Paul J. Trudeau

Senior Lead Geotechnical Engineer

Years Experience (as of February 2002)

At Stone & Webster: 29 With other Firms: 0

Department/Division/Location

Civil and Transportation/Division 52/Boston

Professional History

Stone & Webster, Boston, Massachusetts – 1973 to Present
Massachusetts Institute of Technology – Cambridge, Massachusetts – 1971 to 1973
Stone & Webster, Boston, Massachusetts – 1971 to 1972
Worcester Polytechnic Institute, Worcester, Massachusetts – 1967 to 1971

Areas of Expertise

- Geotechnical Engineering and Design
- Use of Computers In Geotechnical Analyses and Designs
- Managing Geotechnical Investigations
- Geotechnical Instrumentation
- Performing Cross-Hole Shear Wave Velocity Surveys
- NRC Regulatory Compliance, Review, and Implementation for Nuclear Power Plants and Independent Spent Fuel Storage Installations

Awards

Desmond Fitzgerald Medal awarded by the Boston Society of Civil Engineers for "Shear Wave Velocity and Modulus of a Marine Clay," Journal of the Boston Society of Civil Engineers, January 1974.

Computer Hardware/Software Capabilities

Mr. Trudeau has considerable experience using PC and mainframe computer programs for performing geotechnical analyses. He is extremely proficient at developing spreadsheets using Microsoft Excel for solving complex engineering calculations and also is an expert FORTRAN programmer and in programming IBM JCL. He also has considerable experience in using MicroStation for generating report-quality sketches and figures and in using InRoads for plotting contours, subsurface profiles, and determining earthwork quantities.

He is adept at developing batch programs, as well as programming in dBASE, AWK, perl, and developing shell scripts in Unix. He routinely uses these techniques for automatic placement of graphics at correct locations and scales in MicroStation design files for generation of geotechnical figures, such as boring location plans, subsurface profiles, contour maps, and other figures for reports.

Department/Division Assignments

Geotechnical Division Computer Coordinator

Training

40 hours of instruction in Waste Site Worker Protection and 8 hours of instruction in Supervisory Training to comply with OSHA 1910.120(e)(2&3)

Experience Summary

Mr. Trudeau has over 29 years of experience in the engineering industry. Currently, as a Senior Lead Engineer in the Civil and Transportation Department of Stone & Webster, he is Lead Geotechnical Engineer on several Independent Spent Fuel Storage Facility projects. In prior years, as the Geotechnical Division Computer Coordinator, he was responsible for the development, documentation, and maintenance of more than 80 geotechnical computer programs sponsored by the Geotechnical Division of Stone & Webster and for providing consulting for geotechnical computer applications.

Since joining Stone & Webster in 1973, he has served as a Lead Geotechnical Engineer for Independent Spent Fuel Storage Installations (ISFSI) at the Duane Arnold Energy Center in Palo, Iowa, the Private Fuel Storage Facility in Skull Valley, UT, and at Maine Yankee's nuclear plant in Wiscasset, ME; for numerous combined-cycle power plants; and for the Bellefonte Nuclear Plant, Shoreham Nuclear Power Station, Falcon Seaboard Gas Pipeline, TVA Widows Creek Steam Plant, and various projects at the Hanscom Air Force Base. He has also served as a Geotechnical Engineer on several nuclear and fossil power plant projects. In these roles, he was responsible for performing geotechnical investigations, preparing geotechnical analyses, developing geotechnical design criteria for other disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical, and for preparing geotechnical sections of Preliminary and Final Safety Analysis Reports and Environmental Reports. This work was performed in accordance with quality assurance programs that satisfied the quality assurance requirements of Appendix B of 10CFR Part 50 and NQA-1.

He was also responsible for reviewing geotechnical analyses and reports prepared by others on these projects, and for preparing testimony and for testifying at depositions and public hearings. He has also completed 40 hours of instruction in Waste Site Worker Protection and 8 hours of instruction in Supervisory Training to comply with OSHA 1910.120(e)(2&3) and is certified to work on hazardous waste sites.

Mr. Trudeau's field experience includes performing cross-hole shear wave velocity tests in Maine, Connecticut, and Texas; geotechnical boring supervision at Jamesport, Shoreham, and Shoreham West on Long Island in New York and at Wards Island in New York, New York; and a compaction control investigation and intake canal revetment repair at Shoreham Unit No. 1. He has performed inspections of the haul road for transport of 300-ton steam generators at the North Anna Nuclear Power Station in Virginia and has inspected the route proposed for transport of the 800-ton reactor pressure vessel from the Shoreham Nuclear Power Station to Chem-Nuclear's disposal facility in Barnwell, South Carolina. In addition, he has served as Lead Scientist/Field Supervisor of environmental borings that were drilled for site assessment studies performed for New York City Department of Environmental Protection at their Jamaica, Wards Island, and 26th Ward water pollution control plants.

Mr. Trudeau's laboratory experience includes performing index property tests, consolidation tests, resonant column (Hardin Oscillator) tests, and static and dynamic triaxial tests. He was instrumental in selection, installation, testing, and debugging of Stone & Webster's geotechnical laboratory data acquisition system. His educational experience encompasses many aspects of civil engineering, including soil mechanics and foundations, computer programming (FORTRAN), soil dynamics, earthquake engineering, geotextiles, and structures.

Education

Master of Science in Civil Engineering, MIT, Cambridge, Massachusetts – 1973
B.S. in Civil Engineering, Worcester Polytechnic Institute, Worcester, Massachusetts – 1971

Licenses, Registrations, and Certifications

Professional Engineer: Massachusetts – 1977
Maine – 1999
Iowa – 2002

Professional Affiliations

Chi Epsilon: Member – 1969
American Society of Civil Engineers: Member 1971
Boston Society of Civil Engineers Section/ASCE: Member 1971
International Society of Soil Mechanics and Foundation Engineering: Member 1974

BSCES Director
BSCES Awards Committee – Chairman
BSCES Student Chapter Committee – Chairman
BSCES Membership Committee – Member
BSCES Task Force for Younger Members – Member
ASCE National Convention Attendance Committee – Co-Chairman
BSCES Geotechnical Engineering Practice Lecture Series Committee – Member

Publications

Trudeau, P.J., Whitman, R.V., and Christian, J.T., "Shear Wave Velocity and Modulus of a Marine Clay," Journal of the Boston Society of Civil Engineers, January 1974.

Pierce, D.S., and Trudeau, P.J., "Digital and Analog Methods for the Development of Stereoscopic Contour Maps for Geological and Geophysical Analysis," Geological Society of America Abstracts with Programs, Vol. 10, No. 7, 1978.

Experience History

STONE & WEBSTER, BOSTON, MASSACHUSETTS – 1973 TO PRESENT

Mixed Oxide (MOx) Fuel Fabrication Facility (Oct 2001 to Present)
U. S. Department of Energy

As Lead Geotechnical Engineer, responsible for geotechnical engineering efforts associated with the licensing and design of the $500M MOx Fuel Fabrication Facility (MFFF). Responsible for performing geotechnical investigations, preparing geotechnical analyses, developing geotechnical design criteria for other disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical, and for preparing geotechnical reports. This work was performed in accordance with quality assurance programs that satisfied the quality assurance requirements of Appendix B of 10CFR Part 50 and NQA-1.

The MFFF work is being done under a 3-year, $125M base contract for the U.S. Department of Energy. Options 1 and 2 of the contract will include construction management and operation of the MFFF, respectively. The facility will be based on the proven technology of the COGEMA Melox Plant in southern France. The facility will be licensed by the NRC.

Independent Spent Fuel Storage Installation – (March 2001 to Present)
Duane Arnold Energy Center, Palo, IA

As Lead Geotechnical Engineer, responsible for development of field programs and associated engineering services scopes of work for performing subsurface investigations required to document existing conditions for licensing and design of the foundations of the proposed Independent Spent Fuel Storage Installation (ISFSI), and to comply with US Nuclear Regulatory requirements. This effort included reviewing existing geotechnical data, development of the subsurface exploration plan, including boring programs, verticality survey, geophysical survey, and laboratory testing, comparison of bids, selection of the drilling, laboratory testing, and geophysical contractors, and drilling and sampling soil and rock at the site, review and reporting of the results of the laboratory testing of soils from the site, as well as the results of the cross-hole and down-hole seismic surveys that were performed at the site. Also developed the groundwater monitoring procedure and incorporated data collected in the geotechnical report. Responsible for development of the seismic design basis of the facility and for preparation of geotechnical calculations, design criteria, and geotechnical report.

Private Fuel Storage Facility – Skull Valley, UT (Dec 1997 to Present)
Private Fuel Storage, Limited Liability Corporation

As Lead Geotechnical Engineer for the Private Fuel Storage Facility – Skull Valley, responsible for preparation of responses to questions received from the Nuclear Regulatory Commission (NRC) and intervenors regarding geotechnical sections of the Safety Analysis Report and the Environmental Report. Participated in litigation, including developing responses to discovery requests and interrogatories from intervenors and providing depositions, as well as attending meetings and public hearings with the NRC and intervenors, responding to questions regarding geotechnical issues on this project. Recommended and developed proposal to use soil cement to stabilize the near-surface eolian silts to support the cask storage pads at the elevations required for flood protection and to provide enhanced stability against sliding due to the loads associated with the design basis ground motion. Developed additional field programs, including borings, cone penetration testing, and geophysical surveys, and associated engineering services scopes of work (ESSOWs). The cone penetration testing included standard tip and sleeve resistance measurements, resistivity measurements, as well as down-hole seismic shear and compression wave velocity tests and dilatometer tests. Responsible for review of the verification and validation of the software used to process the cone penetration test data. Participated in resolution of survey problems with respect to the locations of the borings performed at the site. Developed laboratory testing programs and associated engineering services scopes of work. Prepared comparisons of bids and participated in negotiations with bidders prior to award of contracts. Supervised execution of laboratory testing and prepared engineering calculations, incorporating results of these studies in developing responses to questions from the NRC and intervenors. Also updated the respective sections of calculations, the Environmental Report, Geotechnical Report, and the Safety Analysis Report to incorporate the change of the design basis ground motion from the original deterministic earthquake to the probabilistic seismic hazard analysis 2,000-yr return period earthquake. These ESSOWs, laboratory testing, and analyses were prepared in accordance with a quality assurance program that satisfied the quality assurance requirements of Appendix B of 10CFR Part 50.

Maine Yankee Decommissioning Project (Oct 1999 to Mar 2000)
Maine Yankee Atomic Power Company – Wiscasset, ME

As Lead Geotechnical Engineer, responsible for development of the subsurface investigation performed using geoprobes to ascertain the soil types of the near-surface soils and the depth to ground water and rock in areas proposed for temporary storage of rubblized concrete from the demolition of existing structures at the plant. This effort was required as part of the development of the solid waste storage permit required by the State of Maine Department of Environmental Protection. Prepared geotechnical sections of the solid waste storage permit application.

Independent Spent Fuel Storage Installation (Sept 1998 to June 2001)
Maine Yankee Atomic Power Company – Wiscasset, ME

As Lead Geotechnical Engineer for the Independent Spent Fuel Storage Installation for Maine Yankee Atomic Power Company in Wiscasset, ME, responsible for all geotechnical activities associated with permitting, licensing, design, and construction of the Independent Spent Fuel Storage Installation (ISFSI). This effort included reviewing existing geotechnical data, development of the subsurface exploration plan and boring and test pit ESSOWs, comparison of bids, selection of the drilling contractor, and drilling and sampling soil and rock at the site. Mr. Trudeau also prepared a project-specific procedure for performing a cross-hole shear wave velocity survey, and he performed that survey to obtain soil properties for dynamic analyses. Also responsible for preparation of the procedure for monitoring observation wells and reducing the data generated by that program and reporting it to the State of Maine Department of Environmental Protection as part of the Site Location of Development Permit process. Responsible for responding to nonconformity and disposition reports, preparation of specifications for construction, including earthwork and subdrain installations. Responsible for preparation of geotechnical calculations, including SHAKE analyses, slope stability analyses, and analyses of bearing capacity, settlement, and reduction of cross-hole velocity data. Also responsible for preparation of geotechnical design criteria and the geotechnical report.

Independent Spent Fuel Storage Installation (Sept 1999 to Feb 2000)
Indian Point Nuclear Power Station, Consolidated Edison, NY

Assisted the Lead Geotechnical Engineer in preparation of the boring location plan and ESSOW for the preliminary subsurface investigation for the proposed ISFSI.

Mystic, Edgar, and Medway Combined Cycle Power Plants (Mar 1998 to Dec 1998)
Sithe Energies, Inc

Geotechnical Engineer

Terminal A Area 8 (Mar 1998 to Oct 1998)
MASSPORT

Geotechnical Engineer, responsible for development of specifications and geotechnical support during construction.

Combined-Cycle Power Plant (Feb 1998 to Feb 2000)
EMI, Rumford, ME and Tiverton, RI

Lead Geotechnical Engineer, responsible for site investigations, development of design criteria and specifications, and providing geotechnical support during construction.

Santeetlah Dam (Dec 1997 & July/Aug 1998)
Tapoco Developments

Geotechnical Engineer and Computer Consultant

Cheoah Dam (Aug 1997 to Sept 1997)
Tapoco Developments

Geotechnical Engineer and Computer Consultant

Big Brown Steam Electric Station, Fairfield, TX (July 1997 to Nov 1998)
TU Electric Company

Geotechnical Engineer, responsible for site investigations, development of design criteria and specifications, and providing geotechnical support during construction.

Building 99 Fuel Oil Storage Facility (June 1997 to Aug 1997)
GE River Works Plant – Lynn, MA

Lead Geotechnical Engineer, responsible for site investigations, development of design criteria and specifications, and providing geotechnical support during construction.

VX Full Scale Plant (April 1997 to January 2000)
U.S. Army Program Manager for Chemical Demilitarization, Newport, IN

Lead Geotechnical Engineer, responsible for site investigations and geotechnical analyses.

Private Fuel Storage Facility – Skull Valley, UT (Jan 1997 to Oct 1997)
Private Fuel Storage, Limited Liability Corporation

Geotechnical Engineer

Building 66 G & L G60TX Foundation (Dec 1996 to Jan 1997)
GE River Works Plant – Lynn, MA

Lead Geotechnical Engineer, responsible for site investigations, development of design criteria and specifications, and providing geotechnical support during construction.

Calderwood Dam (Nov 1996 to Feb 1997)
Tapoco Developments

Geotechnical Engineer and Computer Consultant

9th St Substation (Oct 1996 to Jan 1998)
Potomac Electric Power Co, Washington, D. C.
Geotechnical Engineer

Boston Ramps (Feb 1996 to Dec 1996)
Massachusetts Turnpike Authority
Geotechnical Engineer

Goodhue County Independent Spent Fuel Storage Installation (Dec 1995 to Sept 1996)
Northern States Power Company
Geotechnical Engineer

Central Artery/Third Harbor Tunnel Project (Feb 1994 to January 1997)
Mass. Department of Public Works
Manager of Computer Services for Area 5 Geotechnical Consultant

Granite State Gas Transmission Company (Nov 1993)
Computer Consultant & Database Manager

Bellefonte Nuclear Plant (Oct 1993 to Mar 1994)
Tennessee Valley Authority
Lead Geotechnical Engineer

Chubb & Son, Incorporated (Sept 1993 to Jan 1994)
Geotechnical Consultant

Pease Air Force Base (Aug 1993)
United States Air Force
Geotechnical Engineer

Petersburg Generating Station (July 1993 to Sept 1993)
Indianapolis Power and Light Company

Green Mountain Power Corporation (July 1993)
Geotechnical Engineer, responsible for site investigations.

E. W. Stout Generating Station (July 1993)
Indianapolis Power and Light Company
Geotechnical Engineer and Computer Consultant

Hanscom Air Force Base (Apr 1993 to July 1993)
United States Air Force
Lead Geotechnical Engineer, responsible for site investigations and development of design criteria and the geotechnical report.

Portland Natural Gas Transmission System (Nov 1992 to Apr 1993)
Computer Consultant & Database Manager

Maine Low-Level Radioactive Waste Authority (Oct 1992 to May 1993)
Geotechnical Engineer

Afobaka Dam (Oct 1992 to Jan 1993)
Suriname Aluminum Company
Geotechnical Engineer and Computer Consultant

Widows Creek (Sept 1992 to Feb 1993)
Tennessee Valley Authority
Lead Geotechnical Engineer

General Support Services Contract, Richland Field Office (Sept 1992 to Oct 1992)
U. S. Department of Energy
Geotechnical Engineer

Patriot Generating Station (June 1992 to Aug 1992)
Indianapolis Power and Light Company
Geotechnical Engineer and Computer Consultant

Bellefonte Nuclear Plant (Feb 1992 to July 1992)
Tennessee Valley Authority
Lead Geotechnical Engineer

Petersburg Generating Station (Sept 1991 to May 1992)
Indianapolis Power and Light Company
Geotechnical Engineer and Computer Consultant

North Anna Nuclear Power Station (Sept 1991)
Virginia Power Company
Geotechnical Engineer

EG & G Rocky Flats (Sept 1991)
US Department of Energy
Geotechnical Engineer (SHAKE Analyses)

New Production Reactor (Feb 1991 to Oct 1991)
US Department of Energy
Geotechnical Engineer and Computer Consultant

Widows Creek Steam Plant – Unit 8 (Feb 1991 to June 1991)
Tennessee Valley Authority
Lead Geotechnical Engineer

Hanscom Air Force Base (Jan 1991 to Feb 1991)
United States Air Force
Lead Geotechnical Engineer

Central Artery/Third Harbor Tunnel Project (Mar 1990 to Feb 1992)
Mass. Department of Public Works
Manager of Computer Services for Area 5 Geotechnical Consultant

Hanscom Air Force Base (Jan 1990)
United States Air Force
Lead Geotechnical Engineer

Sludge Management Project (Sept 1989 to July 1990)
New York City Department of Environmental Protection
Geotechnical Engineer / Geotechnical Field Inspector / Lead Scientist/Field Supervisor

Plattsburgh 12 In. Diameter Gas Pipeline (Feb 1989 to Apr 1990)
Falcon Seaboard Pipeline Company
Lead Geotechnical Engineer

Great Northern Paper Company (Feb 1989 to May 1989)
Geotechnical Engineer

Salt Cave Hydroelectric Project (Apr 1986 to May 1986)
City of Klamath Falls, Oregon
Geotechnical Engineer

Bradley Lake Project (Feb 1986 to Oct 1986)
Alaska Power Authority
Geotechnical Engineer

Beaver Valley Power Station – Unit 2 (Oct 1984 to Aug 1985)
Duquesne Light Company
Geotechnical Engineer

Shoreham Nuclear Power Station – Unit No. 1 (Jan 1983 to Mar 1992)
Long Island Lighting Company
Lead Geotechnical Engineer

Malakoff Site (Apr 1982 to Dec 1982)
Houston Lighting & Power Company
Geotechnical Engineer

Office of Nuclear Waste Isolation (ONWI) of Battelle Memorial Institute (Jan 1982 to Oct 1987)
U.S. Department of Energy
Geotechnical Computer Consultant

Western Fuels Association. Inc. (Dec 1980)
Geotechnical Computer Consultant

Patriot Station (Nov 1980 to July 1981)
Indiana Power and Light Company
Geotechnical Computer Consultant

Site X (Oct 1980 to Dec 1981)
Houston Lighting & Power Company
Geotechnical Engineer

Pumped Storage Project (Apr 1980 to July 1980)
Public Service Company of New Mexico
Geotechnical Computer Consultant

Beaver Valley Power Station – Unit No. 2 (Feb 1980 to Mar 1980)
Duquesne Light Company
Geotechnical Computer Consultant

Millstone Unit No. 3 (Feb 1980)
Northeast Utilities Service Company
Geotechnical Engineer

Martin Cooling Dike (Jan 1980)
Florida Power and Light Company
Geotechnical Engineer

Beaver Valley Power Station – Unit No. 1 (Mar 1979 to May 1979)
Duquesne Light Company
Geotechnical Computer Consultant

Haven Nuclear Power Station (Dec 1978 to Jan 1979)
Wisconsin Electric Power Company
Geotechnical Engineer

Office of Nuclear Waste Isolation (ONWI) of Battelle Memorial Institute (Sept 1978 to Nov 1979)
U.S. Department of Energy
Geotechnical Computer Consultant

Stuyvesant & New Haven Sites (Apr 1978 to Sept 1978)
New York State Electric and Gas Corp.
Geotechnical Computer Consultant

Sundesert 500 kV Transmission and Substation Project (Aug 1977 to Dec 1977)
San Diego Gas and Electric Company
Geotechnical Computer Consultant

Shoreham Nuclear Power Station (Oct 1973 to June 1976)
Long Island Lighting Company
Geotechnical Engineer

Jamesport Nuclear Power Station (Aug 1973 to Apr 1977)
Long Island Lighting Company
Geotechnical Engineer

Northfield Mountain Pumped Storage Project (Aug 1973 to Oct 1973)
Northeast Utilities Service Company
Geotechnical Engineer and Computer Consultant

Geotechnical Division Computer Coordinator (Mar 1973 to Jan 1999)

North Anna Power Station (Feb 1973)
Virginia Electric and Power Company
Geotechnical Engineer

Massachusetts Institute of Technology – Cambridge, Massachusetts – 1971 to 1973
Graduate Research Assistant

MR. TRAVIESO-DIAZ: I would like at this time also to introduce three exhibits to Mr. Trudeau's testimony, which are in the book, Exhibits UU, VV, and WW. They are as follows. And copies of these exhibits are in the book that the court reporter has. Of course, new Exhibit VV is also in that book.

Exhibit UU is Stone & Webster calculation numbers 05996.02-G(B)-04, Revision 9, entitled "Stability Analyses of Cask Storage Pads" dated July 26, 2001.

The second exhibit is Exhibit VV, which is identified as Stone & Webster calculation number 05996.02-G(13)-B, Revision 6, entitled "Stability Analyses of Canister Transfer Building" dated July 26, 2001.

And the third exhibit, Exhibit WW, is a one-page document with a long name. It's entitled "Sliding Stability--Dynamic Loads from Holtec for 2,000-Year Earthquake for Pad Loaded with 8 Casks, mu equal to 0.8, and Best-Estimate Soil Properties, c equal to 2.1 ksf and phi equals 0 at the Base of Concrete Pad, Includes Dynamic Active, but No Passive Pressure." I will move that these three exhibits be admitted into evidence.

JUDGE FARRAR: Any objection?

MS. CHANCELLOR: No objection, your Honor.

JUDGE FARRAR: Mr. O'Neill?

MR. O'NEILL: No objections.

JUDGE FARRAR: Then those will be admitted.

(APPLICANT EXHIBITS-UU, VV, AND WW MARKED AND ADMITTED.)

MR. TRAVIESO-DIAZ: Mr. Chairman, Mr. Trudeau is available for cross-examination.

JUDGE FARRAR: Mr. O'Neill?

CROSS-EXAMINATION

BY MR. O'NEILL:

Q. Good afternoon, Mr. Trudeau.

A. Good afternoon.

Q. My name is Martin O'Neill. As you know, I'm co-counsel for the NRC Staff. I'll ask you a few questions today.

Actually, before I begin, your Honors, I just want to say I will be modifying the sequence of my questioning a little bit as presented in the --

JUDGE FARRAR: Cross-examination.

Q. -- cross-examination plans. Still avoiding confusion on your part.

Given the large number of issues, the issues that have been raised in part D of this contention and the fact that the Applicant has addressed these issues in four different pieces of testimony, I'd like to ask you to just briefly restate or summarize the focus of your testimony on part D of the contention.

MR. TRAVIESO-DIAZ: Excuse me, Counsel. Did you mean part B?

MR. O'NEILL: I said part D.

MR. TRAVIESO-DIAZ: I'm sorry. I understood part B.

JUDGE FARRAR: D as in dog.

Q. (By Mr. O'Neill) Feel free to reference your prefiled testimony.

A. It does state it pretty clearly, I thought. I'm a geotechnical engineer working for Private Fuel Storage, so I'll be responding to the geotechnical issues associated with part D. These will include questions about the stability analyses of the cask storage pads and the canister transfer building, primarily.

Q. Thank you. Throughout your testimony

you discuss shear strength of soils. Will you please explain or define this term, what you mean by shear strength of soils?

A. Yes. The strength of soils is comprised of essentially two main components. One of the components is the frictional part of the strength, and the other component is the cohesive portion of the strength. The sliding stability of our canister transfer building, for instance, and the cask storage pads relies primarily on the cohesive portion of the strength of the silty clay, clayey silt layers which are the upper Bonneville -- upper Lake Bonneville sediments.

Q. I know this relates to some extent to part C of the contention. Could you just briefly mention a few methods by which shear strength is determined, just very generally?

A. The shear strength is measured in the field by performing borings, standard penetration testing in borings. And also we have determined shear strengths in the field, relatively speaking, using the cone penetration testing technique. In the laboratory we've conducted unconsolidated, undrained triaxial tests as well as consolidated, undrained triaxial tests as well as cyclic triaxial tests and direct shear testing, all to determine various aspects of the shear strength of the soil.

Q. At this point I'd like to turn your attention to your response to question 16 that is located on page 6 through 8 of your testimony. In your response to question 16 you discuss factors of safety against bearing capacity failure and sliding failure for both the storage pads and the CTB, correct?

A. Correct.

Q. And I know on page 7 -- will you turn to page 7, please. There's a paragraph labeled "For the pads (bearing capacity failure)." Do you see where that is?

A. Yes, I do.

Q. In discussing the factor of safety against bearing capacity failure of the storage pads, you indicate that factor of safety was 2.1 and that this was calculated using the static shear strength value of 2,200 psf, correct?

A. That's correct.

Q. I want to ask, how is that value obtained via 2,200?

A. That was the lowest strength we measured in the UU tests that were performed back in 1996, I believe. Subsequent consolidated undrained triaxial testing confirmed that that was the lowest value that we obtained in all of the -- well, I'm pretty sure all of the testing that we did.

Q. So in your opinion, this particular value represents a very conservative estimate, you know, applicable shear strength of the soils for evaluating bearing capacity, correct?

A. Yes.

MS. CHANCELLOR: Your Honor, I'd like to place an objection on the record. Mr. O'Neill is getting into the area of soils, which is part C of Mr. Trudeau's testimony. Whether or not the UU tests have a shear strength of 2,200 psf I think is an issue that we will get into in C, and I'd object to this line of questioning.

MR. O'NEILL: Excuse me, your Honor.

JUDGE FARRAR: Go ahead.

MR. O'NEILL: I would disagree. I realize there's overlap. As you've emphasized repeatedly, we're dealing with very interwoven issues. And this specific testimony discusses soil shear strength values, and it is intended to reflect what the applicant views as being conservative estimates, and it's fundamental to these particular analyses.

MS. CHANCELLOR: Your Honor, if I may. This is not part of Exhibit -- the calculation UU. This answer 16 relates to some simple calculations that Mr. Trudeau performed by increasing strengths obtained in static strength by 50 percent. Whether or not these tests come from any samples at the site is not in his testimony, and it certainly is -- they are not the values that he used in calculation G(B)-04, Exhibit UU.

MR. TRAVIESO-DIAZ: I think that both the testimony of Mr. Trudeau, testimony of the Staff, and, as we will see, the testimony of State on this aspect of section D of contention Utah QQ brings up numerous references to the testimony of section C only because it's inevitable that there will be some overlap. To me the question is one of degree. To the extent that you need to get into section C to establish points that are important to section D, I think it is okay. What I don't think will be good is to try entire section C here now and then try it again in a few weeks when we go to section C.

So I raise an objection only because I think that there may be some amount of examination that is necessary, and quite frankly, I may do the same when my turn comes for other witnesses. But I would think that you have to draw the line someplace so that this doesn't become a long exercise in trying twice the same contention, the same essential facts.

JUDGE FARRAR: Vicky, would you read the question back, please?

(Requested portion of the record was read.)

JUDGE FARRAR: We're going to overrule the objection consistent with our prior written opinions and rulings in this case that these things are so, these subjects are so interrelated, it's hard to draw these lines. But I think counsel for the Applicant had a good admonition that we are going to try issue C at some point, and that while we'll allow you to poke at it a little bit, let's make sure we don't try the case here.

MR. TURK: Your Honor, may I make an observation also? It's the Applicant who put it in their testimony on part D. It appears on the face of the testimony. Mr. O'Neill's cross-examination on it will be very short, but I think there was a problem perhaps in the way the contention was structured, primarily by agreement of the Applicant and the State. If this was not meant to be part of D, then realistically it should not have been put in this piece of testimony, it should have been put in the Applicant's testimony on C. But it's here, so --

JUDGE FARRAR: It's here; and as we indicated I think on Monday, we can waste more time arguing about these lines. But just so everyone's conscious of that.

Go ahead. You may answer, Mr. Trudeau, if you remember the question.

THE WITNESS: Yes. That's the lower bound strength for bearing capacity for the pads.

Q. (By Mr. O'Neill) Lower bound strength?

A. Correct.

Q. And I guess that was my question, why do you consider it lower bound strength?

A. Because it was the lowest strength we measured in the laboratory testing program from the unconsolidated, undrained triaxial tests, and you can see these results in Exhibit UU, page C3.

Q. Just to move things along here. Mr. Trudeau, in response to question 14, your testimony, you refer to the "well-known phenomenon" in which clayey soils exhibit increases in shear strength when subjected to rapid loadings.

A. Correct.

Q. In what regard is this a well-known phenomenon? Has it been proved through experimental results?

A. Yes.

Q. Could you name a few examples?

A. The seminal paper on it is Casa Grande and Shannon from 1948. But similar results have been recorded in a paper by Shimming in 1966, and current geotechnical references by Das in 1993, Principles of Soil Dynamics, repeats this information.

Q. I notice in your testimony as well you distinguish between dynamic shear strength and static shear strength, correct?

A. Correct.

Q. When you refer to dynamic shear strength, are you referring to this phenomenon, shear strength that results from rapid loadings?

A. Yes. This is, I'd like to add, well known to the State as well. Their expert witness, Steven Bartlett, acknowledged that this is a typical -- I mean, he would have, if I'm correct in his deposition of November 2000, he indicated that he felt that a 30 percent increase in the static strength was reasonable without any additional testing. But the literature that I've cited indicates that strength increases on the order 50 to 100 percent have been noted for clayey soils.

Q. I want to direct your attention again to your response to question 16, pages 7 and 8, in particular your discussion, the bearing capacity failure issued for the CTB. I point you to the top of page 8.

A. Yes, I see it.

Q. You indicate that the soil shear strength of 3,180 psf is adjusted from a value of 2,200 for these soils based on the CPT results, correct?

A. That is correct.

Q. Could you elaborate a little bit more, explain why and how you made that adjustment?

A. As I indicated earlier, we have performed borings at the site, and in these borings we've noticed that the relative strength of the soil is lowest in this silty clay, clayey silt layer near the surface of the site. Deeper in the profile these standard penetration test blow counts are higher.

Subsequent cone penetration testing exhibited the same behavior, so that for the bearing capacity analysis of that great, big map for the canister transfer building, rather than just use the lower bound strength of that silty clay, clayey silt layer that's near the surface about five to twelve feet down, we felt it was more appropriate to take some credit for the increase in strength that we saw deeper in the profile. Because the bearing capacity failure of this large mat foundation will involve the soils bound to a depth at least 250 feet.

Q. And what depth did you consider here?

A. This only included the top 30 feet in assessing the strength of the soils used in the bearing capacity analysis of the canister transfer building. The underlying soils are very dense sands and gravels which are underlain by hard silts that would have even higher factors of safety from a bearing capacity point of view. And those relative increases in strength were just conservatively ignored in this analysis.

Q. So under I guess the bearing capacity theory, you consider a depth in the soil profile that's equivalent to -- is it the width of the foundation?

A. That is correct.

Q. But you did not do this in the case of the storage pads, correct?

A. We did not increase the strength of the silty clay above the 2,200 psf, no, the top 30 feet. We could have done that, yes.

Q. You would have seen --

A. An increase as well.

Q. Thank you. In answer 16 on page 7 you make reference to the 40-40-100 rule. We'll give you a moment.

A. Yes.

Q. Would you explain the meaning and significance of this rule?

A. For earthquake analyses, standard procedures permit the three components of an earthquake to be combined using this 100-40-40 rule. And by this rule you would take one of the components at full strength and the other two, the vertical and the other horizontal, we're talking about one of the horizontals being the 100 percent. The other two would be input at 40 percent.

And you would look at a variety of cases where that 100 percent cycled from the first horizontal, then the orthogonal horizontal and then the vertical. So you're looking at all combinations of this 100-40-40. And the reason this is done is because it's known that the peak acceleration or the peak velocity of an earthquake does not occur at the same time in all three components of a record. So standards have been developed that permit us to analyze using this 100-40-40 rule.

Q. When you refer to standards, what do you have in mind specifically?

A. The main one would be ASCE 4.

Q. In answer 16, just want to make sure I'm clear on something. With respect to soil cement in the CTB, you do take credit for the passive resistance of the soil cement placed around the building?

A. Yes, we do.

Q. And in the case of the storage pads you don't do this, right?

A. That is correct. Although there are some "what-if" type scenarios analyzed in the cask storage pad analyses, the base case does not assume any credit for the soil cement adjacent to the pads.

Q. Would it be possible for you to describe in greater detail perhaps the amount of conservatism that results from not taking credit per se for the strength of the soils around the pads?

A. Yes, I believe I have that in the calculation. If you give me a minute, I'll take a look.

This is in Exhibit UU, and it's addressed starting on page 29. On page 30 near the middle of the page you can see there's an analysis of the factor of safety for the pad to clayey soil in a north-south direction without passive resistance where the factor of safety is 1.52. And just above that there is an analysis for the pad on clayey soil in a north-south direction with the passive resistance included. And you can see that the factor of safety has increased from 1.52 to 2.35.

I might also note that in the east-west direction there was more passive resistance, so as shown on the bottom of page 30, the factor of safety again sliding in the east-west direction is 3.3.

Q. I want to now refer you to answer 19, page 11. You refer -- for the analysis of a row of 20 pads you refer to a hypothetical case and the extremely conservative assumptions that were involved. I believe the specific language is "a hypothetical case based on extremely conservative assumptions." Could you describe that hypothetical case?

A. This is the last paragraph on --

Q. Yes, the very top of page 11.

A. Okay. Yes. That hypothetical case was put into this calc to address the possibility or potential that there might be some cohesionless soils at some depth below the silty clay layer that is underneath the canister storage pads. So we assigned a coefficient -- I mean, a friction angle that would be conservative for those soils and assumed that those soils were directly beneath the pad to simplify the calculation.

Now, for earthquake accelerations as high as we have here at Private Fuel, frictional materials won't provide sufficient resistance to sliding to get a factor of safety of 1.1. So in this analysis we used a method proposed by Newmark for estimating the amount of displacement that might occur should such soils actually exist at the base of the pad, and we determined that the amount of sliding would be on the order of two inches. And because there's no safety-related connections between the cask storage pads and the yard area or any other buildings or anything, these typical connections for a nuclear power plant would include buried piping systems or electrical systems that were required for safe shutdown, that sort of thing. There are none of those kinds of facilities connected to these pads. Therefore, sliding on the order of a couple of inches would be of no consequence to these pads if this hypothetical case really did exist.

Q. It's my understanding, yes, that you actually did perform some calculations with respect to sliding and you had reached a conclusion that it might actually enhance resistance seismic response? Excuse me. Strike that.

Would actually enhance the resistance of the cask to motion?

A. We also did a similar calculation for the pads founded on a silty clay layer, which we do have out there, in which we ignored completely the cohesive characteristics of that silty clay. And in that analysis, which was done in a similar manner to here, we had only the frictional component of the strength available, and for those silty clays we conservatively assigned a value of friction angle of 17 degrees which, based on a conservative table in DM7, naval facilities design manual 7, I think it's .1, indicates that 17 degrees is an obviously conservative number for the friction angle for low plasticity clays.

So based on that even lower friction angle, we obviously would have a situation where sliding would be possible. So we repeated the Newmark analysis for that case as well, and the displacements or the calculated displacements for that case range from two to six inches, as I recall.

Now, we asked -- we also had Holtec do a similar sort of analysis for that same friction angle of 17 degrees, and they did a far more rigorous analysis than this simplified procedure by Newmark that we did, and they used the actual time history of accelerations from Geomatrix for the design earthquake. And they essentially integrated the accelerations twice to determine a displacement of these pads for every time that the acceleration values exceeded the amount that would give us a factor of safety against sliding of 1. And their analysis indicated that the displacements of the pad were on the order of three inches, I believe. 3.4 I think is the correct number. So their analysis, far more rigorous than ours, corroborated the Newmark's method, simplified method that we used here in this calculation.

Q. You distinguished between the frictional component of the shear strength and the cohesive portion. How would you relate in terms of their contribution to the actual shear strength of the soil? Does the cohesive portion provide more and account for more of --

A. For these stiff, unsaturated clays, the cohesive portion of the strength is fair greater than the frictional portion of the strength. When the upward component of the earthquake -- the vertical component of the earthquake acts in the upward direction. Because when it acts upward, it reduces the normal force available and the frictional resistance is calculated as the normal force times the tangent of the friction angle.

Q. I know you indicated in your testimony that PFS is committed to performing -- committed to demonstrate by testing shear strengths the various interfaces.

A. Yes, that's correct.

Q. And that's intended to demonstrate that your design provides resistance to sliding?

A. Yes, that we will indeed have the cohesion that we're relying on for our base case that gives us a factor of safety of 1.27, I believe it is, for the pads, ignoring the passive resistance of the soil cement.

Q. At the same time, though, you have concluded that the amount of sliding or displacement predicted would constitute a safety hazard per se, correct?

A. That is correct. So even if we don't get a cohesive strength and we only have an obviously consumed value of the frictional portion of that strength, the sliding is expected to be on the order of three inches based on -- 3.4 inches, I believe it was, based on Holtec's rigorous analysis of this, and on the order of two to six inches based on the simplified method.

I forgot to mention, but Holtec's analysis also checked the sliding of the casks on top of the pads, and they've determined that having the pads slide under the casks actually minimizes the excursion of the cask on top of the pad. So it's actually beneficial should that occasion happen.

Q. Are you aware of Staff's positions with respect to the nonexistence of the safety hazard, the Staff's conclusion that if sliding would occur there would be no safety hazard? And the second issue you mentioned, increasing the stability of the cask as a result of that sliding?

A. Yes, I've seen them in the SER.

Q. In the SER, okay. And the Staff agrees with those positions, correct?

A. That's my understanding from the way I read that.

MR. O'NEILL: Thank you, Mr. Trudeau. I have no further questions at this time.

JUDGE FARRAR: Thank you, Mr. O'Neill. Before we start the State's cross, Mr. Trudeau, you were here yesterday for the tutorial?

THE WITNESS: Yes.

JUDGE FARRAR: Can you explain in simple language -- you've mentioned both in your testimony today and your written testimony the work of the other organizations. Can you give us just a thumbnail sketch of how your work interrelates with the previous two panels of witnesses, just how you go about your day-to-day activity or your long-term analyses? How does that all fit together?

THE WITNESS: I'm responsible for the geotechnical aspects of the work. That included the boring programs that obtained the samples that were tested in the laboratory. And those results were provided to Geomatrix, who developed the strain compatible properties that Holtec used in their analyses of the pad and cask system and that our structural dynamics people used for the canister transfer building analyses.

We were also responsible for collecting the cone penetration test data that Bob Youngs referred to the other day as being indicative of extremely low variability in the soil properties out at the pad emplacement area.

Does that address what you were trying to get at?

JUDGE FARRAR: Sort of. Do you hire any of these other people, or are they hired by PFS?

THE WITNESS: PFS hired those people, Geomatrix, you mean, and Holtec and CEC.

JUDGE FARRAR: But you're the one

responsible for pulling it all together?

THE WITNESS: Pulling together the geotechnical data, to produce a geotechnical design criteria. That includes the soil properties that we've measured both in the laboratory and in the field and interpreted to develop other parameters required for the pad design, for instance. The design criteria were also provided to Wen Tseng for his people to use in the design of the pad.

JUDGE FARRAR: Ms. Chancellor, you're doing this cross?

MS. CHANCELLOR: Yes, I am, your Honor.

CROSS-EXAMINATION

BY MS. CHANCELLOR:

Q. Good afternoon, Mr. Trudeau. As you know, I'm Denise Chancellor representing the State of Utah.

A. Good afternoon, Denise.

Q. Have you changed employers? You said that you were working for PFS. Are you still with Stone & Webster?

A. I am still with Stone & Webster. But I see PFS everywhere in here where my work is, so I assumed that that was okay.

Q. I just wanted to clarify. And I noticed from your calculation that you've been on the PFS project at least from February of 1997?

A. Yes.

Q. Earlier than that?

A. May I take a look at my resume here? It's been a long time. That's about the right time period, but --

Q. I just noticed the calculation had -- Exhibit UU has a date of February of 1997. So I assumed that you were working on the PFS project when you did the calculation. Right?

A. Oh, yes. I may have been on the project a little earlier than that, but that's what I was trying to check.

Q. And you're the lead geotechnical engineer for the project?

A. Yes, I am.

Q. And you are responsible for the stability, sliding and overturning analysis of the storage pads, correct?

A. Correct.

Q. And that's PFS Exhibit UU?

A. Yes.

Q. And you were the preparer of revision zero, the originator of this calculation, correct? In February of 1997?

A. Co-preparer, yes.

Q. Co-preparer. And of the current revision, revision 9 dated July 26, 2001?

A. That's correct.

Q. And your name appears as a preparer or reviewer in all the revisions except revision 4 and 7; is that correct?

A. Yes, it is.

Q. And you're also responsible for the stability, sliding, and overturning analysis of the CTB, G(B)-13, PFS Exhibit VV, correct?

A. That's correct.

Q. And the current revision is dated July 26, 2000?

A. Yes.

Q. And you prepared revision 6, correct?

A. Yes.

Q. And you prepared revisions 2 and 3 of the CTB calculation, correct?

A. Correct.

Q. But not revisions -- not the original calculation or revisions 1, 4, and 5, correct?

A. Correct.

Q. With respect to both the storage pad and CTB calculations, were all of these revisions done under your direction or supervision?

A. Yes, they were.

Q. Have you been involved in any projects that include dynamic conditions with peak ground accelerations equal to or exceeding those at the PFS site for the 2,000-year return period event?

A. Only the PFS.

Q. Are you familiar with the term "ground response analysis"?

A. Yes.

Q. Could you define that term, please?

A. That's -- I'm assuming you mean response spectrum type analysis?

Q. Modeling of the ground response with a model like SHAKE?

A. Yes. SHAKE is a one-dimensional amplification model of the soil.

Q. And "SHAKE" is in all caps, correct?

A. Yes.

Q. Have you personally performed ground response analyses?

A. Yes, I have.

Q. Are you familiar with the term "soil-structure interaction" as it is used in geotechnical earthquake engineering?

A. Yes.

Q. Can you define that term, to the best of your understanding?

A. Soil-structure interaction analyses are performed to assess the response of structures supported on soils to seismic input.

Q. And are there various components of soil-structure interaction?

A. Components meaning like generating --

Q. Like kinematic -- inertial. Kinematic and inertial?

A. You're getting into an area where I would rely on our structural dynamics people.

Q. Geotechnical engineering is a branch of civil engineering that has to do with subsurface investigations and analysis that support foundation design, correct?

A. Correct.

Q. Isn't it true that you consider your experience to be in soils analysis such as bearing capacity analysis, sediment analysis, etc.?

A. Yes.

Q. Isn't it true that most of your work has involved facilities in the eastern United States?

A. Yes, I believe that's true.

Q. And at these facilities ground motions probably being somewhere in the order of, what, 3 g -- .3 g?

A. Yes.

Q. And that soil-structure interaction is not part of your geotechnical discipline. Is this correct?

A. I do not perform soil-structure interaction analyses, no.

Q. We were talking about factor of safety. Could you define factors of safety generically as it's used by engineering -- in the engineering profession?

A. A factor of safety is commonly defined as the resistance divided by the driving forces, for instance. For a sliding analysis, that would be a definition of the factor of safety.

Q. What is the generally accepted factor of safety for sliding, overturning, and bearing capacity accepted in the engineering profession?

A. For static loads?

Q. The dynamic loads.

A. For earthquake loads plus dead loads it would be 1.1.

Q. Are these same 1.1 factors of safety also recognized by NRC?

A. Yes.

Q. What is the factor of safety against sliding to the storage pads for the design-basis earthquake as presented in G(B)-04-9, PFS Exhibit UU?

A. As shown on page 23 near the top, it's conservatively calculated as 1.27 without including the passive resistance of the soil cement adjacent to those pads.

Q. And you refer to this. Do you call it the design-basis case or the base case?

A. That's the base case. That's what we believe is the minimum factor of safety against sliding of the pads out there on the site.

Q. And what do you mean by the base case?

A. That's the case that we consider the design. We believe that those are the proper lower bound strengths that were used in that analysis rather than the hypothetical cases that we also analyzed in this calculation.

Q. What is the factor of safety against bearing capacity failure for the pads for the design-basis earthquake as presented in G(B) 04-9, bearing capacity? Look on page 59, you might find it.

A. Did you say page 69?

Q. 59, five-nine.

A. Page 59 has what I would say is a conservatively low value of the factor of safety for bearing capacity. It equals 1.17 calculated based on 100 percent of the horizontal forces due to the earthquake acting in both directions at the same time.

JUDGE FARRAR: Ms. Chancellor, let me interrupt just for a second and make sure the record's clear. When you say it's safety factor 1.1, converting that to layman's language, do you mean a 10 percent margin or a 110 percent margin?

THE WITNESS: That's a 10 percent margin.

MS. CHANCELLOR: That was my next question, your Honor.

THE WITNESS: He steals all the easy ones -- good ones, doesn't he?

JUDGE FARRAR: You caught yourself just in time.

THE WITNESS: Sorry.

Q. (By Ms. Chancellor) Have you performed a dynamic stability analysis for a 10,000-year return period event for the storage pads?

A. I have reported the results of some additional analyses in my testimony in that regard showing the additional margin available for that earthquake.

Q. Do you consider in your testimony that that information is a dynamic stability analysis?

A. The dynamic bearing capacity analysis? Is that what you mean?

Q. Well, is it similar to the analysis that you did in Exhibit UU, for example?

A. It's performed using the same methodology. It's just that the parameters were varied to derive -- to come up with the numbers that were shown in my testimony.

Q. Has it been reviewed and gone through quality assurance, quality control, which I assume Exhibit UU has gone through?

A. I have not prepared an official calculation, but the pages that I'm reporting results from have been reviewed by Dr. Tseng.

Q. Have they gone through quality control such as your calculation in Exhibit UU?

A. Well, it's received the same kind of review that an official calc would receive.

Q. So kind of?

A. Well, yes. Because --

Q. Okay, that's fine.

A. I mean, as you can see, Dr. Tseng has reviewed and independently reviewed the rest of these calcs, so he is very familiar with these and he is qualified. The only thing is, I didn't have time to put together an official calc and have a cover page signed off on it. It's the same sort of review that an official calc would have received.

Q. And have you also prepared a dynamic stability analysis of the canister transfer building?

A. Yes.

Q. Will the foundations of the pads have factors of safety greater than 1.1 for a 10,000-year return period event?

MR. TRAVIESO-DIAZ: Ms. Chancellor, in the interest of saving time, if you could refer the witness to the page of the person's testimony you're asking him from, if you can.

MS. CHANCELLOR: Well, I don't know where he's done his 10,000-year return period analysis. He says he has, and I'm just asking him whether it meets the 1.1 factor of safety against sliding.

Q. (By Ms. Chancellor) I mean, it's not something you know off the top of your head, Mr. Trudeau?

A. No, it's not.

Q. That's okay. We'll move on.

In calculation G(B) 04 Rev. 9, which is Exhibit UU, isn't it true that there's a -- that the minimum that you're trying to meet or may have met is a 1.1 factor of safety against overturning, correct?

A. Correct.

Q. And a 1.1 factor of safety against sliding, correct?

A. Correct.

Q. Are you familiar with NUREG 75-87, section 3.8.5, Foundations?

A. Yes.

Q. And are you responsible for assisting in certain -- writing certain sections of the Safety Analysis Report?

A. Yes.

Q. And the Safety Analysis Report on

2.6-45, Revision 21 -- and I have a copy if you would like, if you would like to see it -- it states that in accordance with NUREG 75/078, Section 3.8.5, Foundations, section Roman Numeral 2.5, "Structural acceptance criteria. The minimum factor of safety against overturning and sliding due to extreme environmental conditions such as the design-basis earthquake is 1.1." Does that sound familiar?

A. Yes, it does.

Q. And so do you consider that NUREG 75/087, the section that I mentioned, is that the NUREG guide that you are planning to meet?

A. Yes, with the caveat that that's been superseded by NUREG 0800.

Q. You've done a new -- you've done one like Judge Farrar. That was my next question. And NUREG 0800 section 3.8.5 --

A. Has similar verbiage.

Q. -- has similar verbiage. And is it PFS's -- never mind.

A. If I'm not mistaken, the verbiage is that the factors of safety are found acceptable if they meet these guidelines.

Q. Do you have -- what do you have in front of you there? Do you have a copy of your testimony and the PFS exhibits?

A. Yes.

Q. Okay. If you would turn to answer 7 on page 3. Actually, you don't really need to look at the answer. I'll just ask you a question. Is it common in engineering practice to design with factors of safety against sliding and bearing capacity failure below 1.1 for natural phenomena such as earthquakes?

A. For sliding, sometimes; for bearing capacity, generally no.

Q. It's acceptable to have less than a 10 percent margin of safety against sliding for earthquakes; is that correct?

A. That's correct. Some buildings are designed to have the foundation move underneath them so that they minimize the forces that the earthquake inputs to the building. These buildings are referred to as base isolated structures.

Q. Are you familiar with any nuclear structures that are base isolated structures?

A. No.

Q. Are you familiar with the term "reliability analysis"?

A. I've heard the term.

Q. Do you know what it is?

A. I haven't performed any reliability analyses.

Q. Do you know what the term is? Can you define the term?

A. I could only give you a guess.

Q. Have you performed any -- have you estimated the uncertainty in your estimates of the factor of safety?

A. No.

Q. In either the pads or the CTB?

A. No.

Q. And why not?

A. I was working with lower bound strengths to develop conservative estimates of what the lower bound factors of safety would be.

Q. Does this imply that the factors of safety that you have used in your evaluations do not have any uncertainty?

A. I'm intending to imply that the uncertainty would be that the factor of safety should be higher.

Q. It doesn't go the other way?

A. Not when you're working with lower bound strengths. And when you're ignoring -- conservatively ignoring the effects of passive resistance, for instance, on the side of the pad.

Q. In answer 9 you define -- would a definition of failure be a factor of safety equal to or less than 1.0?

A. Yes.

Q. If uncertainty exists in your evaluations of the soil and dynamic loadings, is it possible to have failure even if the estimated factor of safety is greater than 1.1?

A. I guess so.

Q. Have you calculated the probability of failure for seismic stability of the pad and its uncertainties?

A. No.

Q. Have you done the same -- have you also conducted a similar analysis for the CTB --

A. No.

Q. -- foundations?

A. No.

Q. I'd like to hand out a copy -- it's an exhibit from the SAR and it's a fold-out. It's a fold-out sheet. And while it is convenient to be in Salt Lake City, I didn't realize that we would need this until today, and I've had to do it on two pages. If you would like to see the original, I have it here.

JUDGE FARRAR: Do you want this marked for identification as a state exhibit?

MS. CHANCELLOR: Yes, I do, but I can't remember what the next number is, your Honor. State's Exhibit 175.

JUDGE FARRAR: Okay, let's pause and let the court reporter mark that for identification.

(INTERVENOR EXHIBIT-175 MARKED.)

Q. Mr. Trudeau, I've handed out Figure 7.2-7, Cask Storage Pads, Private Fuel Storage Facility, Safety Analysis Report, Revision 22, consisting of two 11 by -- 8 x 11 sheets of paper. And the SAR, this is one sheet so I haven't stapled them together. And if you would put the two together so that it overlaps just slightly.

JUDGE FARRAR: Ms. Chancellor, I think you said 7.2.7. You meant --

MS. CHANCELLOR: 4.2-7. That's why I'm not an engineer, your Honor.

A. You notice I got that one, too.

Q. I'd like to talk a little about soil cement, and looking at this diagram I think will help us as we go through the explanation. Under the storage pads there will be cement treated soil, correct?

A. Correct.

Q. And the intent is to remove the top silty layer which is also called the Eolian soil; is that correct?

A. That's correct.

Q. And you would replace the top silt layer with a one- to two-foot layer of cement treated soil; is that correct?

A. That's correct.

Q. And then on top of the cement-treated soil you would place a three-foot-thick concrete pad; is that correct?

A. That's correct.

Q. And the total depth of cement-treated soil and the concrete storage pad would be five feet, correct?

A. Yes; but for clarity, I think we should refer to the material above the base of the pad adjacent to the pad as soil cement.

Q. I'm getting to that one next.

A. Okay.

Q. Okay. Around --

A. I do what I can to move this thing along.

Q. Around the pads, if we start at a depth of five feet, because we've got five feet under the pads with the three-foot-thick concrete pad and the cement-treated soil.

A. That's correct.

Q. So if we start at a depth of five feet, and the layer from five feet to three feet will be a two-foot layer of cement-treated soil, correct, that extends out from under the pads to around the pads?

A. That's correct.

Q. And on top of the cement-treated soil there will be a two-foot four-inch layer of soil cement?

A. That is correct.

Q. And then there will be eight inches of gravel on top of the soil cement, correct?

A. That is correct.

Q. So going from the top to the bottom, we've got an eight-inch layer -- around the pads, we've got an 8-inch layer of gravel on the top, correct?

A. That's correct.

Q. And then we have a two-foot four-inch layer of soil cement beneath the gravel, correct?
A. That is correct.
Q. And then we have a two-foot layer beneath the soil cement of cement-treated soil, right?
A. That is correct.
Q. Okay. We're all on base one.
Isn't it true that PFS's soil cement testing program is on hold?
A. Yes.
Q. Isn't it true that a large part of PFS's soil cement testing program will be done if and when PFS obtains a license from NRC?
A. That's my understanding, yes.
Q. In answer 14 on page 5 you state, the base case you analyzed "conservatively uses shear strengths of the clayey soils based on static strengths measured in direct shear tests." Is that correct? It's towards -- it's on page 6, actually, towards the end of the answer, five lines from the bottom.
A. Yes.
Q. And if you would turn to Exhibit UU, attachment C, page -- it's actually the sixth page

from the end of the calculation.

MR. TURK: Is there a page number on it on top?

MS. CHANCELLOR: It's a little difficult to find. It's -- oh, page 32. Easier to find if you count six pages from the end of the calculation. Attachment C, page C2 is across the long dimension and across the top of the page is P 32.

Q. (By Ms. Chancellor) Do you have it, Mr. Trudeau?

A. Yes. This is page C-2 of attachment C of calc G(B) 4.

MR. TURK: Could we have a minute while we look for it, your Honor?

JUDGE FARRAR: Yes.

MS. CHANCELLOR: Okay, go to the end of --

MR. TURK: Excuse me, just one minute. Okay. Thank you.

Q. (By Ms. Chancellor) Isn't it true that you conducted only one set of three tests to define the failure envelope for the undrained shear strength?

A. For the pad emplacement area, yes.

Q. For the pad emplacement area. And if you look at the top of Figure 7, Direct Shear Tests. It states boring C-2, sample U-1C. Do you see that?

A. Yes, I do.

Q. And the undrained shear strength tests were conducted from this one sample; is that correct?

A. Correct. That sample was obtained near the base of the pads.

Q. And the size of the pad emplacement area is 99 acres, correct?

A. That's a number I've heard recently. I'm not sure exactly that it's the right number, but it's about that, I believe.

Q. And so the undrained shear strength is taken from one sample from this 99-acre area, correct?

A. For the pad emplacement area, right.

Q. For the pad emplacement area. In answer 16 on page 6 you state that you have performed several simple calculations to estimate how much the factor of safety against sliding would increase if the shear strengths of the clayey soils was increased 50 percent from the strengths obtained in the static strength test account for the well-known phenomenon of dynamic strengths of clayey soils, etc. And isn't it true that you have no test data for any of the simple calculations you performed with respect to the dynamic strengths of clayey soils?

A. Excuse me? You just pointed to some strength data.

Q. Isn't it true that you have no test data for the 50 to 100 strength tests -- strength -- for the 50 to 100 percent increase in strength due to strain rate? Did you understand that question?

A. No, I'm afraid I did not.

Q. Isn't it true that the 50 to 100 percent increase in strength is attributed to shear strain rate effects?

A. Yes, that well-known phenomenon that the dynamic strength of clayey soils is 50 to 100 percent greater than the measured static strength.

Q. And you have no data, site-specific data to support that representation, correct?

A. We have a variety of static strength tests that have been performed on the site soils.

Q. In answer 18 on page 9 you state the factor of safety against sliding between the base of the concrete pad and the underlying cement treated soil is 1.98, correct?

A. Correct.

Q. Isn't it true that you have conducted no testing at the interface between the base of the concrete on the underlying cement-treated soil?

A. Not to date. We are committed to performing those tests.

Q. But to date you have not performed those tests?

A. That's what I said.

Q. Isn't it true you have not confirmed this factor of safety by performing shear strength testing of the cement-treated soils mixture using PFS soils?

A. I'm sorry, I didn't catch that.

Q. Isn't it true that you have not confirmed the 1.98 factor of safety by performing shear strength testing of the cement-treated soils, mixing them with the PFS soils?

A. Yes. That testing remains to be completed. That factor of safety is based on the measured strength of the silty clay soils, and we fully expect that the cement being added to the Eolian silt soils will result in higher strengths than what are used to calculate that factor of safety.

Q. In answer 16 on page 6 you go through a whole bunch of -- a couple of pages of description. None of -- in answer 16, none of these descriptions are included in calculations that you submitted to the NRC. Is that correct?

A. That's correct. They're calculated based on the same methodology that's described in the calculation.

Q. But they're not part of UU, for example?

A. That's correct.

Q. In answer 17 on page 8 you state that the base case, the sliding of the pads has several conservatisms built into it such as cohesive strength and passive resistance provided by soil cement, correct?

A. We're looking at A17?

Q. Yes.

MR. TRAVIESO-DIAZ: I'm sorry. I don't see that on A17.

A. The base case did not include the passive resistance of the soil cement. That is one of the analysis assumptions referred to in A17 as being conservative.

Q. You refer to conservatism throughout your testimony. Have you analyzed the potential loss of strength of soil cement due to shrinkage or settlement cracking?

A. The strength from the cement-treated soil that we need and the strength that we need from the soil cement will not be adversely affected by shrinkage cracks.

Q. Did you analyze the potential loss of cohesive strength of soil cement due to tensile cracking from a seismic event?

A. No. I don't understand how the cohesive strength would be affected by tensile cracking.

Q. Did you analyze the potential cracking of soil cement resulting from kinematic interaction with foundations?

MR. TURK: Could I hear that one again, please?

Q. Did you analyze the potential cracking of the soil cement resulting from kinematic interaction with the foundations?

MR. TRAVIESO-DIAZ: Mr. Chairman, as I said earlier, I'm not objecting to these kind of questions, but if we're going to go so much into section C, the witness is qualified to answer them.

I don't recall ever asking him the questions, but I think we are running the risks of inefficiency that was stated before. All of this is section C, and we'll be talking about it at great length, I'll assure you.

Q. (By Ms. Chancellor) I'll go on to section D. Right, soil cement is in section C.

Did you analyze for pad-to-pad interaction?

A. No.

Q. And the potential increase in seismic loading resulting from the interaction of the pads?

A. There won't be interaction between the pads if the pads don't slide. Our base case, the pads don't slide.

Q. Did you include the potential amplification of ground response underneath the pads due to pad flexibility?

A. Pads aren't flexible, so that doesn't need to be included. So no, I didn't include it.

Q. Did you include potential amplification due to soil-structure interaction?

A. Inasmuch as -- yes, inasmuch as I used the dynamic loads for the casks from CEC's calculation, those effects are included.

Q. Did you perform site-specific testing of the upper Bonneville clays to confirm that the strain rate effects are as large as you claim?

A. Which strain rate effects are you talking about?

Q. 50 to 100 percent increase as is in your testimony.

A. No. There's been no testing of rapidly loaded samples. Might I add, that's why we don't use an increase in the static strengths for the base case.

Q. Was accidental torsion included in the design of the pads?

A. Not geotechnically. I'm not sure whether the pad designer included that effect or not.

Q. Do you know whether Holtec included the potential variation in ground motion by using multiple time histories to determine the forces to the pads?

MR. TRAVIESO-DIAZ: Excuse me. Again, I don't want to interject, but this is way out of the scope of what this witness has testified.

MS. CHANCELLOR: That's fine. I'll withdraw that question.

I'd like to have marked as State's Exhibit 175 --

MR. O'NEILL: 176.

MS. CHANCELLOR: Oh, 176. This is a Holtec proprietary document, and I assume that we're going to treat this like the other Holtec documents. It's called Multi-cask Seismic Response at the PFS ISFSI, Holtec Report HI-971631, and it consists of a cover page, a reviewer and certification log as page 3 and page 4.

JUDGE FARRAR: All right, we'll have the reporter mark it for identification as State 176, with the reminder to everyone that this is a proprietary document and it will be handled separately from other exhibits.

(Discussion off the record.)

(INTERVENOR EXHIBIT-176 MARKED.)

MS. CHANCELLOR: As a housekeeping matter, can I move for entry of State's Exhibit 175, the two pages from the Safety Analysis Report?

JUDGE FARRAR: Any objection?

MR. TRAVIESO-DIAZ: No.

MR. O'NEILL: No.

JUDGE FARRAR: It will be admitted.

(INTERVENOR EXHIBIT-175 ADMITTED.)

Q. (By Ms. Chancellor) Mr. Trudeau, you stated that you started on the PFS project in about February 1997, correct?
A. Yes.
Q. And who else from Stone & Webster would have been involved in the PFS project at that time?
A. Project manager, project engineers.
Q. And who is the project manager?
A. John Donnell, Jerry Cooper.
Q. Are you familiar with this document, Multi-Cask Response to PFS From 2,000-year Event? And the original that I have handed you excerpts from is dated May 19, 1997.
A. I think I've seen a later one of these for the 2,000-year earthquake.
Q. Okay. If you would turn to page 3 of this original report, and if you would please read the sentence that begins at the bottom of page 3.
A. "The characteristics"?
Q. That's correct. And continue over the page.
A. -- "of the pad are based on the assumption that the 30-foot by 64-foot section response to seismic excitation as a rigid body. This assumption has been based on the

recommendation of the project architect and engineering group responsible for the ISFSI design of the PFS facility.

Q. And is Stone and Webster the project architect and engineering firm responsible for the design of the PFS facility?

A. Yes.

Q. Did you make the recommendation to Holtec that the pad should be treated as rigid for seismic analysis?

A. No.

Q. Do you know who at Stone & Webster recommended that the pads be treated as rigid?

A. No.

Q. Are you aware of the calculation by ICEC that was performed for the structural design of the storage pads?

A. Yes.

Q. Do you know whether the original CEC calculation was performed in October of 1999?

A. No.

MS. CHANCELLOR: I'm shifting gears, your Honor.

JUDGE FARRAR: It's almost five after. Why don't we take a break until 4:15 and switch reporters and then press on.

(A recess was taken.)

Q. (By Ms. Chancellor) All set, Mr. Trudeau?

A. Yes, thank you.

Q. What does the term peak ground acceleration mean?

A. That's the maximum acceleration of the ground due to the earthquake.

Q. Who developed the estimates of peak ground acceleration for the vertical and horizontal direction at the PFS site?

A. Geomatrix, Bob Youngs.

Q. I've been asked to slow down by Mr. Turk, and I'm trying.

Isn't it true that peak ground acceleration was developed in the free field by Geomatrix?

A. That's correct.

MR. TURK: Your Honor, let me note, I'm not asking Ms. Chancellor to slow down. I only asked her for a couple of seconds after the answer, in order to make sure we could write it before the next question.

JUDGE FARRAR: Remember in first grade

it said in your report card, in addition to reading and writing, plays well with others.

MR. TURK: I find myself having to respond often. I don't look to initiate these things.

Q. (By Ms. Chancellor) Okay. So we're up to PGA or peak ground acceleration by Geomatrix was developed in the free field; correct?

A. Correct.

Q. And the free field doesn't have any structures placed on it; is that correct?

A. Correct.

Q. If there are no structures, an obvious point is that peak ground acceleration does not include soil-structure interaction; correct?

A. Yes.

Q. Would you turn -- I'm going to ask you some questions about Answer 28 on Page 14. In Answer 28, you discuss why you lose peak ground acceleration to get seismic loads for the pad itself; correct?

A. Correct.

Q. Isn't it true that the dynamics stability analysis that you performed of the CTB, you have used the dynamic response of the building and the mat in terms of structural acceleration response on the soil-structure interaction model?

A. Yes.

Q. If you would turn to Exhibit VV at Page 49. That's okay, we'll do this later.

Isn't it also true that the pad dynamic stability analysis, you estimated the pad response based on peak ground acceleration times the weight of the pads?

A. Correct.

Q. In the stability analysis of the pads, you are estimating the response of the pads; correct?

A. Correct.

Q. Holtec has provided a seismic analysis of the pads and provided ICEC with the dynamic loads acting on the pad; correct?

A. That's my understanding.

Q. Were you here for the discussion this morning we had --

A. Excuse me, if I'm not mistaken, the loadings that Holtec provided to CEC were at the top of the pad, I believe.

Q. And are you aware whether that was -- were you here this morning when we had the discussion about Holtec force time histories that they just had provided to the State?

A. I was in and out this morning, so I'm not sure I heard the particular discussion you're talking about.

Q. The very first thing this morning?

A. I was late this morning.

Q. Oh, shame you on, Mr. Trudeau.

A. They told me to tell the truth here.

Q. I'm very pleased that you are doing so.

Isn't it true that you considered soil-structure interaction for the canister transfer building in your analysis contained in Exhibit VV?

A. Inasmuch as the loads came from our structural dynamics people, yes.

Q. If you look at Exhibit VV on Page 49, which is Table 2.6-11, Foundation Load into the Canister Transfer Building.

A. Correct.

Q. What is the zero period acceleration response at the base of the mat from the results of soil-structure interaction performed for the CTB?

A. I'm sorry, say again, please.

Q. If you look at the value for the AX.

A. Okay. I believe it would be shown for joint one, which is 1.047g.

Q. Isn't it true that one reason for the increase in ground motions in the CTB is that the ground motion in the horizontal direction has been amplified by the response of the CTB and its mat foundation?

A. I believe that's correct, yes. But the CTB is a much -- it's a building that sticks way up out of the ground. So it's not surprising that there's some soil-structure interaction effects for that building.

Q. And the 1.047g in the AX direction to the CTB on Page 49 of Exhibit VV, is that about a 47 percent increase over PGA?

A. Well, it's .71 to 1.047.

Q. Do you have a calculator?

A. Yes.

Q. Could you tell me what the percent difference is between CTB response on Page 49 of your calculation to that of peak ground acceleration?

A. 47 percent looks good to me.

Q. Isn't it true that a 40 percent increase is a significant increase in the zero period acceleration?

A. Yes.

Q. Isn't it true that similar amplification is possible for the storage pad?

A. I don't believe so.

Q. I'd like to show you -- I don't know whether everybody has a copy of this. It's the -- in the State's exhibits as well as the Staff's exhibits is some excerpts -- in State's exhibits are some excerpts from the Luk report, and the Staff has also introduced the Luk report as Staff's Exhibit P. I'd like to show -- and State's Exhibit is 115. Have you seen the report prepared by Dr. Luk and others from the Sandia labs?

A. No.

Q. I'd like to show you Figure 17 from the -- from Dr. Luk's report. Luk or Luck, Sherwin?

MR. TURK: Luk.

MR. O'NEILL: Luk.

MS. CHANCELLOR: Luk, and Mr. Goodluck.

Q. (By Ms. Chancellor) Do you see Figure 17 on that document that Ms. Nakahara just gave you?

A. Yes.

Q. Is it correct that Figure 17 shows the acceleration of the pad?

MR. TURK: Your Honor, I'm going to object unless the witness can tell us whether he's ever seen this document before or understands what it is.

MR. TRAVIESO-DIAZ: In fact, Your Honor, I think he just testified he's never seen it before.

MS. CHANCELLOR: I'm asking him to review it. There's just one particular point I want to make with this exhibit.

MR. TURK: Well, Your Honor, she's showing him the figure without showing him the accompanying text in the report. And I think the witness has already said he hasn't seen this report before. We're not adding to the record. We're merely adding to confusion.

MR. TRAVIESO-DIAZ: Moreover, I believe that we have a number of people who can try and address this particular document, and it has already been addressed by other witnesses.

JUDGE FARRAR: It may be from your point of view, it's going to be addressed by other people, but this is cross-examination, she's entitled to test the limits of this witness's knowledge. You may argue later that if he's deficient in this knowledge, it doesn't matter, but she's entitled to test him. And, Mr. Turk, what happened to Mr. O-Neill?

MR. TURK: Mr. O-Neill is here, Your Honor.

MR. O'NEILL: I would state that the State has --

JUDGE FARRAR: No, no, no. Who's talking for the Staff at this point, was the point of my question?

MR. O'NEILL: I will, Your Honor.

Q. (By Ms. Chancellor) Mr. Trudeau, if you will look at Page 33 of the exhibit.

JUDGE FARRAR: But wait.

MS. CHANCELLOR: I beg your pardon.

JUDGE FARRAR: I had overruled the Applicant's objection, but the Staff's objection about making sure the witness has all of them if he's not familiar with this report, that he not only -- that he has some context is a fair point.

MS. CHANCELLOR: I'll hand the witness the entire report, Your Honor.

MR. TURK: Your Honor, may we ask which

dated report the witness was handed?

MS. CHANCELLOR: It's Revision 1 that we received on April 2, the day after you prefiled testimony, Mr. Turk.

Q. (By Ms. Chancellor) If you would look -- let me just step you through it and then if you've got -- if you need time to review, that will be just fine. I just have a couple of questions, and if you need more time to review -- I don't want you to have to sit on the stand and read the entire report. If you look on Page 33, there's a designation of the description where various -- of various locations beneath the pad. Do you see the Point D prime center of pad, top of soil cement layer on Page 33?

A. Page 33. I'm just a little confused, because the Page 33 I was looking at a moment ago was different than this report, I thought. This Page 33 has a point labeled D.

Q. Do you see the Point D prime on Page 33?

A. Yes, I do.

Q. And D prime is the center of the pad, top of soil cement layer; correct?

A. That's what it's labeled, yes.

Q. And if you turn the page to Figure 17, Soil-structure Interaction Comparison Best Estimate Soil Profile Data. Do you see that?

A. Yes, I do.

Q. And D prime is the red line; correct?

A. According to the legend, yes.

Q. What's the acceleration of -- what's the maximum acceleration of D prime?

MR. TURK: Excuse me, Your Honor. Would you repeat that question, Ms. Chancellor.

Q. (By Ms. Chancellor) Yes, what's the maximum acceleration of D prime?

A. Something less than minus 3g.

Q. And that value is greater than peak ground acceleration of 0.11g; is that correct?

MR. TURK: I'm sorry, could you say it again? You're asking -- Your Honor, I recognize that Mr. O-Neill is responsible for this witness, I'm responsible for defending Dr. Luk and his report. And what you're having done here is a witness who has never seen a report being asked to provide record evidence of what a report means.

MS. CHANCELLOR: No, I am just showing --

MR. TURK: We have witnesses here who are able to explain the report. I don't think this witness should be interpreting a report on the spot. It's a hundred pages long. It has clarifying text that explains what these data are. Whether they're filtered data or unfiltered data or any other qualifications. We're going to have to go through lengthy cross by me in order to establish the limits of this witness's ability to testify about the document, and it's clear from the get-go, he has no ability to testify about it. So why are we wasting time on it?

JUDGE FARRAR: I thought when I overruled the Applicant's objection, Ms. Chancellor, that you were going to use this report to test this witness's knowledge as opposed to getting into the record what the report contains. And I thought the basis for overruling the Applicant was to give you some leeway to test his knowledge rather than to put material in from this report which we have ample occasion to do later.

MS. CHANCELLOR: Your Honor, I just have one question and it deals with, if you see 3g in the Luk report and Mr. Trudeau used a value of .711g, then how does he account -- how do you account for the discrepancy that there's no amplification of ground motions in the storage pad? That's the sole reason for going through this exercise.

MR. TURK: And I would object to that question because it's not shown the witness understands what this 3g number may mean.

MS. CHANCELLOR: If he doesn't understand it, he should so testify.

JUDGE FARRAR: Wait, wait, wait. We've said this a couple of times before. Talk to me, not to each other.

Second, I'll permit this one question for this limited purpose, which I understand why did the witness testify to one -- or how does the witness reconcile his testimony with the information in this report. But it's a fair answer for the witness to say, I don't understand what this report is saying.

MR. TRUDEAU: I would need more time to understand what this report is saying.

Q. (By Ms. Chancellor) Thank you, Mr. Trudeau.

Still on Page 14, question -- Answer 28 of your testimony. Mr. Trudeau, in Answer 28, you discuss that radiation damping is high, therefore the foundation response is the same as peak ground acceleration input you used; is that correct?

A. Basically, yes.

Q. Do you have any -- have you performed any calculations to support your view?

A. Just the simple calculation of what's shown here, the results as shown here.

Q. How did you obtain the 50 percent damping?

A. Using the critical damping as two times the square root of the stiffness times the mass for vertical motion.

Q. And the basis for selection of that?

A. That's an equation from Page 5 of Numark & Rosenblue. I believe that equation is also shown in Holtec's reports where I received the stiffnesses and mass values to arrive at those numbers that are shown on A28.

Q. And in your calculation of the CTB on Table 2.6-11, did you use the same method for calculation of the mat accelerations that reach 1.047g?

A. What on page -- Table 2.6-11?

Q. Did you use same the methodology as you used in Answer 28?

A. No.

Q. And in Answer 28, is not something that you have submitted to the Nuclear Regulatory Commission as a licensing document; is that correct?

A. That's correct.

Q. In the last part of your Answer 28, you state that the factor of safety against sliding is 1.27 and against bearing is 1.17; correct?

A. I believe A28 only dealt with sliding, and 1.27 is the right number for that.

Q. In calculating this factor of safety, you didn't consider potential amplification of accelerations of the pad in a horizontal direction, correct?

A. In the answer to A28, is that what we're talking about?

Q. In calculating the factor of safety of 1.27 against sliding, isn't it true that you did not consider potential amplification of acceleration of the pads in the horizontal direction?

A. That is correct, for the value of 1.27. But the response to A28 demonstrates that a factor of safety of 1.25 is arrived at when I use Holtec's time history of acceleration values, that I understand do include the amplification effects that you're talking about.

Q. And that's not part of Exhibit -- of your calculation Exhibit VV -- I'm sorry, UU that you submitted to the NRC; is that correct?

A. That's correct.

Q. And you did not consider potential amplification of acceleration of the pads in the horizontal direction -- in the vertical -- no, horizontal. Vertical. I said horizontal before. Vertical; is that correct?

A. Correct.

Q. If radiation damping is as high as you claim, doesn't this indicate that soil-structure interaction is very important?

A. The radiation damping is part of the soil-structure interaction, and because of the radiation damping, that this structure, the peak ground acceleration is not a bad value. There is not much amplification because of the characteristics of the structure as opposed to the canister transfer building -- this is a three-foot thick concrete pad that's buried below grade, whereas the canister transfer building is a massive concrete structure. It's not unexpected that there would be some difference in the dynamic characteristics of these two structures.

Q. And GB04 does not include soil-structure interaction -- your analysis in GB04, Exhibit UU does not include soil-structure interaction; is that correct?

A. No, it includes the dynamic loads from CEC's calculation, which were based on Holtec's loads, I understood at the top of the pad. So therefore, all of the cask loadings which make up the bulk of the loading for this pad and cask structure, do include the effects of soil-structure interaction. It's just the three-foot thick pad that we're talking about is not being included as an amplification from the soil-structure interaction analysis.

Q. What is the static bearing pressure of the pads?

A. It's a little bit below 2 KSF.

Q. And what is the static bearing pressure of the CTB?

A. I believe that also is a little below two, but just bear with me a moment. The actual static bearing pressure under the CTB is shown on

Page 47 of Exhibit VV, which is SAR Table 2.6-9 and it's 1.46 KSF. Similarly, the actual bearing pressure under the pads is shown on Page 106 of Exhibit UU, which is table 2.6-6 from the SAR, and it equals 1.87 KSF.

Q. Thank you, Mr. Trudeau.

JUDGE FARRAR: Ms. Chancellor, are you moving to a new subject?

MS. CHANCELLOR: Yes, I am, Your Honor.

JUDGE FARRAR: Then let me follow up on the previous interchange.

Mr. Trudeau, on that question Ms. Chancellor wanted to ask you from the Luk report, where she wanted to compare something in that with your testimony, let me make sure I understand. You're saying that when she showed you the tracing in the Luk report and what it referred to, that's not something you can look at, know what it is and give a response to; is that what you're saying?

MR. TRUDEAU: What I saw on that report was some indications of directions or what I thought might be directions, so I'm not sure what the axes were for that report, without having seen the report.

The other thing that I saw in the title of the figure led me to believe that that was for the hypothetical case where we looked at only friction resistance of the pad based on a fee angle of 17 degrees, which as I've testified, results in sliding of the pad. To me that perhaps is an indication that the amplification might be different than it would be for our base case with the cohesion that we expect to demonstrate by testing that exists at that interface.

So without going into the report in more detail, you know, I've got at least those two questions having seen it for only five minutes.

JUDGE FARRAR: We're not going to finish this witness tonight, are we?

MS. CHANCELLOR: Oh, absolutely not, Your Honor.

JUDGE FARRAR: Then you'll be able to look at this report tonight and be able to answer the question tomorrow?

MR. TRUDEAU: Well, I will look at the report tonight and hopefully I will be able to answer the questions tomorrow.

JUDGE FARRAR: Okay.

MS. CHANCELLOR: You've been given a

homework assignment, Mr. Trudeau.

I'm moving onto a new question.

MR. TRAVIESO-DIAZ: Could you give us a copy of the report? I'm not sure we have it. We have it?

MR. TRUDEAU: The correctly dated one?

MR. GAUKLER: Which one?

MR. TRUDEAU: There was some question about date on that.

MS. CHANCELLOR: I don't know what the differences are in the two reports. I don't think there's any differences in Figure 17, but I could be wrong.

JUDGE FARRAR: There's a difference in the pagination, and somehow the report changed between March 8th and March 30th.

MR. TURK: There was additional work done on Tables 9 and 10, which pushed back the pages by one page.

JUDGE FARRAR: So the information is the same on those two pages?

MR. TURK: My understanding is that Figure 7 is the same in both reports.

MR. TRAVIESO-DIAZ: My problem is the following. I'm sure in order for Mr. Trudeau to come up with that figure, it has to appear in the text. And I have no idea without having the actual report that he's been asked about, how he can interpret what that figure means. We all have various copies but I want to make sure that what he's looking at is the same document for which Ms. Chancellor would like an answer. That's the reason why I asked her to go ahead and get a copy of the document that she wants to ask him about.

MS. CHANCELLOR: It is Staff's Exhibit P, and PFS has a copy of Staff's Exhibit P, and the Holtec witnesses, Dr. Tseng and Dr. Soler testified at length about this report.

JUDGE FARRAR: I think that's fair, that counsel for the Applicant either will find it in their papers or can get it from the Staff.

MS. CHANCELLOR: If they have problems, we'll be glad to make a copy, but we'd rather not kill any more trees.

MR. TURK: Your Honor --

JUDGE FARRAR: Wait, Ms. Chancellor. I once worked for the American Paper Institute, and we planted them and we cut them down.

MS. CHANCELLOR: Shame on you, Your Honor.

JUDGE LAM: He's known as my mentor.

MR. TURK: Your Honor, there may be a shortcut to all of this.

JUDGE FARRAR: Okay.

MR. TURK: If Ms. Chancellor asks in a hypothetical way, would someone's report that has a figure in it of something approximating 3gs, would that affect your opinion? That's a hypothetical statement. Now, whether she could ever connect that up to the fact, I think is very suspect, because I think she's trying to lead to things that don't exist.

JUDGE FARRAR: Well, let's --

MR. TURK: But I don't mind the hypothetical question if she thinks she can connect it.

JUDGE FARRAR: I think this -- no, because the hypothetical would get too complicated. Let's let the witness see what it is. He can then answer the direct question and we'll have the matter resolved.

Go ahead, Ms. Chancellor.

Q. (By Ms. Chancellor) On Page 19 on Answer 32, you testified that the effect of shear strains that develop in the clay layer is 0.13

percent of the upper Bonneville clay and about 0.0034 percent of the soil cement overlaying the upper Bonneville clay; is that correct?

A. That's correct.

Q. I know this is part of Section D, but just so that we're not all confused, the top layer of the soils at the PFS site is the eolian silt; correct?

A. Correct.

Q. And then the next layer is the upper Bonneville clay, and that's sometimes called clay silt?

A. Silted clay, clay silt.

Q. Does it matter if you reverse them?

A. I've tried to refer to them that way all the time, but it doesn't really matter. It's the same whether you call it one way or the other.

Q. And the soils of interest at the PFS site are the topsoil layer, soil cement and the upper Bonneville clays; correct?

A. Correct.

Q. You also testified that because of the small shear strains, subsequent interaction between the pads will be insignificant; is this correct?

A. Correct.

Q. From which calculation were these effective shear strains obtained?

A. As it says in A32, they're from calculation 05996.02-G(PO18)-2, Rev.1, which is a Geomatrix calculation of the strain compatible soil properties at the site.

Q. Isn't it true, that this calculation is for the free field ground motion?

A. Yes, it is, but we have so much soil cement out at that site, that we included the soil cement layer on the top of the profile. I mean the soil cement extends 250 feet plus or minus around the canister transfer building and it includes the entire area of the bad and placement area, extending at least a hundred feet beyond the pads at the perimeter.

Q. And the Geomatrix calculation does not include the masses and stiffnesses of the concrete pads; correct?

A. That's correct.

Q. And it does not include the mass of the storage casks; correct?

A. That is correct.

Q. Isn't it true that you use the effective strain obtained from the free field ground motion analysis for the case of determining the dynamic shear strains that developed underneath the pad?

A. That's correct.

Q. And the pad is loaded with eight casks each weighing about 360 kips?

A. That's correct.

Q. That's about 2,880 kips; correct?

A. The numbers that I have in my calc, I believe are 2,852 kips based on 356 and a half kips, but it's ballpark.

Q. I'll go with your numbers, then, Mr. Trudeau.

Isn't this the case where there will be significant interaction between the foundations and the soils where a heavily loaded foundation resting on top of deformable soils is -- let me -- where a heavily loaded foundation resting on top of deformable soils is subject to high levels of ground motion, won't there be significant interaction between the foundation and the soil?

A. I have to disagree for the pads, because if that were true, the factor of safety that I reported based on Holtec's loads, the time history of accelerations back on Page 15, in my Answer 28, would never have come out as close as they are to the 1.27 calculated for the PGA value.

Q. Isn't it correct that you have not performed a soil-structure interaction analysis?

A. That is correct.

Q. In Answer 32, you have suggested in numerous places that additional passive -- oh, let me -- it's on Page 19, Mr. Trudeau.

A. Thank you, I'm there.

Q. Oh, okay. In Answer 32, you have suggested in numerous places that additional passive resistance to sliding will be provided by soil cement that is placed adjacent to the pad; correct?

A. I'm sorry, I don't see that in A33. Could you please repeat the question.

Q. In Answer 32 --

A. Oh, 32, I'm sorry.

Q. 32, yes. Not 33, 32,, on Page 19.

A. And again, what about A32?

Q. You have suggested in numerous places that additional passive resistance to sliding will be provided by the soil cement that is placed adjacent to the pad?

A. In my testimony today, but that's not in A32; right?

Q. Oh, it's not? But you have -- that is part of your testimony; is that correct, Mr. Trudeau?

A. That is correct.

Q. Can you explain how this additional passive earth pressure or force is mobilized and how it is available to resist sliding?

A. It is my belief that that soil cement plug will be bonded to the underlying Bonneville deposits, silty clay litter by the cement treated soil as will be the pads. Therefore, the pads will not slide, the plug of soil cement will not slide.

Q. Will there be some transfer of the dynamic load from the pad to the adjacent soil cement plug that is placed in between the pads as passive earth pressure is mobilized?

A. If the pads wanted to slide, they would mobilize the passive resistant.

Q. You mention a soil cement plug. What do you mean by a soil cement plug?

A. By that, I'm referring to the soil cement adjacent to the pad between the pads.

Q. In the five-foot direction?

A. Yes, in this case, but it exists any other direction, as well.

Q. And is that five-foot cement plug in the longitudinal direction?

A. Yes.

Q. Is this five foot plug -- isn't it true that the five foot cement -- five foot plug is not structurally tied to the pads?

A. That's correct.

Q. There's no rebar, for example?

A. Exactly.

Q. In Answer 32, you state, "It is therefore anticipated that the pad and soil cement plug between the pads will deflect in phase with the underlying soils, meaning that the interaction between the pads will be insignificant."

Do you see that? It's at the very end of Answer 32.

A. Yes.

Q. Does your conclusion that the pads will deflect or move in phase include the entire row of pads moving in phase?

A. Yes.

Q. Dr. Wen Tseng was responsible for the structural design of the pads; correct?

A. Yes.

Q. And Dr. -- Dr. Wen Tseng's background is in dynamic structural analysis and soil-structure interaction; correct?

A. That's what I understand.

Q. Are you aware that Dr. Tseng stated in his deposition that soil cement around the pad is not structurally -- not integrated structurally or structurally tied with the pads and cannot be viewed as an integrated foundation mat during earthquake cycle? Are you aware of that?

A. I am now.

Q. Would you like to see a copy of his deposition or will you take that on representation?

A. That's fine.

Q. If the pads and soil cement cannot act as an integrated mat, then do you agree that there is the potential for pad-to-pad interaction?

A. Only if the pad slides. The factor of safety against sliding is 1.27. That's a 27 percent margin above the point at which it might slide.

Q. And that's for the base case; correct?

A. Yes, without including the passive resistance of this soil cement that we're talking about.

Q. And that's for soil cement under -- that includes the soil cement under the pads, cement treated soil under the pads; correct?

A. Yes, that includes the cement treated soil under the pad.

Q. Isn't it true that you did not consider the load transfer from one pad to an adjacent pad in your sliding analysis, in any of the cases whether you're talking about the base case or the hypothetical case?

A. The base case has no loads to be transferred because the pads do not slide. The hypothetical cases would -- where the pads would slide, it is possible that there would be loads transferred from pad to pad. Those loads are included in the hypothetical cases that looked at the whole column of pads.

Q. Are the Bonneville clays a deformable body under dynamic loading?

A. I believe that's correct.

Q. Is it true that for your base case for sliding analysis, that it assumes that the majority of the peak undrained shear strength of the upper Bonneville clays will be available to resist the sliding of the pads?

A. Yes.

Q. Have you evaluated the amount of horizontal displacement that is required to mobilize the peak undrained shear strength of the upper Bonneville clays?

A. I believe I have.

Q. How much displacement?

A. Your question again was how much horizontal displacement is required?

Q. Have you evaluated the amount of horizontal displacement that is required to mobilize the peak undrained shear strength of the upper Bonneville clays?

A. That information is in Exhibit VV for the canister transfer building. It's similar in effect to the pads because the depths involved are the same.

Q. And what page is that on, Mr. Trudeau?

A. Excuse me?

Q. What page is that on?

A. It's Page 19. It says that a horizontal displacement of the mat in this case, assuming one percent yield ratio for the five-foot height would be a displacement of .78 inches. Now, that conservatively overestimates the amount of displacement required because the soil cement will be much stiffer than the soil for which these relationships are derived.

Q. Isn't it true that the base case for the pads does not include the soil cement acting as a buttress?

A. That's correct.

Q. And in the canister transfer building, the soil cement in the base case does not use soil cement as a buttress?

A. That's correct.

Q. Then how -- why is the CTB case applicable for the displacement?

A. This would be an upper bound of how far it would have to go to build up the passive resistance if it needed that. That's not the question you were asking?

Q. No.

A. I guess I misunderstood the question.

Q. I'm asking the amount of horizontal displacement that is required to mobilize the peak undrained shear strength of the upper Bonneville clay? And if you're referring to the canister transfer building, you've got a different base case, don't you, than for the storage pads?

A. Correct.

Q. Do you want another homework assignment, Mr. Trudeau?

A. That would probably be better than taking up time here.

Q. If you would turn to the third to the last page of the pad stability calculation, Exhibit VV, Attachment C, and it's C1 -- Page C1/3. And on the top it's got direct shear.

MR. TRAVIESO-DIAZ: You mean Exhibit UU?

MS. CHANCELLOR: Oh, yes, UU, thank you. My Vs look like Us.

MR. TRUDEAU: Was that Page C1?

Q. (By Ms. Chancellor) Page C1/3. It's the third-to-the-last page in the calculation, and it's entitled Direct Shear Test Boring C, C-2 sample U-1C.

A. Is that not Page D1?

Q. C as in Charlie. Maybe I'm in the wrong category. Just a second. This is the CTB calculation -- is this the right one? Sorry. You know your calculations better than we do, Mr. Trudeau. It's D1/3, and it's the fourth page from the end in the pad stability calculation.

A. Yes.

Q. You're already there.

Do you agree that the direct shear tests for boring C-2 on the page that I had referred you to from the sample U-1C indicates that there is about 0.025 inches of horizontal displacement when the sample reaches its peak strength of 2 KSF?

A. That's correct.

Q. Do you agree that the shear strain associated with the peak strength is about 2.5 percent, which is 0.25 divided by 1.00 times 100?

A. I do not believe that's the correct way to calculate strain from this direct shear test.

Q. Do you have a calculator there?

A. Yes.

Q. Could you calculate it?

A. Calculate which?

Q. Shear strain, please.

A. The shear strength?

Q. Strain. We have to do a Dr. Chang. Strain, S-T-R-A-I-N. Strain.

A. The shear strain, you need to know some more about the sample dimensions in order to get that shear strain from these test results.

Q. What if you assumed it was one inch thick?

A. Well, you could assume that if you'd

like.

Q. Okay.

A. But I don't know that that's what it was.

Q. Is it fair to say that the thickness of the upper Bonneville clay is about eight foot thick in the pad and placement area?

A. Yes.

Q. An approximate eight-foot layer and for shear strain of -- let's just assume the shear strain is 2.5 percent. What is the corresponding amount of horizontal displacement to mobilize the peak undrained shear strength?

A. If we assume that it's --

Q. 2.5 percent.

A. A shear strain of two and a half percent?

Q. Correct.

MR. TRAVIESO-DIAZ: Excuse me, for the record, is this a hypothetical?

MS. CHANCELLOR: Because Mr. Trudeau did not agree to the 2.5 percent, yes, it is.

MR. TRAVIESO-DIAZ: I think Mr. Trudeau testified that in order to calculate strain based on this figure, he needs more information than what the figure provides.

MS. CHANCELLOR: We're doing a hypothetical.

MR. TRAVIESO-DIAZ: Okay.

MR. TRUDEAU: Which units would you like?

Q. (By Ms. Chancellor) Inches, please.

A. I come up with 2.4 inches.

Q. Okay. Maybe you can verify this with your homework assignment.

MR. TRAVIESO-DIAZ: I'm sorry, I have no idea what's going on here.

JUDGE FARRAR: Yeah, I don't know. In other words, I don't know how he can verify a hypothetical. If you want him to find out information that he didn't know, he can find out information he didn't know. I think you said about the sample size.

MR. TRUDEAU: (Nodding affirmatively.)

JUDGE FARRAR: To convert the hypothetical and do a real example, but I don't know -- Ms. Chancellor, let's be more specific about what you'd like to have him do. These borings are not a standard size?

MR. TRUDEAU: This is not a boring.

This was a soil specimen that was likely trimmed into a direct shear box. So we need the dimensions of the direct shear box, the size of the gap during the test.

JUDGE FARRAR: Do you want him to find out that information so you can ask him?

MS. CHANCELLOR: Your Honor, I'm sorry, I was being a little flip. I was referring to a question that I had asked earlier, whether he had evaluated the amount of horizontal displacement that is required to mobilize the peak undrained shear strength of the upper Bonneville clay, and Mr. Trudeau was having difficulty on the stand finding that and he was not going to take up our time and was going to look for that this evening. And so maybe that is a way in which the hypothetical may no longer become a hypothetical and may become a real case.

JUDGE FARRAR: Do you understand what she's asking you to do or what you volunteered to do, Mr. Trudeau?

MR. TRUDEAU: At the moment, no. If I could have the assignment in writing, it would help.

JUDGE FARRAR: Ms. Chancellor, if you'd

say again what you would like him to do.

Q. (By Ms. Chancellor) Right. It refers to a question I asked previously. Have you evaluated the amount of horizontal displacement that is required to mobilize the peak undrained shear strength of the upper Bonneville clay? And I'd like to know how much is this displacement?

MR. O'NEILL: In the context of the storage pads?

MS. CHANCELLOR: That's correct. .

JUDGE FARRAR: Do you understand the --

MR. TRUDEAU: Yes.

JUDGE FARRAR: And that's something you can do and bring back tomorrow?

MR. TRUDEAU: I believe so.

JUDGE FARRAR: Okay, thank you.

MR. TURK: May I ask approximately how long it would take you to perform that analysis and to get the data you need?

MR. TRUDEAU: I need to find out if the data is available in the SAR or not. I'm not sure. I may have to call Stoeten to get the information, in which case I can get it first thing in the morning.

MR. TURK: The reason I ask, Your Honor,

is we're going to be asking him to read a 100 plus page report overnight also, so that he can testify with some knowledge about the Luk report.

MS. CHANCELLOR: I mean --

JUDGE FARRAR: Well, he won't have to read the whole thing, probably.

MR. TURK: I don't know that.

JUDGE LAM: He's a fast reader.

MR. TURK: I don't know that.

JUDGE FARRAR: He can read enough to become familiar enough if he can't do all this, he can come back tomorrow and tell us he was unable to do it.

MS. CHANCELLOR: I believe for the displacement that we're asking for, Mr. Trudeau only needs to know the thickness of the direct shear box. Is that true, Mr. Trudeau?

MR. TRUDEAU: The shear strength during a direct shear test may also be a function of the gap during the test. These test results are reported in terms of horizontal displacements. I'm not sure right now how to get to horizontal strain from these results. So I'm going to need to rely on Dr. Chang again.

JUDGE FARRAR: Ms. Chancellor, we're

sort of at the point we set as a limit on ourselves. I also, since we're reaching a point of -- not diminishing returns, diminishing patience among the parties --

MS. CHANCELLOR: This is a good stopping point, Your Honor.

JUDGE FARRAR: I think that's where I was headed. Then we'll come back at nine tomorrow, but counsel had indicated this would be a good time to meet up here and start to discuss the future schedule, which we can do off the record and then put on the record tomorrow morning. So we'll go off the record now.

(The proceedings were concluded for the day at 5:25 p.m.)

CERTIFICATE

This is to certify that the attached proceedings before the United States Nuclear Regulatory Commission in the matter of:

Name of Proceeding: Private Fuel Storage, LLC

Docket Number: Docket No. 72-22-ISFSI

ASLBP No. 97-732-02-ISFSI

Location: Salt Lake City, Utah

were held as herein appears, and that this is the original transcript thereof for the file of the United States Nuclear Regulatory Commission taken by me and, thereafter reduced to typewriting by me or under the direction of the court reporting company, and that the transcript is a true and accurate record of the foregoing proceedings.

/s/ Diana Kent

Diana Kent
Official Reporter
Neal R. Gross & Co., Inc.