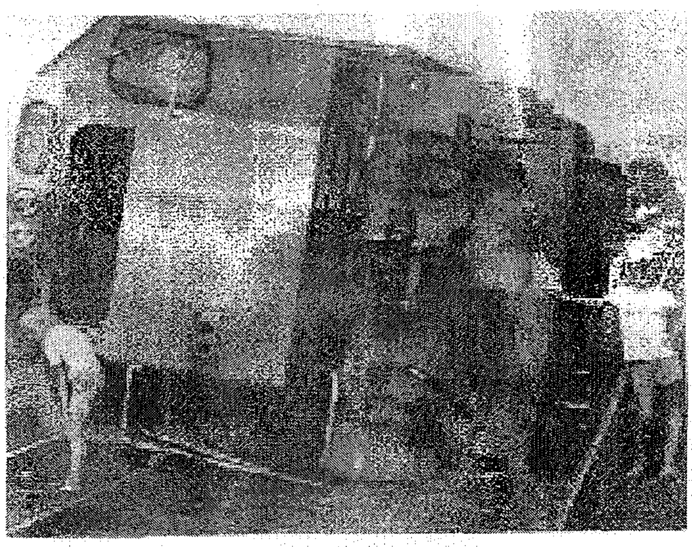


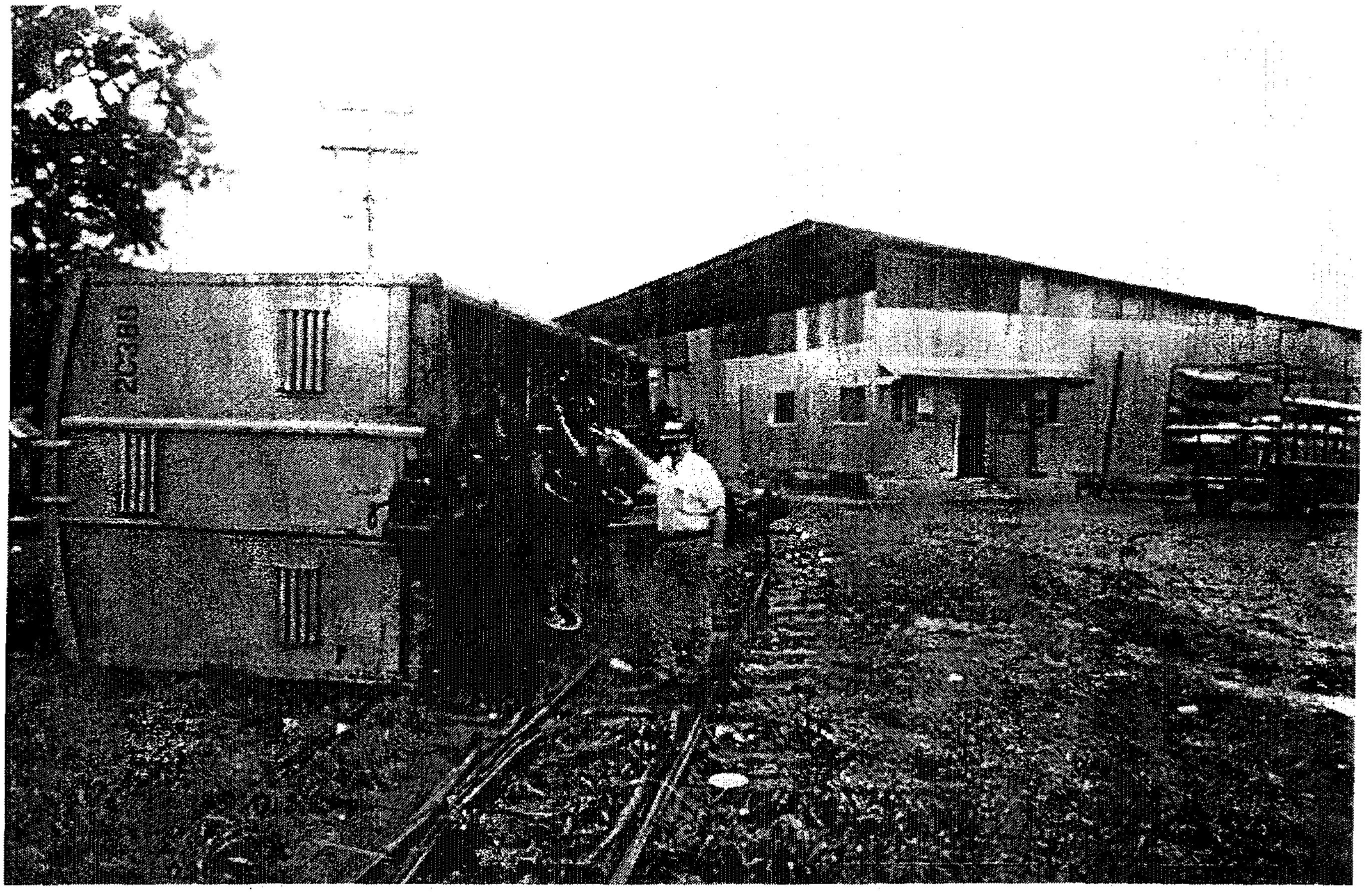
Figure 9: *Edgecumbe.* Overturned 67-ton diesel train engine in the rail yard. The engine was stationary at the time of the earthquake. The driver, who was in the cabin, reported that the engine moved both side to side and up and down before overturning. He escaped uninjured from the top side.

Source of Fig. 9 - Summary of the 1987 Bay of Plenty, New Zealand Earthquake, EQE International, 1987.

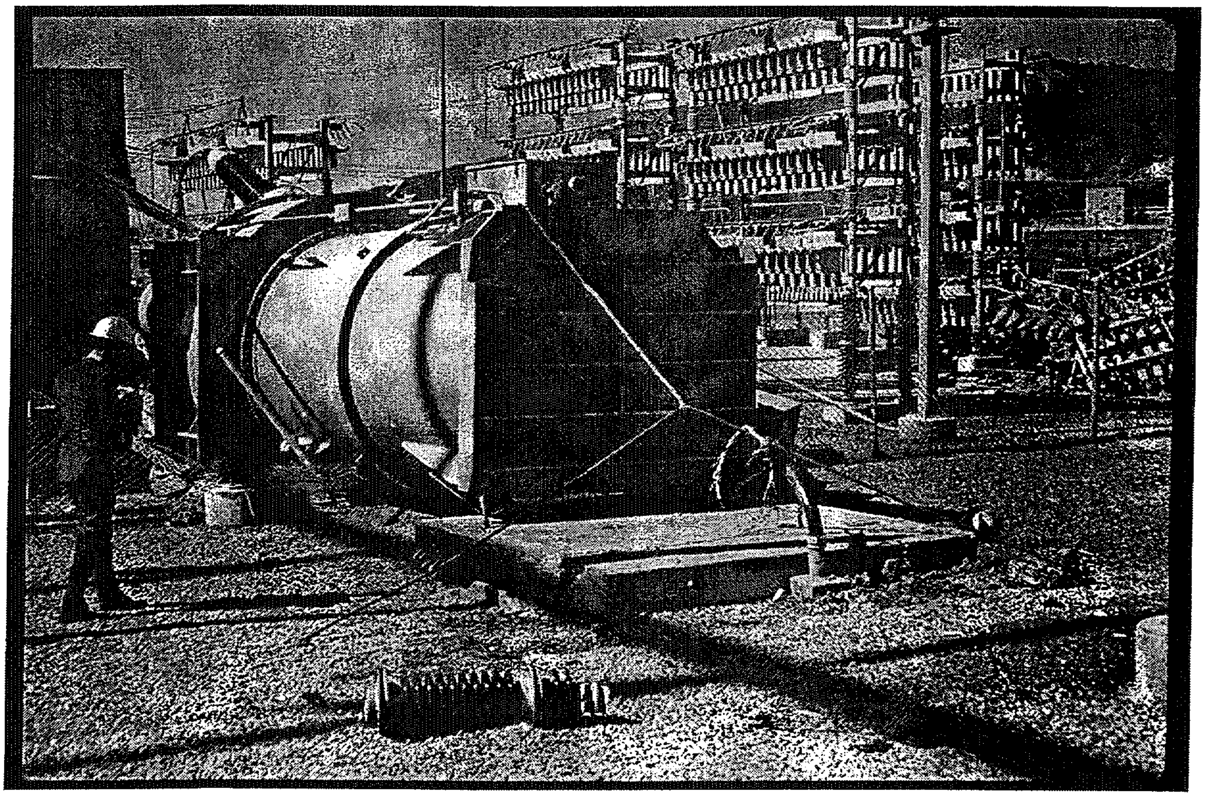
The earthquake magnitude was a $M_L = 6.2$. The focal depth of the earthquake was estimated at 6 miles. The earthquake produce approximately 6 miles of discontinuous surface rupture and a complex main scarp about 3.5 miles long striking southwest roughly 0.5 miles east of Edgecumbe. No strong ground motion instrumentation was available, but EQE investigators estimate horizontal pga of 0.5 to 1.0 g in Edgecumbe.

The EQE team also report that damage to smaller unanchored tanks and equipment was widespread. However, all equipment at a steam plant was anchored and the damage was superficial.

State's
Exhibit 111



Overturned railroad boxcars at banana packing plants near Sixaola on the Panama-Costa Rica border (photo and caption from Slides on the Costa Rica Earthquake of April 22, 1991 - Set III: Performance of Industrial Facilities and Lifelines - Earthquake Engineering Research Institute).



"Overturned equipment was badly anchored."

Location: Sylmar Converter Station
Feb. 9, 1971 San Fernando Earthqukae
M = 6.5

Source of slide:

Steinbrugge Collection, Earthquake Engineering Research Center, University California, Berkeley.

Slide No. S4301

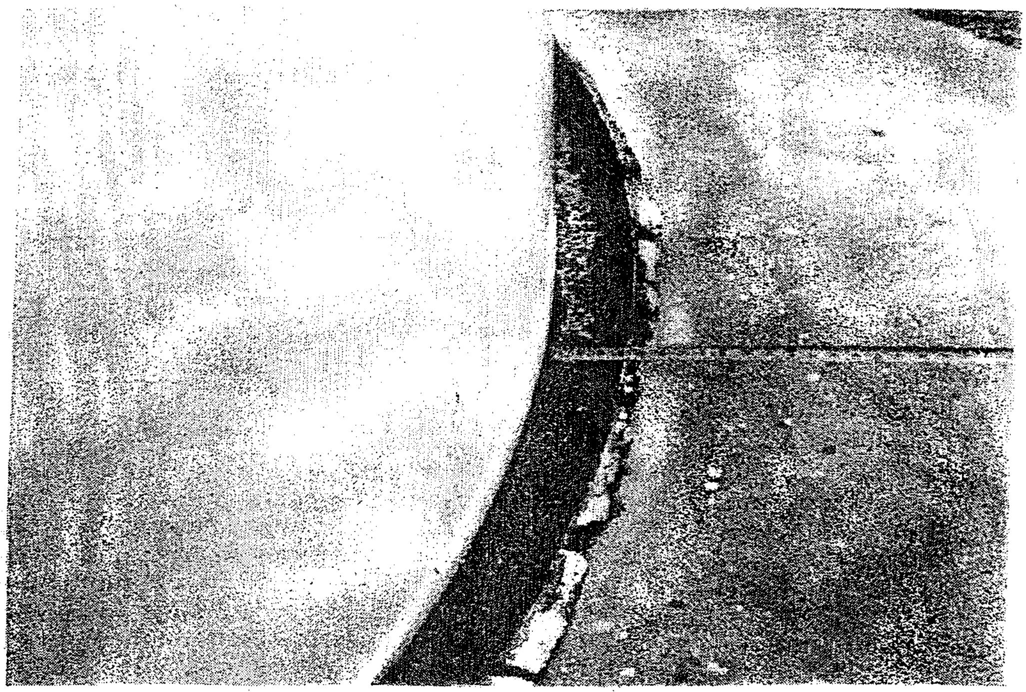


Figure 6: The four gasoline storage tanks located in the southern end of Coalinga did not leak in spite of sliding about 4 inches, as shown in the lower photograph. Sliding was common for unanchored equipment and structures throughout the area.

Source of Fig. 6 - Summary of the May 2, 1983 Coalinga, California Earthquake, EQE Incorporated, 1986.

The earthquake magnitude was a $M_L = 6.7$. It was centered near the town of Coalinga. Available ground motion records in the Coalinga vicinity indicate that peak ground accelerations were high. Depending on the location, peak accelerations ranged from 0.20 g to over 0.60 g.

At the Shell water treatment plant the following was reported:

"Extensive sliding of unanchored tanks with rupture of attached piping. Yielding of supports for anchored tanks." (p. 3).

LEARNING FROM EARTHQUAKES

LESSONS FROM LIFELINE ENGINEERING ON ELECTRIC POWER SYSTEMS:

- THE MOST IMPORTANT DAMAGE AND LOSS CONTROL PROCEDURES INCLUDE:
  1. *SEISMIC TIES* TO PREVENT LARGE RELATIVE DISPLACEMENTS AND POUNDING BETWEEN STEAM GENERATION EQUIPMENT AND SUPPORTING STRUCTURES.
  2. *ANCHORAGE* OF EQUIPMENT AND TANKS TO PREVENT OVERTURNING, SLIDING, AND POUNDING AS WELL AS PIPING-SYSTEM INTERATION PROBLEMS.
  3. *DESIGN* OF SUPPORTING STRUCTURES FOR EQUIPMENT TO ACCOMODATE THE FREQUENCY-DEPENDENT EFFECTS OF SOIL AMPLIFICATION OF GROUND MOTION.
  4. *DESIGN* OF CONSTRUCTION JOINTS TO PROVIDE AN ADEQUATE GAP TO ELIMINATE IMPACT BETWEEN TURBINE PEDESTAL AND POWERHOUSE STRUCTURES.
  5. *DESIGN AND INSTALLATION* OF BATTERY RACKS WITH ADEQUATE ANCHORAGE TO PREVENT COLLAPSE.

From: Slide set - Learning From Earthquakes IV (LFE IV) Earthquake Engineering Research Institute.

This slide shows that a lesson learned from the performance of past earthquakes is:

"Anchorage of equipment and tanks to prevent overturning, sliding, and pounding as well as pipe-system integration problems."

Thus, past experience and common sense suggests that the storage casks at the PFS facility should be anchored to the pads.

A common misconception is that heavy objects are not easily moved by earthquakes. Actually, their large mass means they experience large inertial loads. Newton's second law formula, F = ma, or the inertial force equals the mass times the acceleration, conveys this idea quantitatively. This 93-ton bronze statue of Buddha in Kamakura, Japan slid more than a foot in the 1923 Kanto earthquake. (Photo and caption from Non Structural Damage Slide Set, Earthquake Engineering Research Institute).

From DOE-STD-1020(94), p. C-59

"Engineered anchorage is one of the most important factors affecting seismic performance of systems or components and is required for all performance categories. It is intended that anchorage have both adequate strength and sufficient stiffness to perform its function. Types of anchorage include: 1) cast-in-place bolts or headed studs, 2) expansion or epoxy grouted anchor bolts and 3) welds to embedded steel plates or channels. The most reliable anchorage will be achieve by properly installed cast-in-place bolts or headed studs, undercut type expansion anchors, or welding. Other expansion anchors are less desirable than cast-in-place undercut, or welded anchorage for vibratory environments (i.e., support of rotating machinery), for heavy equipment, or for sustained tension supports. Epoxy grouted anchorage is considered to be the least reliable of the anchorage alternatives in elevated temperature or radiation environments.

CONDENSED TRANSCRIPT

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

In the Matter of PRIVATE FUEL STORAGE, LLC, (Private Fuel Storage Facility)	Deposition of: Farhang Ostadan Docket No. 72-22 ASLBP No. 97-732-02-ISFSI

March 8, 2002 - 10:30 a.m.

Location: Parson, Behle & Latimer

201 South Main Street, #1800

Salt lake City, Utah 84145

Reporter: Diana Kent, RPR

Notary Public in and for the State of Utah

State's
Exhibit 112

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SHEET 12 PAGE 89

89

A. That's right.
Q. Okay. Very good. Is there any portion of the calculation that led you to conclude that the pads are not rigid?
A. Yes. So many of the results towards the end of the calculation that summarize the displacements, vertical displacements. I think they performed two analyses. One was with C-Star or a SAP program. I forget. And the other with SASSI. They showed the results.
Q. This may refresh your memory. I'm going to mark this one as Exhibit Number 30.
(EXHIBIT-30 WAS MARKED.)
Q. Let me identify for the record this document. It is a document titled Declaration of Dr. Farhang Ostadan, it is dated January 30, 2001, and it bears the caption of this proceeding. Are you familiar with this document that's been marked as Exhibit 30?
A. Yes, I remember it.
Q. Did you prepare it?
A. Yes.
Q. Okay. Now, you will turn to Page 6 of Exhibit 31.
A. Okay.

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Q. Paragraphs 24 and 25.
MS. CHANCELLOR: Just want to place an objection on the record. To the extent this deals with Contention L, it is not part of this deposition. The witness may go ahead and answer. I notice that it relates to summary disposition of Utah L.
MR. TRAVIESO-DIAZ: I am happy to represent to you that the question and the answers are not going to deal with Contention L at all.
MS. CHANCELLOR: Okay.
Q. (By Mr. Travieso-Diaz) Take a look at Paragraphs 24 and 25.
A. I read that, yes.
Q. And 25.
A. Yes.
Q. Now, looking at Paragraph 25, does that refresh your memory as to what portions of the calculations --
A. Yes.
Q. Is that table 5.2.5-1 on Page 214 the one you are thinking about?
A. And there are other tables. There are a bunch of tables. But this is one place, one example you can find this.
Q. All right.

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(EXHIBIT-31 WAS MARKED.)
Q. For the record, I would identify what Exhibit 31 is. Exhibit 31 is comprised of the cover page of I believe the same calculation that you made reference to on Paragraph 25 of Exhibit 30. And the second page of the exhibit is Figure 5.1-1 of the calculation.
A. Right.
Q. The next page of the exhibit is Section 5.2.5 of the calculation. The next page of the exhibit is table 5.2.5-1. And the last page is table S-2?
A. Right.
Q. Will you turn to table 5.2.5-1?
A. Yes.
Q. Which I believe is the one you made reference to --
A. Yes.
Q. -- on the other exhibit.
A. Yes.
Q. Would you tell me, perhaps by reviewing that table, where the excitation, where the load was applied in this analysis?
A. I don't think it indicates where the loads were applied, if that is what your question is. I can't see that here.

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Q. Would you look at the notes under the table itself in the first sentence. That could help you.
A. Which table?
Q. Same table. Just the explanatory note.
A. Just the note?
Q. Yes.
A. Okay. I understand the note. But it does not tell you where the loads were applied, if that was your question.
Q. The way I read the note, and maybe you can correct me if I'm wrong, says the application of the load was on node 249.
A. No, that's not true. What he is saying is that near application of the load, that's near application, there's 10 percent difference between the two results.
Q. Okay.
A. But the loads are applied at the interface point between the cask and the pad, depending on how many casks you have; if you have two, four, or eight.
Q. So it would be at the edge of the cask, the place where the cask --
A. No. I think for vertical, if I'm not mistaken, you have or they provided Holtec four vertical time histories, force time history, at four points. And

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for horizontal, I think they provided one time history for each direction and CEC divided the nodes on the contact points. It's all in the calculations.

Q. Why don't you turn to the page that has text that is labeled 5.2.5. Look at the second paragraph on that page and tell me if that refreshes your recollection of where the force was applied or the load was applied.

A. This is one case they studied which they are talking about single vertical time histories applied at the second quadrant of the first cask, node 249. That is one study case. But that is not their basic case for design.

Q. What I'm trying to see if I understand from you is for the case that is displayed on Table 5.2.5-1, whether your understanding, by looking at the note under the table and explanatory text on the preceding page, whether your understanding is that for that case, the load was applied at node 249.

A. I need to look at the whole calculation. I think what you see here in this table is not what is talked about in the second paragraph.

Q. Is it your view that the description 5.2.5 doesn't apply to the computation which results are shown on Table 5.2.5-1?

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A. No. The description of 5.2.5 is very general in the first paragraph. They talk about what they did and the time histories shown and the figures and so on and so forth. I'm not certain these results you are showing relates to the specific case they are talking about in the second paragraph or does it relate to a generic case where casks are all there and loads are employed at a contact force. But I can assure you -- it's a very good calculation, actually. But I can assure you that there are other tables that they show clearly where loads are applied and what the results are.

Q. I do happen to have the calculations here. I hesitate to introduce it as an exhibit because it is several hundred pages long. This is what I would like to do in the interest of saving time: If you could look at this at a break and tell me after the break whether you agree or disagree, based on your review, that in fact the load is applied on node 249.

A. Okay.

Q. So to save time, let's proceed on the assumption that the load is applied at node 249 and if it is not, then all your answers would be of no significance.

A. That's a pretty poor assumption. But it

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doesn't reflect the reality that the loads are applied only on one node.

Q. But that's actually what I'm trying to get to. I guess that in order to assess the results on table 5.2.5-1 you have to understand what case was analyzed.

A. Fair enough.

Q. And what load was applied where. So my understanding, and I think it is pretty good, is that for that case the load was applied on node 249. We can do it two ways. We can take time off now, take a break, and give you whatever time you need to review this calculation, or else we can proceed on the assumption that it was on node 249 and you can confirm that it was or tell me that it was not.

MS. CHANCELLOR: I'd instruct the witness to review the calculations so that he is not guessing or that the record will accurately reflect what his opinion is.

Q. Okay.

(A break was taken.)

Q. While we were on the break we decided that we are going to mark as a separate exhibit, and that will be number 32, another table, table D-1(d) of the ICEC calculations, parts of which were previously marked

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as Exhibit 31.

(EXHIBIT-32 WAS MARKED.)

Q. And what we are going to do, if I understood our conversations during the break, first I'm going to ask you questions on Exhibit 31 under the assumption that the load for the table that is presented on Table 5.2.5-1, that the load on that case is applied in node 249. And then we will talk about Table D1(d). Is that agreed?

A. Very well. That's fine.

Q. Assuming that the load, the single load, is applied on node 249 -- would you refer back to Figure 5.1-1 on Exhibit 31. It's the second page of the exhibit.

A. Yes.

Q. Is that sort of a map showing where the pads and the casks are with respect to the nodes that were considered in the analysis?

A. Yes. It is a finite-element model of CECSAP, yes.

Q. So the record is clear, what do we mean by "nodes" in the finite-element analysis?

A. The mat has been discretized and element node numbers have been assigned for the analysis.

Q. So again for the not trained, including

myself, that means that the model essentially represents the structure as a series of points or nodes?
A. **That's right.**
Q. All right. And you take a look at Table 5.1-1. Node 249 would be essentially at the edge of Cask 1. You could say on the lower quarter of the pad, if you will, that is under Cask 1. Is that correct?
A. **Yes. That's correct.**
Q. All right. Now, if you applied a single load on a node located such as node 249, would you expect to get uniform responses or uniform deformations across the entirety of the casks and the pads underneath?
A. **Assuming the load is applied only at node 249?**
Q. Correct.
A. **I would not expect to see constant displacement on all nodes.**
Q. Turn for a second with me to Table 5.2.5-1 and you will have to flip back between the map and the table. Let's look at nodes 222, 235, 248, 261, and 274. Would those nodes represent the left edge of the pad where the load was applied on node 249?
A. **Would you slowly go over the node numbers again?**

Q. 222, 235?
A. **Just one second. Okay.**
Q. 248?
A. **Yes.**
Q. 261?
A. **Right.**
Q. 274?
A. **Yes.**
Q. If you take a look at the table, what I believe are displacements?
A. **Yes.**
Q. And would you look at the displacements for each of those nodes that I mentioned to you; 222, 235, 248, 261, 274. Those are the nodes that are the closest to the applied load; right?
A. **That's right.**
Q. Do you see a difference in the amount of vertical displacement when you go from, say, node 222 to node 274?
A. **A small difference.**
Q. Is that what you would expect in a case like we are talking about; a single load applied to a single node and you have different displacements depending on your distance from the application of the load?

A. **Let me make the observation based on the assumption we have made. That the load is being applied only at node 249.**
Q. Absolutely, yes.
A. **As you indicated before, rigidity is a relative measure. If that is the case, only one load is applied to the pad, this is unrealistic with the real field condition that we might have, two, four, six or eight casks. So the total earthquake loads are not being applied here. If our assumption is correct, this seems to be a parametric study which just applied at one node, one vertical time history.**
Q. And if, in fact, the assumption that the load was applied at node 249 is correct, would it be appropriate to look at the displacements shown on this table as representing the behavior of the pad under an earthquake excitation?
A. **No, it would not. Exactly my point.**
Q. Okay. So this, in fact, looking at this table for purposes of determining displacement would not be the thing to do?
A. **With that assumption that we have made.**
Q. Of course. Assuming that the load was put where we said.
A. **That's correct.**

Q. Let's turn to your Table D1(d) for a second. And since you suggested we look at it maybe you can tell us what we should look at it for.
A. **Maybe I should do what?**
Q. Tell me what we should look at it for.
A. **What we are looking at now, ICEC calculation page number 234 in which they show a summary of the vertical displacement and the bearing pressure for various scenarios they have analyzed. And the scenarios are for load bounce soil properties, best estimate, and upper bound. And each case has been analyzed for cases with two cask conditions, four cask conditions, and eight cask conditions. And we see vertical displacement at various nodes. So if I go, for example, to a two cask lower bound case, I'll see node 1, 7, and 13 have a displacement amplitude of 4 to 4.7. Of course there's a scaling factor on top of the table. But then for the same load case, same soil case, if you move down to node 287, 293, 299, you see displacement three to five times larger there.**
Q. Okay. Are nodes 287, 293, and 299 on the same pad as nodes 1, 7, and 13?
A. **I expect them to be all on the same pad, yes.**
Q. So this would be going from the -- we are

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going to look at the map, assuming that 1, 2, 3 is the edge of the pad. And 287, 288, and 289 are at the other edge.
A. That's right.
Q. Tell me what learning we derive from looking at the displacements across the two edges.
A. Well, what you are seeing is the displacement varies by a factor of four to five times.
Q. Now, is this a case in which there was uniform loading applied to the cask or what conditions under which the load was applied?
A. If you look at the two-cask column, the two casks are being loaded and the loads responding to two casks are being applied.
Q. I'm sorry. Where is that load applied?
A. Okay. For two casks, at the beginning of the calculation they clearly define which nodes are being loaded for two casks, which nodes are being loaded for four casks, and so on. It's not in this table here, but it's been defined in the cask.
Q. My question to you is are they placing a single load or loads on various nodes? What loads are applied where?
A. And my understanding is, again, we are talking about the loads which are dynamic loads. They

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are time histories provided by Holtec. For vertical force, Holtec provides four time histories at the four corners of the cask. And for horizontal, if I'm not mistaken, they provide one in each direction. But CECSAP divides to four location? So they are uniform. I can't tell which nodes are being loaded here based on this table. We have to go back to the few earlier pages of calculations to identify. I don't know those nodes.
Q. Well, what I'm trying to see if I can understand you help me to figure out, is with respect to nodes 1, 7, and 13, whether the load that has been applied are symmetrical with respect to those three nodes 1, 7, and 13 as the load that is applied to a corresponding other edge of the mat, which would be 287, 288, and 289?
A. There's no load applied to the edges of the mat. For example, let's look at the two-cask. I'm looking on page or sheet number 20 of Exhibit 31 where they show the final element for CECSAP. So for example, let's say they are analyzing the two-cask scenario. We see on the top part of this figure there's Cask 1 and Cask 2. So what I expect to have done is the vertical time histories for each cask were applied at the four corners of the cask. For example, cask 1 would be 249, 225, 253, 277 if I read this correctly. And so on for

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Cask 2. So that's how the load is applied.
Q. So your understanding is that the load is applied at the four corners, if you will, of Cask 1? And where would the load be applied with respect to Cask 2?
A. The same four corners. Just follow the same logic here; 254, 231, 259, and 283.
Q. So this is the situation in which you would apply load -- would you assume that the other casks are present on the pad or only those two casks?
A. There are three scenarios they analyzed. In one, only two casks were present. The other one, four casks were present. And in the third one, all eight casks are present. So they have analyzed the three scenarios.
Q. All right. And for the eight-cask scenario, you would be looking at the tables on this Exhibit 32 that are labeled "8 casks"?
A. That's right.
Q. And I see that those tables have LB, BE, and UB as captioned. What do you think those are?
A. Load bounce profile, best estimate profile, upper bounce profile.
Q. So if you wanted to find out what is the computation's best estimate of the displacement, you

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look at the middle column?
A. That's right.
Q. If you were to look at, say, the eight-cask case, and you assumed that the load combinations are as you described before and now applied to all eight casks, the best estimate of displacements would be on the column that says BE, 8 casks?
A. That's right.
Q. All right. And what you would ask us to concentrate on would be, for example, the displacements on nodes 1, 7, and 13 versus the displacements on nodes 287, 293, and 299?
A. And you have the middle one, too; 144, 150, and 156 here.
Q. What conclusion do you derive by looking at that column?
A. I look at this and I see node 150 has a value of 12.39. And the maximum value I see in this column corresponds to node 1, which has 23.66. So almost a factor of two.
Q. And what physical reality, if you will, what does that --
A. That tells me the cask or the pad is not deforming rigidly. It has little deformation.
Q. Would you translate the dimensions of this

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displacement to me into inches or parts of an inch? It says, "Maximum displacements Zd (x 10-3 feet). Is that thousandths of a foot?

A. **Yes.**

Q. How many inches is a thousandth?

A. **Very small.**

Q. So you are saying that there is a factor or two difference between 12 (x 10-3) and 23.66 (x 10-3)?

A. **That's correct.**

Q. So your assumption as to whether this cask is flexible or rigid will be based on the difference in the displacement between those two points, whatever that is?

A. **Exactly. They are very small.**

Q. If it is very small, like a fraction of one inch, that would still lead you to the conclusion that there is flexion?

A. **It could be large. But if the difference wasn't there, you would assume it is rigid. They are small but there is a difference.**

Q. How small does the difference have to be before you can practically assume it is rigid?

A. **Well, I haven't done any separate calculation to suggest that number. But I think that suggests to me that the assumption of rigidity, full**

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rigidity of the mat, is not supported by these results.

I would like to point out another point as long as we are on Exhibit 32.

Q. Yes.

A. **Basically you were talking about whether the soil spring dash spots that were calculated and used by Holtec are appropriate or not, with respect to the foundation agility. If you go back to the column of two casks, and you notice the difference in sign, node 1, 7, 13 are positive, node 144, 150, 156 and others are negative. Do you see that?**

Q. Yes.

A. **What this tells me is that part of the pad is uplifting, it is moving up, whereas the other part is moving down. I don't know whether this movement is large enough to cause any suppression or not. But that also concerns me that under some condition, like two casks, while it vibrates you can potentially have the other edge of the pad separate from the soil which again goes back, in the assumption of calculation of spring and dash spot, assuming pad is rigid and in full contact with the soil is quite valid here. But this also violates that assumption.**

Q. Well, in terms of physical reality, understanding as we do that everything is deformable to

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some degree, wouldn't you expect that if you have a body and you apply force and there's deformation, a part of it will go up and a part of it will go down? Is there any way to avoid that?

A. **Yes. I agree there is always deformation. And I frankly would not bring up any of these comments if we had enough margin in our designs and in our foundation stability calculations. One would oversee these, and these differences might not be important. But when we talk about a very slim margin, these points become important. One has to make sure that they are properly reflected conditions we have.**

Q. Let me ask you a different question because we talked about this a little bit before, in connection with the angle of arrival of the waves and so on. But the question here is different. Can you tell from this table whether all the displacements occur at the same point in time?

A. **I cannot tell that, no.**

Q. Is it possible that if you were to compute for the eight cask case, the displacement at node Number 1 which is minus 23.66, and you would compute the displacement at node 150, which is minus 12.39, and the times were different, that you could get a different result?

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A. **It is likely possible.**

Q. And all the table says is this is the maximum displacement. It doesn't say it was the maximum simultaneous displacement; right?

A. **I agree with you.**

Q. Any other observation you want to state on this?

A. **I want to follow, based on your notion, if you look at the specific time, the differences could be larger or smaller.**

Q. That is true. Are you familiar with this ICEC calculation, not just this table but in general; what he was doing it for and the purpose and so on?

A. **Yes, I am.**

Q. Would you describe for the record why the calculation was run?

A. **ICEC calculation was primarily done to design the pads; structural design of the pad to come out with the rebars and the steel and the location of the rebar and steel.**

Q. So it was a design calculation?

A. **It was a design calculation.**

Q. You refer -- Interrogatory Number 5, the response. You refer to the Holtec calculation and I believe -- actually you refer to several calculations

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which have the same apparent problem. You have the stability calculation performed for Stone & Webster is on Page 13.
A. **Yes.**
Q. And you refer to the Holtec calculations at the beginning of the Answers to Interrogatory.
A. **Right.**
Q. Is it your view that all these calculations are similarly flawed in that they assume that the pads are rigid, whereas you --
A. **No. You are talking about two different rigidities here. Let me explain that.**
Q. Okay.
A. **The rigidity that I talk about with respect to Holtec calculations is really deformation of the concrete pad.**
Q. Okay.
A. **And whether or not that is valid. And the impact of that would be on the calculation of soil spring and dash points.**
Q. Okay.
A. **And coefficient of friction.**
The rigidity I talk about with respect to the Stone & Webster calculation, stability analysis, has to do with the way they have calculated the seismic load

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in the stability calculation. And the way they have calculated the seismic loads for stability analysis, they took the weight of the concrete pad, they took the weight of the casks, and for example for coefficient of .8 they observe a limit of the sheer that can be transferred to the pad based on that coefficient. But then they went ahead and calculated the inertia of the pad by using peak ground acceleration, which is a design motion and has nothing to do with the structural response or pad response. So this is only valid if the foundation, and I'm talking about the soil and whatever is under the pad, was fully rigid. If that was the case, then one could use the pga to estimate the inertia of the pad. But that is not the case; we have soil, and this foundation has a natural frequency, and therefore they should have used acceleration that corresponds to the natural frequency of the system, which is truly the structural response of the pad and not the design motion.
Q. See if I understand what you are saying. Even though both concerns you raised referred to rigidity, they are different structures that are covered by the concern, if you will. In the one case is the pads in the Holtec analysis, and in the other case it is not only the pads but the soil underneath in the case of

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the Stone & Webster analysis. Is that correct?
A. **Yes.**
Q. Now, concentrating for a moment on the Holtec analysis, what is your understanding of what that analysis was done for? For what purpose?
A. **The purpose of that analysis, 2000-year motion, was to show that casks sliding on the pads have limited displacements, they would not impact each other, and they would not tip over due to seismic excitation, and also generate seismic loads so it can be used to structurally design the pad.**
Q. Is the Holtec calculation a design calculation that results in design calculations and materials or --
A. **No. It just produced results that was used by ICEC.**
Q. Is it your experience in the many years of practice that when you have two calculations that are used for different purposes you may make differing assumptions and both calculations still remain valid?
A. **As long as the assumptions are conservative, that could happen, yes.**
Q. So if they are conservative, you could, for example, in the design calculation for the pads, take into account some stability because you are trying to

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come up with number, sizes, and so on. Whereas in analysis you could presume they are rigid; providing, as you said, that conservative assumptions are made. Correct?
A. **The assumption of rigidity of the pad in the Holtec analysis is unconservative.**
Q. Why is that?
A. **Because once you assume the pad is rigid, calculation of soil spring and soil damping, which play a very important role here, would not be correct. It overestimates the damping of the pads, and damping takes out seismic loads.**
Q. Would that overestimation depend on the extent of the actual deformation of the pad?
A. **Yes, it does.**
Q. So if it was a small deformation it might be unconservative but the error would be small?
A. **I think what is important in radiation damping is not really the amplitude of the displacement but the relative motion of the nodes. If the pad is rigid and moving together, it has a tremendous radiation capacity. It dissipates energy as it impacts the soil. But whereas when it is flexible and moves differently at different locations, no matter how much that difference is, you don't have this uniform motion and dissipation**

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phenomena. Therefore, the radiation damping would be overestimated by rigidity assumption.

Q. But what I'm trying to get a sense from you, if you have it, is how much does the loss of the ability to take credit for that rigidity and the way you described is impaired or reduced, if you have some flexibility in --

A. I don't have a number to propose but I said if I had a large number margin in design I wouldn't have raised this issue. We should view it in light of the margin we have.

Q. Is this calculation by Holtec you referred to the one in which they estimate -- well, what is the purpose or what are they looking at in that calculation? What are they computing?

A. The purpose of that calculation was to estimate the movement of the cask, whether or not the cask tipped over, and then generate seismic loads for design of the pads.

Q. Okay. And this is different from the calculation which we spoke about before that had to do with the potential tipover of the cask; is that right?

A. Yes. That's a different one.

Q. And your view is that this other calculation also has a very small margin?

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A. Yes. All events translate to the stability of the foundation which has a very small margin.

Q. Do you recall, based on your review, what the margin is in the calculation?

A. I think for sliding we are as low as 1.2.

Q. And you have or you don't know sitting here today how much would that margin be used if the extent of deformation of the pad as shown in Exhibit 32 were to be taken into account; do you?

A. I do not know how much it would impact that. But this issue, plus other issues combined, concerns me with that margin.

Q. Okay. Would you know how much the loads on the pad would change or the downward loads from the pad on the soil would change on account of taking the flexibility of the pad into consideration?

A. I know -- let me provide you with this observation: ICEC received the loads from Holtec and they applied it to the cask, the model of the pad, I'm sorry, the soil spring attached. As a result of this calculation, they calculated the total forces from the cask and the pad transferred to the soil and they are summarized in these tables. There's a force for X direction, Y direction, Z direction.

If you take the force that is, for example,

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one horizontal X direction, and you divide it by the total weight of the pad and the cask, you come out with the effective acceleration is something less than .6 g. This tells me a good deal of the force is missing. If we have this cask with this much weight and you had the pad with this much weight, even though the cask is sliding at .8, total inertia should add up to something larger than pga of design motion, which is .71 or so. So I think the ICEC calculation shows me that the loads that are given to them are not adequate. They do not reflect the total load of the cask and the pad.

Q. Let me clarify, because again I need to understand. When we talk about the load, are we talking about vertical loads or horizontal loads here?

A. At this moment I was talking about horizontal loads.

Q. Horizontal in terms of sliding.

A. Yes.

Q. You don't have any feel, sitting here today, how much of the horizontal loads would change?

A. Could be anywhere from 20 to 60, 70 percent.

Q. And is this based just on your prior experience?

A. It's a general judgment.

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Q. Okay. Would there be an impact on the vertical loads?

A. Yes. The vertical load, we have another dilemma. Stone & Webster performed a stability analysis of the paths. One key assumption there is you will look at the sliding and overturning of the pad, assuming horizontal earthquake and vertical earthquake are acting. And typically this calculation is done assuming the vertical force is working against you, is lifting the building in the opposite direction. And they have done that logic right, except that in selection of an acceleration to estimate the vertical inertial force, they again use the pga of design motion, which has nothing to do with the structural response. This is the lowest number on the design curve. There's no justification why they use the smaller number. I would have expected the number would be higher.

In fact, when I look at the ICEC set of results, they show the natural frequency of foundation when they apply the Holtec forces. The natural frequency for lower bounds are around 5 hertz and this estimate is on 8 hertz. Upper bounds is around 11 hertz. So if I have to pick acceleration for inertia, I will go to my design response spectrum using these frequencies and read off the acceleration rather than a

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pga, which is a very high frequency and smallest number on the curve.
Q. So that the record reflects this clearly, when you are talking about the natural frequency of foundations, what do you encompass in the term "foundation"? Is it a pad with soil underneath?
A. Pad, soil, and cask combined.
Q. So that the natural frequency will be an ensemble that comprises the cask, the soil, and the pads?
A. That's correct.
Q. And your view is that the natural frequency on that combination of soil, cask, and pads is somewhere between 5 and 11 hertz?
A. That's correct. And it is shown in the ICEC calculation.
Q. How is it shown in the ICEC? I take that calculation will give you information only as to how the pad behaves; right?
A. No. There's much more in there.
Q. Oh, tell me.
A. They have plotted what they call transfer functions. And that shows the frequency response of the system of soil, pad, and cask. And when the transfer function peaks to highest value, that's the natural

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frequency of the system. And it is clearly shown. If you go for lower bound, you see a number around 5 hertz, 8 hertz, 11 hertz. Now, on top of that, what you could do is take the weight of the pad and the cask, and do a simple frequency calculation of stiffness over mass. And the stiffness is given by ICEC in all directions. You would come out with the same numbers. You get about 5, 8, and 11, which is very consistent.
Q. Do you have a view as to what the natural frequency of the soil alone, assuming you have no pads or casks, is?
A. I haven't thought about this. I could look at it and come up with a view. But it doesn't really affect the design issues we are talking about. Not in my mind.
Q. Why not? Wouldn't you want to know the contribution that the pad would make, for example, for a natural frequency as opposed to the contribution you get from the soil?
A. No. I talk about the natural frequency of the cask, pad, and soil together. That's important. But you just talk about the natural frequency of the soil column alone, no. That is already included in the design motion in Geometrics' calculation and reflects in their time history. So it is taken care of.

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Q. Are you saying that what would be omitted, then, would be the contribution of the pads and the casks to a natural frequency, because you already have the soil included in the input?
A. What is immediate here is in the Stone & Webster estimate of seismic load for the path, in the horizontal and vertical direction, they use the pga of design motion, which has nothing to do with the structural response, the pad response. They should have used acceleration corresponding to the pad response. And there's a disconnect there. And we go on.
When you look at this, the calculation for canister transfer building, they went to the dynamic analysis of canister transfer building, identified the structural response in terms of acceleration, multiplied by the mass, and obtained a load, which is correct. But when it comes to the cask and pad, for some reason that's not clear to me, they could have gone to Holtec and said, "What is the acceleration of the cask? What is the acceleration of the pad," a similar philosophy as canister transfer building, and estimated the load. Rather, they choose to use the design motion value, pga to get the load.
Q. What is pga?
A. Peak ground acceleration.

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Q. And does that correspond to a horizontal frequency and natural frequency?
A. It has nothing to do with any structural response. It is one number in the design motion.
Q. And it could be corresponding to the response at any of a number of frequencies, then?
A. No. It represents the response at very high frequency.
Q. Okay.
A. Which is the smallest number on the curve.
Q. Okay. So that I finally understand what you are saying, what you are saying is that in their analysis, Stone & Webster picked essentially a ground motion acceleration that corresponded to very high frequency, natural frequency, if you will. Whereas they should have moved further down in the curve --
A. They should have used an acceleration corresponding to the response of the pad.
Q. Okay. Now I understand. Thank you.
Go back for a moment with me to the -- did you review the Holtec calculations also from the viewpoint of determining whether they used the correct natural frequency in their analysis of the forces on the casks themselves?
A. One concern I have about that aspect of

CALCULATION COVER SHEET

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RECORD OF ISSUES

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

[X] Nuclear Quality Assurance Category [ ] Non-Nuclear Quality Assurance Category

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

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

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

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

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CALCULATION SHEET

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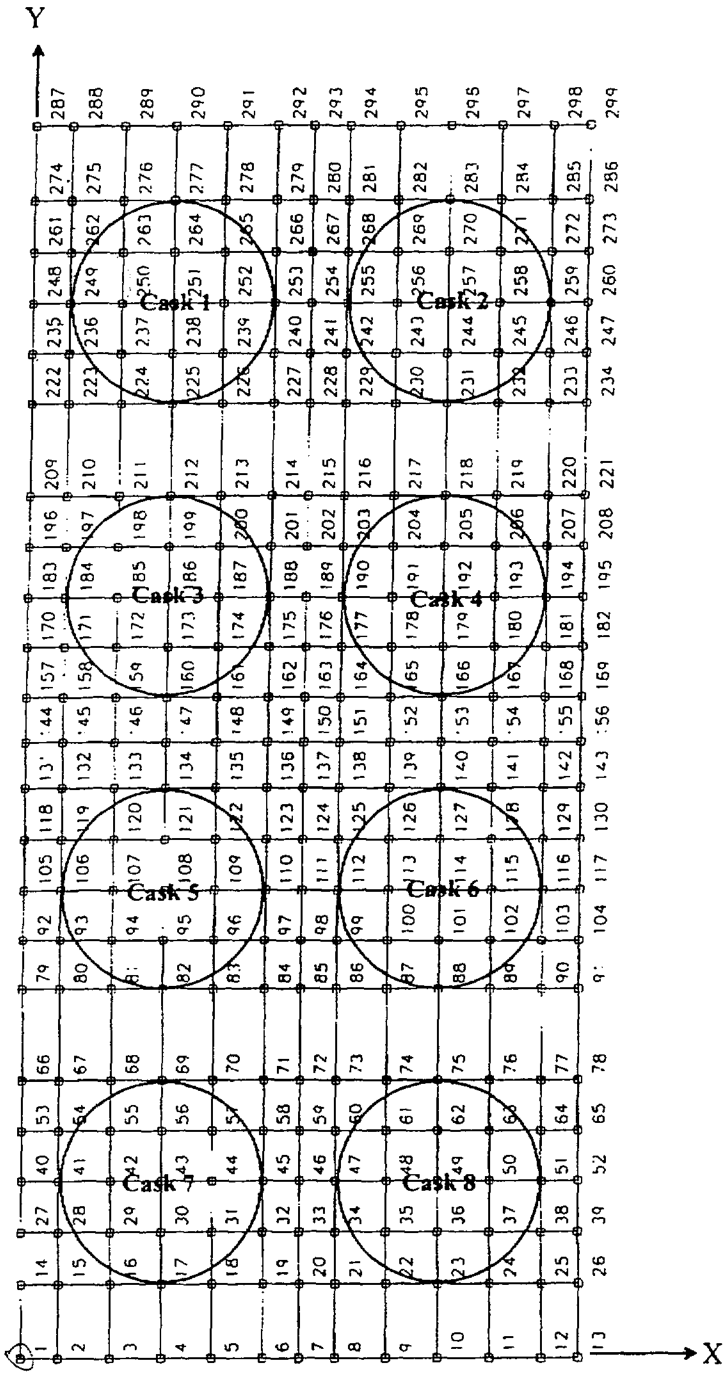

Figure 5.1-1 CECSAP Finite-Element Model with Node Numbers

5.2.5 COMPARISON OF CECSAP AND SASSI RESULTS

Results of the CECSAP and SASSI analyses, in terms of maximum displacements, maximum bending moments, and maximum shear force are shown and compared in Tables 5.2.5-1, 5.2.5-2, and 5.2.5-3 respectively. This comparison is performed for lower-bound, best-estimate, and upper-bound soil conditions as shown in the tables. The displacement time histories at selected nodes for SASSI and CECSAP are compared in Figs. 5.2.5-1 through 5.2.5-9 for lower-bound, best-estimate, and upper-bound soil conditions. Similarly, moment time histories for plate element 217 from SASSI and CECSAP are compared in Figs. 5.2.5-10 through 5.2.5-18. The printed input and output files for SASSI and CECSAP analyses are given in Attachment B.

The CECSAP dynamic models are the same as given in Section 5, except a single vertical force time history is applied at the second quadrant of the first cask (Node No. 249). Analyses are performed for the lower-bound, best-estimate, and upper-bound soil conditions.

The maximum displacements from CECSAP are consistent with the displacements from the SASSI. Maximum bending moments and maximum shear forces from CECSAP are consistently higher than the results from SASSI. Thus, the maximum bending moments and shear forces from CECSAP are used for the design of the pad.

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Table 5.2.5-1

Maximum Vertical Displacements (ft) at Selected Nodes

Selected Node No.	Lower-Bound Properties			Best-Estimate Properties			Upper-Bound Properties		
	SASSI (A)	CECSAP (B)	% Diff. [(B)/(A)-1]100	SASSI (A)	CECSAP (B)	% Diff. [(B)/(A)-1]100	SASSI (A)	CECSAP (B)	% Diff. [(B)/(A)-1]100
144	0.0067	0.0058	-14	0.0055	0.0027	-51	0.0043	0.0014	-67
157	0.0076	0.0069	-9	0.0061	0.0035	-43	0.0047	0.0018	-61
170	0.0086	0.0084	-2	0.0069	0.0046	-34	0.0052	0.0026	-50
183	0.0099	0.0101	2	0.0078	0.0059	-25	0.0057	0.0036	-37
196	0.0114	0.0120	5	0.009	0.0076	-16	0.0066	0.0049	-26
209	0.013	0.0141	8	0.0102	0.0094	-8	0.0077	0.0065	-16
222	0.0164	0.0180	10	0.013	0.0134	3	0.0095	0.0099	5
235	0.0182	0.0202	11	0.0142	0.0153	8	0.0106	0.0117	10
248	0.0195	0.0220	13	0.0152	0.0165	9	0.0113	0.0130	15
261	0.0201	0.0230	14	0.0152	0.0172	13	0.0111	0.0127	14
274	0.0203	0.0236	16	0.015	0.0173	15	0.0104	0.0125	21
287	0.0202	0.0242	20	0.0146	0.0182	25	0.0096	0.0119	24
288	0.0184	0.0279	52	0.0132	0.0162	22	0.0087	0.0103	18
289	0.0161	0.0184	14	0.0112	0.0131	17	0.0074	0.0083	12
290	0.0138	0.0155	12	0.0096	0.0109	13	0.0063	0.0062	-2
291	0.0116	0.0128	10	0.0082	0.0086	5	0.0052	0.0048	-8
292	0.0098	0.0120	23	0.0067	0.0069	4	0.0043	0.0034	-20
293	0.0083	0.0085	3	0.0057	0.0057	1	0.0038	0.0028	-25
294	0.0069	0.0070	1	0.0049	0.0047	-4	0.0031	0.0023	-26

Notes: The displacements obtained from CECSAP at nodes near application of load (the pad interfaced-forcing function) at Node 249, are about 10% higher than those obtained from SASSI. However, the displacements obtained from CECSAP at nodes away from application of the load, which have relatively smaller magnitude than those at nodes near the application of load, are somewhat lower than those obtained from SASSI. For location of nodes selected in this Table, see Fig. 5.1-1.

See Attachment B for SASSI and CECSAP comparison results.

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		CALC. NO.	G(PO17)-2	REV. NO.	3
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Table S-2
Maximum Vertical Displacements and Soil Bearing Pressures
Live Load

Node No.	(Z_l)max ($x10^{-2}$ ft.)							
	subgrade modulus = 2.75 kcf				subgrade modulus = 26.2 kcf			
	2 Casks	4 Casks	8 Casks	7 Casks + OLT	2 Casks	4 Casks	8 Casks	7 Casks + OLT
1	13.06	11.29	-50.97	-57.81	0.61	1.16	-4.83	-5.30
7	13.02	11.28	-50.97	-41.84	0.59	1.14	-4.84	-4.42
13	13.06	11.29	-50.97	-25.83	0.61	1.16	-4.83	-3.50
144	-11.82	-26.36	-52.73	-78.21	-0.70	-2.89	-5.78	-7.95
150	-11.93	-26.35	-52.71	-61.06	-0.76	-2.89	-5.79	-6.31
156	-11.82	-26.36	-52.71	-43.87	-0.70	-2.89	-5.78	-4.65
287	-42.54	-62.26	-50.97	-100.20	-5.13	-5.98	-4.83	-11.81
293	-42.59	-62.25	-50.97	-80.88	-5.16	-5.98	-4.84	-8.48
299	-42.54	-62.26	-50.97	-61.84	-5.13	-5.98	-4.83	-5.47
	Maximum Soil Bearing Pressure q_{zl}[(1)] (ksf)							
1	0	0	-1.402	-1.590	0	0	-1.264	-1.390
7	0	0	-1.402	-1.151	0	0	-1.267	-1.159
13	0	0	-1.402	-0.710	0	0	-1.264	-0.917
144	-0.325	-0.725	-1.450	-2.151	-0.185	-0.757	-1.514	-2.082
150	-0.328	-0.725	-1.450	-1.679	-0.199	-0.758	-1.516	-1.653
156	-0.325	-0.725	-1.450	-1.206	-0.185	-0.757	-1.514	-1.219
287	-1.170	-1.712	-1.402	-2.756	-1.345	-1.567	-1.264	-3.094
293	-1.171	-1.712	-1.402	-2.224	-1.352	-1.565	-1.267	-2.222
299	-1.170	-1.712	-1.402	-1.701	-1.345	-1.567	-1.264	-1.434

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