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NUCLEAR REGULATORY COMMISSION

Title: Private Fuel Storage, LLC

Docket Number: 72-22-ISFSI; ASLBP No. 97-732-02-ISFSI

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UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

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

U. S. Nuclear Regulatory Commission
Sheraton Hotel, Wasatch Room
Salt Lake City, Utah 84114

On May 2, 2002 the above-entitled matter came on for hearing, pursuant to notice, before:

MICHAEL C. FARRAR, CHAIRMAN
Administrative Judge
U. S. Nuclear Regulatory Commission

DR. JERRY R. KLINE
Administrative Judge
Atomic Safety & Licensing Board Panel

DR. PETER S. LAM
Administrative Judge
Atomic Safety & Licensing Board Panel

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I N D E X

E X A M I N A T I O N

E X H I B I T S

Wednesday, May 2, 2002 9:00 a.m.

P R O C E E D I N G S

JUDGE FARRAR: Good morning, everyone. Last night after we recessed the formal proceedings we had a long discussion, an hour or more with counsel, on how to finish up the hearing. I think that -- and we didn't come to final decisions but for everyone's information, it looks like we will go the six weeks here, ending around Friday, May 17; not do the proposed seventh week; reconvene here early in June for a week, and we had a little flexibility in that; and then if there was more to do it appeared it would not be an undue burden on the State, given the issues and the end of the school year, to do a final week in D. C. the week of -- going back to May, the week of May 13, the spillover week, we would do some seismic and attempt to finish up aircraft. So that was the basic plan. Have you all discussed that in any greater length since then?

MR. GAUKLER: No, we haven't, Your Honor.

JUDGE FARRAR: Did I describe the general plan accurately?

MR. GAUKLER: I believe you have.

JUDGE FARRAR: Then are there any preliminary matters before we resume the cross-examination of Mr. Trudeau? All right. Then the State may continue its cross-examination.

CROSS-EXAMINATION

BY MS. CHANCELLOR:

Q. Good morning, Mr. Trudeau.

A. Good morning, Ms. Chancellor.

Q. As I recall, there was a homework assignment?

A. Yes.

Q. Have you had a chance to evaluate the amount of horizontal displacement required to mobilize the peak and drain shear strength of the upper Bonneville clays?

A. Yes.

Q. And have you been able to conclude how much displacement there would be?

A. The displacement is shown on the plot you referred to on Page D-1/3 of Exhibit UU. It's approximately 0.025 inches.

Q. Could you go through that a little, Mr. Trudeau?

A. The direct shear test is a test to measure that displacement, and the results are plotted as a function of stress versus that horizontal displacement.

Q. And the number you came up with was 0. -- what was that again?

A. 0.025 inches, as shown on the plot to the peak.

Q. Okay, thank you, Mr. Trudeau. Do you know the static Youngs elastic modulus of soil cement?

A. We have not measured the static Youngs modulus yet. We have estimates of what it should be for the soil cement in our various calculations.

Q. And what is that estimate?

A. I'm sorry. For the soil cement or the cement-treated soil?

Q. Soil cement.

A. No, we have not estimated that for the soil cement. Not to my knowledge.

Q. Would the Youngs modulus of soil cement be significantly higher than that of the upper Bonneville clay?

A. Yes, I believe it would be.

Q. Any idea how much; 50 percent, 100

percent, higher?

A. It would be significantly higher. I don't know that it would be 50 or 100, but it would be a significant difference.

Q. If the soil cement is much -- if the soil cement plug between the pads carries a large amount of inertial load during reversal of an earthquake cycle, where would that load be transferred to?

A. A large inertial load for the soil cement plug? Is that what you said?

Q. That's right. Soil cement plug between the pads.

A. I'm not sure it's fair to characterize the inertial force attributable to the soil cement plug as large, especially compared with that of the pad. But the inertial force of the plug would be imposed on the -- well, on the underlying cement-treated soil, would be my guess, first. And if that wasn't strong enough to sustain that force, then it would be imparted onto the pad.

Q. To the adjacent pads?

A. That's correct. But it is my belief that it's smaller than the strength, the shear strength available at the base of the plug. So the plug will stay in place, as will the pad.

Q. And what will happen to the inertial loads from the adjacent pads?

A. The adjacent pad is bonded to the cement- treated soil layer as well as the pad that you were referring to the one being adjacent to. So they all stay in place. The shear strength at that interface is strong enough to keep the pad in place, keep the soil cement plug in place, and the shear strength at the base of the cement-treated soil is strong enough to keep both of those in place. So the whole row of pads will be bonded to the underlying clay soils.

Q. And you haven't done any testing yet of the interface between the soil cement and the pads; correct?

A. That is correct.

Q. On Answer 19, I believe on Page 19 you calculated a factor of safety for sliding of an entire row of ten pads running in the north/south direction; correct?

A. That is correct.

Q. What analysis have you done to conclude that that row of pads will act as an integrated unit during a design-basis earthquake?

A. The analysis results of which I just spoke; that the soil cement plug is bonded to the underlying cement-treated soil so that it will stay in place. The pad adjacent to that soil cement plug on either side will stay in place. So inasmuch as they are bonded at their bases, they are going to behave as a single unit. They don't rely on the connection between the pad and the soil cement for any integration, or whatever you want to call it.

Q. Does your opinion include the kinematic and inertial stresses that are due to soil structure interaction?

A. As I said yes today, yes, it includes those from the cask dynamic loads.

JUDGE FARRAR: Have we had a definition of kinematic on the record yet? If not, let's have one.

Q. That's to you, Mr. Trudeau, I believe.

A. Excuse me?

Q. Could you define "kinematic"?

A. When you say "kinematic", I'm assuming you are referring to the dynamic loads that would be the result of the soil structure interaction analysis.

JUDGE FARRAR: Did you say kine-- K-I-N-E- --

MS. CHANCELLOR: Yes.

JUDGE FARRAR: I'm sorry. I thought it might have been spelled differently and meant something different. Kinematic.

MS. CHANCELLOR: It's the accent. Sorry.

Q. (Ms. Chancellor) Have you considered soil structure interaction on the integrity of the soil cement and cement-treated soil?

A. Only inasmuch as the dynamic loads included the cask dynamic loads from the CEC analysis. And those loads were developed based on the time history of forces that they got from Holtec for the cask.

Q. So does this imply that you do not think that soil structure interaction effects are important to you?

A. As I said yesterday, we -- I used the peak ground acceleration for the buried pad in the soil cement layers in determining the dynamic loads. And as my testimony demonstrates, based on the plot of the factors of safety against sliding, based on Holtec's time history of forces for those springs, this was not an unreasonable assumption. The factor of safety that I came up with using the peak ground acceleration was 1.27. And the minimum, based on the peak force at one instant of time, in that time history resulted in 1.25 factor of safety. As you can see in that Exhibit UU, the average factor of safety for sliding for that pad is approximately ten.

Q. And isn't it true that you testified yesterday that you haven't completed any soil structure interaction analysis?

A. I haven't, no.

Q. Okay. Still on Question 19, the case of 20 pads in a row, what was the factor of safety against sliding if the pads were underlain by cohesionless, that is sandy or silty, material?

A. The results are shown on Page 42 of Exhibit UU. For this hypothetical case, all of the cohesion was ignored and only the frictional portion of the strength of the clay soils underlying the cement-treated soil are assumed to be available. And for that case the factor of safety was 0.44.

Q. And did the factor of safety involve sliding?

A. I would say it would if that were, indeed, the case. If those strengths were the correct strength, yes.

Q. Isn't it true that cohesionless material is found at about eight to ten feet below the current ground surface at the PFS site?

A. There are some soils at portions of the site that would or could be characterized as cohesionless soils. However, even those soils have some cementation that would impart a cohesive character to those soils. The factor of safety against sliding for those soils would be higher than the number that I just cited for the obviously conservative value of friction angle for the clay soils.

Q. Higher than 0.44. Is that what you are saying?

A. Yes.

Q. The foundation of the pads at the PFS site is on a concrete mat; correct?

A. Say that again, please?

Q. The pad foundation at the PFS site is a concrete mat; correct?

A. That is correct.

Q. And it is three-feet thick?

A. Correct.

Q. And it is embedded approximately three feet?

A. That's correct.

Q. So this would be considered a shallowly embedded foundation.

A. It's a shallow foundation, yes.

Q. Isn't it true that this is a very cheap or inexpensive design compared to deeply embedded foundation, with piles and that sort of thing?

A. Shallow foundations are cheaper than deep foundations such as piles, yes.

Q. Isn't it true that a shallowly embedded pad foundation of the pads does not have large capacity to resist sliding, overturning, and uplift as compared with a deeply embedded foundation?

A. In general, perhaps.

Q. Isn't it true that for the type of foundation that PFS proposes, that you must allow the cask to slide in order to reduce the inertial forces acting on the pad?

A. I don't think that that is a correct statement. The pads do slide, but whether they must slide or not, I'm not sure would affect the results that much.

MR. GAUKLER: Your Honor --

Q. Did I say the "pads" to slide? I meant the cask.

A. If I said pads, I meant casks, too.

Q. It's early in the morning for both of us.

JUDGE FARRAR: Let's start again and ask the question the way you want to so we are sure we will have the right question and answer.

Q. Isn't it true that for the type of foundation that PFS proposed, that you must allow the cask to slide in order to reduce the inertial forces acting on the pad and still meet your factor of safety of 1.1?

A. I don't think I can agree to that. I can say that there is a section of my calc where I recognize that the cask forces imparted to the top of the pad are reduced to that value. But I'm not -- I don't have the information to conclude that we would not have a factor of safety of 1.1 if I had not taken that known reduction that would be available.

Q. But it is part of PFS's design that the casks are allowed to slide and thus will reduce the inertial forces acting on the pad?

A. That is correct.

JUDGE FARRAR: Ms. Chancellor, could I follow up on that for a moment?

MS. CHANCELLOR: Certainly. Any time, Your Honor.

JUDGE FARRAR: If I heard the question right, the alternative to that would be, as has been suggested for Diablo Canyon, bolting the casks to the pad. And I take it that was the point of the question; that if you bolted the casks to the pad then you'd have more forces affecting the pad.

THE WITNESS: That's the way I understood it. Whether they were bolted or just had a higher coefficient of friction than .8.

JUDGE FARRAR: Okay.

Q. (Ms. Chancellor) On Page 13 of the calculation, Exhibit UU, the sliding analysis for the pads --

A. That was 13?

Q. Yes; one, three. It's "Overturning Stability of the Cask Storage Pad", on page 13.

A. Yes.

Q. And here you have calculated a factor of safety against sliding, against overturning, for the storage pads; correct?

A. Correct.

Q. And in calculating the driving moment of the casks -- let me just go through this. So if you look at the caption "Overturning Stability of the Cask Storage Pad", you'll have the factor of safety against overturning is defined as FS sub OT equals the sum of the --

A. Resisting moment.

Q. Resisting moment plus the sum of the driving moment. Is that correct?

A. Yes. Divided by.

Q. Divided by. I know a division sign.

In calculating the driving moment of the casks you assume that sliding of the casks would occur; correct?

A. I assume that there was no moment connection between the base of the cask and the top of the pad because they are not anchored there. Therefore, I place the horizontal dynamic loads from the casks at the top of the pad.

Q. And you did assume sliding; correct?

A. I'm sorry?

Q. You did assume sliding with respect to the horizontal driving forces you calculated, the 696 kips; correct? Towards the bottom of the page.

"When the vertical inertial force --"

A. I see where you are talking.

Q. It's the last paragraph.

A. Okay.

Q. I was just helping the others find where it is. It's the last paragraph on Page 13; correct?

A. That is correct.

Q. So assuming sliding when the earthquake horizontal driving force you calculated was 696 kps; correct?

A. This is correct. The uplift from the earthquake reduces the normal force of the cask applied to the pad. This is the worst case for overturning because it reduces the resisting moment as well. And for that case, .8 times the resultant normal, which is the coefficient of friction for sliding between the cask and the pad, results in that 696 kip value.

Q. And what is the moment arm that you use to calculate the overturning moment for the casks?

A. I used half the thickness of the pad. Half the height of the pad, I mean.

Q. So the moment arm is three feet? Is that correct?

A. It was one and a half -- I'm sorry. Excuse me. You are right. I used the full height of the pad. It's three feet.

Q. Is the overturning moment you calculated for this case equal to three feet times 696 kips, or 2088 kip feet?

A. Yes.

Q. How did you calculate the 696 kips driving force? Excuse me. Horizontal force.

A. That calculation is shown in a table at the bottom of Page 21, the Case III line that is indicated as uplift.

Q. Does this value presume the casks are sliding?

A. The 696 does, yes.

Q. Where did you apply the horizontal overturning force? Is that applied at the base of the cask? Is that applied at the top of the pad?

A. For the cask load, yes.

Q. And is that because sliding has been initiated that you applied it there?

A. Yes.

Q. Do you consider this to be the most critical case for calculating the overturning moment due to the casks?

A. Yes.

Q. Do you agree, for the case you analyzed, that sliding is initiated after the horizontal driving force reaches 696 kips, based on a coefficient of sliding friction of 0.8?

A. I'm sorry. Would you repeat that please?

Q. Certainly. Do you agree, for the case you analyzed, that sliding is initiated after the horizontal driving force reaches 696 kips, based on a coefficient of sliding friction of 0.8?

A. Yes. That's correct.

Q. Do you agree that for the case you analyzed, the .8 coefficient of friction, that sliding may not be initiated if the horizontal force is less than 696 kips?

A. If the horizontal force is less than 696 kips, then my driving moment decreases. On the bottom of Page 13, where it says three times 696, it will equal three times some number less than 696. That result is in the denominator of the overturning factor of safety calculation. So the factor of safety would go up.

Q. That's not quite my question. The question is that if the horizontal driving force is less than 696 kips, isn't it true that sliding may not be initiated before you reach or before the horizontal driving force reaches 696 kips?

A. Yes.

Q. Okay. We are taking this bit by bit.

When the horizontal driving force is slightly less than 696 kips and sliding has not initiated, isn't it true that the moment arm for the horizontal driving force of the casks acts at the center of mass of the casks?

A. Yes.

Q. And that moment arm would be the three-foot thick pad plus the about twenty-foot cask. So that would be divided by two, that would be about 13 feet. Correct?

A. Correct.

Q. Above the base of the pad. So 690 kips would be slightly less than the value required to cause sliding; correct?

A. Yes.

Q. Okay. And the resulting driving -- the resulting overturning moment due to the cask when using 690 kips would be 13 times -- 13 feet times 690 kips; correct?

A. That's correct.

Q. And you can check this on a calculator, that would be 8970 kip feet; 13 times 690?

A. Correct.

Q. Isn't this overturning moment significantly higher, the 8970, than the 2080 kip feet that you used in your calculation?

A. Yes.

Q. Is it fair to say, then, that the calculation you have presented for overturning of the pads did not include this case?

A. Yes.

Q. Isn't it fair to say the factor of safety against overturning is significantly less than presented on Page 13 of Exhibit UU?

A. Yes. But it would still be greater than the 1.1 criterion.

Q. What have you done to validate the assumption of sliding at .8 times the normal stress at the base of the casks?

A. That was one of the inputs that I was provided as part of the criteria for these casks.

Q. Is this provided to you by Holtec?

A. The project, whether it came from Holtec or from the project, I'm not sure. But it's been the criteria that these cask-to-pad forces would be determined for both the .2 and the .8 ends of the range to cover the full range of credible coefficients of friction.

Q. So you just accepted that assumption, correct? You didn't validate it?

A. Correct.

Q. Did you evaluate potential for cold bonding of the cask with the pad and how that may impact sliding of casks?

A. No.

Q. Are you aware that Dr. Wen Tseng testified that the dynamic deflections he has calculated for the pads in the ICEC calculation Private Fuel Storage, PFS storage pad design provision 3 of 234, that he has testified that they do not include any deflections resulting from short- or long-term settlement?

MR. TRAVIESO-DIAZ: Objection, Your Honor, I want to reserve my objections to any questions concerning long-term settling.

JUDGE FARRAR: Same ruling. You may answer.

A. Could I have the question read back, please?

Q. Are you aware that Dr. Wen Tseng, in the

ICEC calculation, did not include any deflections resulting from short or long term settlement of the pad?

A. Am I aware that he did not include it in his calc? Is what that you are asking?

Q. That's correct.

A. I'm not intimately familiar with his calc, but if I'm not mistaken he does include in his calc some coefficients of subgrade reaction values that he received from me, that would have been an indication of short-term settlements of that pad as part of his design of the pad.

Q. Have you calculated the deflections in the pad that result from short-term and long-term settlement?

A. No.

Q. Have you considered how deflections in the pad due to dynamic and short-term and long-term deformations may impact the top surface of the pads?

A. No.

Q. And how those deflections may impact sliding?

A. No.

Q. Dr. Tseng, in his testimony, said that the force time histories of the cask indicate there are times when the casks are uplifted along one edge. Were you here for that testimony?

A. I'm not -- I was here for much of the testimony, but I don't recall that.

Q. Did you consider how the partial uplift may concentrate stresses and affect sliding?

A. No.

Q. Have you included possible imperfections of the pad's surface resulting from construction imperfections and how that may affect sliding?

A. No.

Q. Have you evaluated the potential impacts if the casks don't slide as you assumed at the horizontal driving force of 696 kips?

A. Have I --

Q. Evaluated the potential impacts if the casks do not slide as you assumed at a horizontal driving force of 696 kips?

A. No.

Q. If the casks do not slide and the full inertial loads were allowed to act in a horizontal direction, wouldn't there be the potential for overturning of the pads and the cask system?

A. I don't believe so. The cask is not

bonded to the top of this pad. In my estimation, that cask would have to overturn before the pad cask system could overturn. It's my understanding that Holtec has demonstrated that the casks do not overturn.

Q. Let's go back to your calculation on Page 13. Did you assume there the horizontal acceleration of the pads is 0.711g?

A. Yes.

Q. What is the basis of that assumption?

A. That is the peak ground acceleration in the horizontal direction.

Q. Did you evaluate the pads from any potential soil structure interaction effects and how those effects may change the acceleration of the pads?

MR. DIAZ: Excuse me, Your Honor. I have been quiet about this but this is, by my count, the seventh time she asked the same question. I think the record is being burdened by this. I think we are willing to stipulate that Mr. Trudeau, in his calculations, did not include soil structure interaction.

MS. CHANCELLOR: That's fine, Mr. Diaz, but it is not necessary. I will move on.

Q. (Ms. Chancellor) The equation on Page 13 -- just one second.

In your calculation after you've got the sum of the resisting moment divided by the sum of the driving moment, you've got Wp times Wc B/2 (1 - a sub v). Correct?

A. This is on Page 13?

Q. This is on Page 13.

MR. TURK: I think we have --

JUDGE FARRAR: It's not Wp times Wc. It is Wp plus Wc.

MS. CHANCELLOR: There's no plus sign in there.

JUDGE FARRAR: There's a plus down when they do or when he does the calculation.

MS. CHANCELLOR: Okay. I was just reading the text. That's correct.

Q. (Ms. Chancellor) What does the term Wp mean?

A. It's the weight of the pad.

Q. And Wc?

A. The weight of the casks.

Q. And B?

A. B?

Q. B as in boy.

A. That's the width of the pad in the short direction.

Q. And that is 15 feet; correct?

A. No. That is 30 feet.

Q. Divided by 2 is 15?

A. Correct.

Q. I got that one. And A sub V is the acceleration in the vertical direction?

A. Correct.

Q. By this equation, what happens to the resisting moment when the vertical acceleration A sub V approaches a value of 1g?

A. The resisting moment goes to zero.

Q. You have used a value of 0.659g to calculate the resisting moment; correct?

A. No. I have used 0.695.

Q. What did I say? Didn't I say that?

A. You said 0.659 .

Q. Oh, .695.

A. The correct number is 0.695.

Q. Did you consider the potential amplification of the vertical acceleration of the pads?

A. No.

Q. Did you consider resonance or any

increase in acceleration occurring from the pad cask system?

A. No.

Q. Are you aware that such an analysis has been performed for the CTB?

A. For the what?

Q. Are you aware such an analysis, the resonance, has been performed for the CTB?

A. For the CTB?

Q. Right. Let's go to the CTB Table 2.6-11. CTB calculation.

JUDGE FARRAR: And where is that?

MS. CHANCELLOR: I beg your pardon. That's on Exhibit VV. It's on Page 69 of the CTB cask.

MR. TURK: Is that part of the original VV or the --

MS. CHANCELLOR: No. It's the one that Mr. Travieso-Diaz handed out. The back half of the calculation.

MR. TURK: May we go off the record for a moment, Your Honor?

JUDGE FARRAR: Yes.

(Discussion off the record.)

Q. Do you have the page, Mr. Trudeau?

THE JUDGE: The page again?

Q. It's page 69 of Exhibit VV.

JUDGE FARRAR: The expanded replaced version.

Q. That's correct. The one handed out yesterday. If you look at the vertical acceleration, A sub v, for the CTB, does this show that the vertical acceleration at the bottom, at the soil, bottom of the mat has increased from 0.695g to 0.92g?

A. No.

Q. Okay. What does the vertical acceleration at the mat elevation show on Page 69 of the CTB calculation?

A. That is shown in the column labelled Ay. It is 0.78g. Perhaps the confusion would be cleared up if you looked at Page 52 which shows the coordinate system; y is the vertical direction.

Q. It does show some amplification, correct, in the CTB?

A. A little, yes. We discussed this yesterday. This CTB is a five-story building that sits way above the ground. The pad is three feet and buried. So it is not unusual to expect that the amplification due to soil structure would be different for those two distinctly different structures.

Q. In Answer 30 of your testimony on Pages 17 and 18, and also I'll be referring to Pages 46-51 of the pad analysis calculation, PFS Exhibit UU.

MR. TRAVIESO-DIAZ: Do you mind going through those pages again? I lost it.

MS. CHANCELLOR: Sure. It's Answer 30 on Pages 17 and 18 of Mr. Trudeau's testimony. And on the pad sliding analysis, Pages 46 through 51.

MR. TRAVIESO-DIAZ: Thank you.

MS. CHANCELLOR: You're welcome.

JUDGE FARRAR: 46 through 51 of --

MS. CHANCELLOR: Of PFS Exhibit UU which is the pad stability analysis.

Q. (Ms. Chancellor) Isn't it true that on Pages 46 through 51 of PFS Exhibit UU, that you have included the Newmark sliding block analysis in an attempt to show acceptable deformation for the case of sliding on a deeper sandy, silty layer?

MR. TRAVIESO-DIAZ: Excuse me. I object to the characterization of the calculation as an attempt.

JUDGE FARRAR: As an attempt?

Q. (Ms. Chancellor) In order to show --

MR. TRAVIESO-DIAZ: All right.

Q. (Ms. Chancellor) -- acceptable deformation for the case of sliding on a deeper sandy, silty layer?

A. What is actually analyzed here is a hypothetical case that those deeper cohesionless soils might actually exist at the bottom of the pad.

Q. And this deeper layer is approximately eight to ten feet. Correct?

A. I believe that's --

Q. And the factor of safety against sliding is less than 1.1 for this case. Is that correct?

A. That is correct.

Q. Doesn't the simplified Newmark sliding block analysis assume the block is rigid?

A. I believe that's correct.

Q. And you've used this Newmark sliding block analysis to show that the analysis is acceptable.

A. That the displacements were minimal. And in that sense there are no safety-related connections to these structures; that it's not a concern.

Q. Does the simplified Newmark sliding block analysis account for vertical acceleration in calculating the potential displacement?

A. Yes.

Q. Did you --

MR. TURK: I missed the last question? Could I hear the question again?

Q. Sure. Does the simplified Newmark sliding block analysis account for vertical acceleration in calculating the potential displacement?

A. Yes.

Q. Did you use peak horizontal ground acceleration in calculating the maximum resistance coefficient?

A. I believe so.

Q. In performing the Newmark sliding block analysis, did you consider the potential for pad-to- pad interaction?

A. Did I consider the potential for pad-to-pad interaction in what?

Q. In performing the Newmark sliding block analysis?

A. No.

JUDGE FARRAR: Ms. "Chancellor, let me

back up a question. Is "I believe so" slightly different from "yes".

THE WITNESS: I think the answer is yes, but it's been a while since I have looked at this. If you would want me to spend a little time, I can confirm that.

JUDGE FARRAR: I will leave that to Counsel as to the significance of the answer.

MS. CHANCELLOR: We will take Mr. Trudeau's representation. That's fine, Your Honor.

THE JUDGE: Well, his representation is he thinks so, but he is not sure. In other words, his representation, and I'm not criticizing him for it, but his representation is he is 95 percent sure --

MS. CHANCELLOR: That's fine. If we have any problems, we can have Dr. Bartlett clarify that later.

JUDGE FARRAR: Okay.

Q. (Ms. Chancellor) Did the sliding Newmark sliding block analysis account for fault directivity or what is sometimes referred to as fault fling?

A. No.

Q. Isn't it true the Newmark technique was developed from acceleration time histories that were normalized to peak horizontal ground acceleration of 0.5 G?

A. Yes.

Q. And isn't it true that you used peak ground acceleration at this site of 0.711g?

A. That is correct.

Q. I'd like to turn, now, to the canister transfer building. In your testimony in Answers 33 through 37 you discuss the sliding calculation for the CTB; correct? Is this in -- the sliding calculations for the CTB, canister transfer building, is PFS Exhibit VV. It's just a general question. Take your time.

A. I guess they do all apply to the sliding in the CTB.

Q. Okay. And you also, in those same answers, you discuss the potential for cracking of soil cement buttress from settlement?

A. I'm sorry. I didn't catch the question. I also do what?

Q. In those answers you also discuss the potential for cracking of the soil cement buttress from settlement?

A. Yes.

Q. And from other factors. Right? And how this may affect sliding of the CTB. Just a general overview of these answers. Is that correct?

A. Yes.

Q. What is the total settlement estimated for the CTB resulting from short-term and long-term settlement?

A. I believe it's on the order of three inches, near the center of the building.

Q. Was this three-inch settlement taken into consideration in the structural design of the mat foundation for the CTB?

A. I don't know.

Q. Do you know whether there is a final calculation for the structural design of the CTB mat foundation?

A. I believe there is.

Q. Has it been submitted to the NRC?

A. I don't know.

Q. Who is responsible for that calculation?

A. I assume our structural department; probably Bruce Ebbeson, who will be here later.

Q. Do you know whether the three-inch total settlement estimated for the CTB, whether that was taken into account when designing the soil cement buttress around the CTB?

A. It does not affect the soil cement buttress so it was not taken into consideration in the design of the soil cement buttress.

Q. In Answer 35, you state that the stress distributional loading of the CTB is distributed over a wider area of the soil profile.

A. As you go deeper in the profile, yes.

Q. Have you calculated the 2-D stress distribution to the CTB and the adjacent soil cement buttress?

A. The 2D stress distribution is calculated as part of the settlement analysis. And as I said earlier, it doesn't affect the soil cement buttress.

Q. What is the depth of the top of the upper Bonneville clay in relation to the bottom of the CTB mat?

A. The mat is founded on that clay.

Q. And what is the thickness of the Bonneville clays at the PFS site?

A. The upper layer is like eight to twelve feet thick, I'd say. And there are other Bonneville deposits that constitute the top 30 feet, another clayey layer down near the bottom of that upper 30 feet, and in between there's some silts that are sandy silts, on occasion.

Q. Is the upper Bonneville clay the most compressible layer in the soil profile at the PFS site?

A. I believe so.

Q. Have you performed tensile strength testing on the soil cement to determine the tensile capacity of the soil cement buttress?

A. No. Tensile strength does not affect passive strength. Passive resistance is a compressive strength.

Q. Is it your testimony that there will be no cracking of the soil cement buttress around the CTB due to shrinkage and curing of the soil cement?

A. No.

Q. The CTB is a relatively large and heavy building; correct?

A. Correct.

Q. What is the loading or varying pressures for the soils in kips per square feet?

A. I believe it is a little less than one and a half kips per square feet.

Q. What is the maximal horizontal acceleration response of the CTB mat as given to you by the structural mechanical group?

A. You are looking for an acceleration value?

Q. That's correct.

A. That value is 1.047g from Page 49.

Q. And this is the value in the Ax direction; correct?

A. Correct.

Q. What is the free field peak horizontal ground acceleration response of the adjacent soil cement buttress?

A. .71g.

Q. It's correct to say, isn't it, that there's a significant difference between the maximum horizontal response of the CTB of 1.047g and the free field acceleration of 0.711g; correct?

A. There is some difference.

Q. Is that difference about 47 percent?

A. Yes.

Q. Have you analyzed the inertial affects or interactions resulting from these very different accelerations and the large mass of the CTB interacting with the soil cement?

A. No.

Q. Have you analyzed what effects these two very different accelerations will have on the integrity of the soil cement buttress?

A. No. The accelerations in the vicinity of the structure should be closer to the value for the structure than out in the free field, however. So it's not like there will be an abrupt change right at the structure, in my opinion.

Q. Have you analyzed for potential out-of-phase motion resulting from this inertial interaction?

A. No.

Q. Is the soil cement buttress structurally tied to the CTB mat foundation?

A. No.

Q. Have you quantified the amount of separation that will occur as a result of this interaction?

A. No.

Q. Relatively speaking, how stiff is the CTB mat compared to the adjacent soil cement buttress?

A. It is much stiffer.

Q. A factor of 50, 100? Any idea?

A. I would say at least 10 times, but I'm not sure it is 50 or 100 times.

Q. Have you evaluated -- whatever that stiffness contrast is, have you evaluated how it will affect the integrity of the soil cement buttress under dynamic loading?

A. No. The soil cement is strong enough to resist the horizontal loads to be applied by the canister transfer building to it. The stiffness of the soil cement is such that it will minimize the movement of the canister transfer building because it is much more stiff than the soil that would normally be used for this purpose.

Q. And this would be true for 2000 year design basis earthquake.

A. That's what our design basis is, yes.

Q. Not for a 10,000 year earthquake?

A. Well, the principles that I'm describing are true.

Q. So you're saying that the integrity of the soil cement buttress under dynamic -- that there wouldn't be any effect from a 10,000 year earthquake?

A. I don't believe I have a sliding calc for the CTB for the 10,000 year earthquake in my calculation.

Q. Have you calculated the amount of

bending and torsion that will be introduced into the soil cement buttress as it interacts dynamically with the CTB mat foundation --

A. No.

Q. -- for the base case?

A. No.

Q. Have you analyzed the amount of bending that will be introduced in the soil cement buttress due to short- and long-term settlement?

A. No.

Q. In Answer 37, you state that theoretically the CTB might move a small distance; correct?

A. Correct.

Q. If it does so, has it met the requirement to have a factor of safety against sliding of greater than 1.1 in that instance?

A. If this movement was the result of strains associated with reaching a passive condition, I would say yes.

Q. So sliding is acceptable for meeting a factor of safety of 1.1?

A. Let me back up. In that case it wouldn't have been sliding. It would have been in response to elastic deformations to build up the strain required to reach passive resistance. So even in that case, it wouldn't be true sliding. It would be a horizontal displacement. But the soil column underneath it is straining, elastically, to reach that passive resistance required.

Q. So there's sliding and then there's other sorts of sliding?

A. Well, it's not another sort of sliding. It's a translation -- it's a shearing of the soil column. The soil column is deformable and when the load is removed, it rebounds elastically. Believe you find a discussion of that in the calc for the CTB.

Q. Based on your experience, Mr. Trudeau, can you cite one nuclear facility for which soil cement has been used to resist seismic loadings from the structure?

A. We cited several in our SAR. The one that is notable is the one that used soil cement to preclude liquefaction during an earthquake at a nuclear power plant in South Africa.

Q. And liquefaction is different than horizontal resistance to sliding; correct?

A. Yes. It's a different mechanism.

Q. Can you tell us of any nuclear project with similar design notions or intensity as PFS whose structures have a shallow or no embedment such as the pads at the PFS site?

MR. TRAVIESO-DIAZ: Your Honor, again we have probably in the vicinity of 60 pages of testimony on soil cement on Section C. All of the issues are prominently displayed there and we may be starting to waste time to go through this here today.

MS. CHANCELLOR: This is not soil cement, Mr. Travieso-Diaz. This is shallow embedment of the foundation.

MR. TRAVIESO-DIAZ: Then I must have mis- heard your question. I thought I heard you talk about facilities using soil cement.

MS. CHANCELLOR: That was the previous question.

MR. TRAVIESO-DIAZ: I apologize.

JUDGE FARRAR: Let's read it back.

(The record was read as follows:

"Can you tell us of any nuclear project with similar design notions or intensity as PFS whose structures have a shallow or no embedment such as the pads at the PFS site?")

MR. TRAVIESO-DIAZ: I withdraw my

objection.

JUDGE FARRAR: Thank you.

A. I'm not familiar with other nuclear facilities with ground motions comparable to these.

Q. (By Ms. Chancellor) And you testified that the CTB mat would settle approximately three inches. How many nuclear buildings are designed for up to three inches of settlement?

A. That's not an uncommon settlement for large, mat-supported buildings such as this.

Q. A nuclear related facility?

A. That's correct. And those structures are loaded far greater than this structure is. I mean 8 ksf is a typical loading for a reactor building for instance.

Q. Do you know of any nuclear facilities that, during its design, it was discovered that a major active seismic fault dipped below the site yet the design of PFS essentially remained unchanged?

MR. TRAVIESO-DIAZ: I have to object here. That is parts one and two of Contention L. I think we are straying far afield.

JUDGE FARRAR: I think we are getting near the end so I will overrule the objection, but let's make sure we keep on point here.

A. I don't know of any other nuclear facilities that meet that criterion.

Q. And the PFS design, except for the structural soil cement element, essentially has not changed after PFS discovered that that major active fault dipped under the site; is that correct?

A. I don't believe so. I believe that the earthquake changed as a result of that fault study, and soil cement was added to address that. The structure, the CTB structure, has gotten much larger than the original intention specifically to address those larger accelerations.

Q. And the same for the pads; the only difference in the pad design was to add some soil cement or cement-treated soil?

A. Well, the soil cement is a much improved product. It is a good engineered product for resisting these forces from these earthquakes.

Q. Okay. Thank you, Mr. Trudeau. I have no questions.

JUDGE FARRAR: You did say in the immediately previous answer that the earthquake did change or did not change?

THE WITNESS: It has changed, yes.

JUDGE FARRAR: I just didn't hear what you said.

(Board conferred off the record.)

JUDGE FARRAR: The Board will have some questions. Dr. Lam?

JUDGE LAM: Mr. Trudeau, good morning.

THE WITNESS: Good morning.

JUDGE LAM: In your pre-filed testimony, answer to Questions 8 and 9.

THE WITNESS: Yes.

JUDGE LAM: My understanding of what you were saying there is that the factor of safety of 1.1 represents a margin of about 10 percent. Is that correct?

THE WITNESS: That is correct.

JUDGE LAM: And also you further indicated that this is a desirable level because of the reversal of acceleration forces and also the short time duration that peak acceleration would occur. Is that a correct reading?

THE WITNESS: Yes.

JUDGE LAM: Now, given there are substantial uncertainties involved in any seismic analysis, and given there's only a ten percent margin of safety here, I'd like to hear your opinion on the adequacy of this 1.1 factor of safety. To provide you with some background, most of the factors of safety that I'm aware of in other industries involve a substantially higher value. Would you elaborate a little bit on the adequacy of 1.1?

THE WITNESS: I think that the point is very distinctly demonstrated by Exhibit WW. This is a plot of the factor of safety against sliding of the pad, using the time history of forces from Holtec's analysis of the pad cask system, the soil structure interaction analysis that they did. And you can see that as the accelerations change during the earthquake, the factor of safety changes. And it's vividly evident by this plot that the average factor of safety during this whole event is a number that is much, much greater than the ten percent margin. It is close to ten, in this particular instance, for the worst case soil properties.

In addition, I might add that this is not unusual or unexpected. The standard review plan of the NRC finds that this is an acceptable margin for loadings that include the design-basis earthquake loadings.

JUDGE LAM: Yes, indeed. I am aware that you had plotted the time history of the factor of safety. So most of the time you don't even go near 1.1. But my question really has to be of, look, in this analysis by having a factor of safety of 1.1 you permit the design or the structure of consideration to approach a value that is only 10 percent margin.

THE WITNESS: For one instant in time.

JUDGE LAM: Right. For one instant in time. Now, let me give you an analysis. Let us say there's a cliff. My factor of safety is how close am I to the cliff? Now, if I'm a mile from the cliff, I could approach it with another 100 feet, 200 feet. Now, if I only permit ten inches of separation between me and the cliff, even though time of consideration is instantaneous, if I get closer and closer to that, ten percent margin may or may not be sufficient. Assuming -- let's not talk about me. You might fall off the cliff. So be it. But let's talk about something more precious. Your grandson, or my grandson. Would you permit him to get closer and closer to the cliff even for half a second? Is that a reasonable approach to this?

THE WITNESS: I don't think so, in that the soils don't respond that rapidly. So that by the time this factor of safety gets down to this 1.21 for this peak force from this time history of forces, the force has gone in the other direction so it has re-righted the structure, so to speak, in less than a fraction of a second. I mean it is .005 seconds of the time step for this time history.

JUDGE LAM: But would that small fraction of time precipitate a failure? For example with a large building --

THE WITNESS: It certainly won't for a sliding situation or a bearing capacity situation or an overturning situation, the three forms of stability analyses that we are concerned with geotechnically.

JUDGE LAM: Why wouldn't it be --

THE WITNESS: Unless you had a situation where the soil strengths were somehow reduced as you got to that point. And in that case, then reversal of the forces might not be sufficient to preclude a failure. But we don't have that situation here at the PFS site.

JUDGE LAM: Why would it not apply to

the situation where overturning is an issue? Let's say I violate a factor of safety.

THE WITNESS: Because it would take longer than even a few seconds for that physically to occur. I mean, that is hundreds of times longer than the earthquake has had a chance to reverse direction, you know, several hundreds of times, in the time required for that to happen. It just physically can't instantaneously overturn when that factor of safety point, at .005 second interval, occurs. You would need to have a long period where the factor of safety was lower than 1 in order for that to happen.

JUDGE LAM: So you basically are describing the phenomena of impulse loading.

THE WITNESS: I believe that's a fair statement, yes.

JUDGE LAM: The next question, I have, Mr. Trudeau, is to follow up on Ms. Chancellor's questions on soil cement. Now, in your testimony, in Exhibit UU, you mentioned that this concept or you frame it as this engineering feature has been looked at since 1976. Just how extensively has soil cement been used?

THE WITNESS: Soil cement has been used

since the early 1900s. I'm not recognizing the 1976 reference that you are citing.

JUDGE LAM: In Exhibit UU, Page 24. Mr. DeGroot, 1976.

THE WITNESS: The DeGroot study was published at that time. But soil cement itself has been used since the early 1900s.

JUDGE LAM: I see.

THE WITNESS: Especially for pavements, road pavement construction in areas where they don't have adequate quality aggregates for construction of road bases. So it's been used for nearly a century.

JUDGE LAM: So there has been extensive data collected.

THE WITNESS: Correct.

JUDGE LAM: Now, for the sake of discussion here, isn't it true there are other alternatives for the Applicant? Instead of using soil cement, if the pad thickness were to be increased that would have solved the same problem, wouldn't it?

THE WITNESS: Increasing the pad thickness was not an option because of the cask, the nonmechanistic cask tipover analysis. That would have caused the decelerations of the cask, when it hit that stiffer pad, to be too high.

JUDGE LAM: I see. So using the soil cement involves a little bit more than cost consideration here.

THE WITNESS: Correct.

JUDGE LAM: Thank you, Mr. Trudeau.

THE WITNESS: You are welcome.

JUDGE FARRAR: Did I understand that answer to be a little more than cost consideration or a lot more?

THE WITNESS: For the soil cement?

JUDGE FARRAR: Yes.

THE WITNESS: The soil cement is an excellent means of bonding these pads to the surface of this clay layer. You will hear more about this next, when we get to Part C, I'm sure. But this upper Bonneville clay layer here at Skull Valley is much different than the Bonneville clays here in the Salt Lake area because it is a partially saturated soil and it is a stiff clay. We keep referring to having tested the weakest zone, the softer zone. But this is a competent clay at the site. And we are using the soil cement sort of as a glue to insure that we can retain the strength of that clay, the surface of the clay. And we know we are going to get a good bond because of the cementitious nature of the cement- treated soil and the concrete. DeGroot indicates that these bond interface strengths are very easily obtained. So I'm confident that this is a good way to make this site work and to preclude sliding of these pads during an earthquake.

Even if you don't assume that the cohesive strength that we are relying on will stick the pads to the clay, they are not going to go anywhere. I mean, they may move a few inches, but such a movement has been demonstrated as to be beneficial to the cask excursions. So I think we have a good design here that will prove to work out fine.

JUDGE FARRAR: What alternative -- when you were faced with this problem, what alternatives did you consider?

THE WITNESS: The obvious alternative is to put a structural fill beneath the pads. But because of the high earthquake loads, you can't get enough frictional resistance between the concrete and the underlying clayey soils to resist slide.

JUDGE FARRAR: When you say "structural

fill", what does that mean?

THE WITNESS: A compacted granular earth fill that would have been imported from somewhere. And I understand that that has great considerations in the environmental aspects of all these truck trips moving that material to the site, and the problems of wasting the materials to get this eolian silt layer off the site. So here we have a way of using the eolian silt in a way that increases its strength so it will be even stronger than these competent Bonneville clays and stick the pads to the surface of the clay at the site.

JUDGE FARRAR: Were any other alternatives given serious consideration?

THE WITNESS: You could go to a deeper foundation, perhaps. But pile foundations in nuclear structures, there's a lot of calculational or -- it's a difficult analysis to try to justify to the NRC that these piles are going to work the way you want them to.

JUDGE FARRAR: "Piles" meaning --

THE WITNESS: Pile foundations down into the denser sand and gravel layer, perhaps.

JUDGE FARRAR: Driving steel I-beams?

THE WITNESS: That's correct. Or

concrete-filled pipe piles or caissons. Other options available would include putting in what they call Franki piles, which are a pressure-injected concrete footing type of a -- let me back up. That is probably not a good option here because it doesn't sustain a lot of lateral loads, and lateral loads were a consideration. So a pile foundation is just not warranted because this method is a good method for making the site work.

JUDGE FARRAR: Getting back to the factor of safety. If you look at Exhibit WW, how many peaks are there for every five seconds.

THE WITNESS: The time step here is .005 seconds.

JUDGE FARRAR: And the ten percent factor of safety struck me as minimal compared to things I think I have heard in my lifetime about other significant structures, or airplanes or things that are important that they work. You started to answer that in response to Judge Lam's question, but could you elaborate on how that compares with the norm in other areas?

THE WITNESS: Well, for seismic loadings, this is the norm, as evidenced by the

Standard Review Plan of the NRC. That wouldn't be there if it wasn't a reasonable, conservative number. And it is conservative because of all of the other conservativisms that go into the analysis.

JUDGE FARRAR: But it is less than -- it is less than you do for bridges or airplane wings or --

THE WITNESS: Not for seismic loadings, no. Not for the earthquake loadings.

JUDGE FARRAR: Okay. New subject. In answer to Ms. Chancellor, I think you said there were no active faults right under other nuclear facilities. Did you exclude Diablo Canyon, because it's not right under --

THE WITNESS: Excuse me. What I meant to say was that I'm not familiar with other nuclear sites well enough to be able to say whether there are such facilities.

JUDGE FARRAR: Oh. I thought you said there were none.

THE WITNESS: No. I don't know of any, but that doesn't mean there are none.

JUDGE FARRAR: Okay. Just to clarify the record, Diablo Canyon and North Anna came to mind but Diablo Canyon is not right under the site, and North Anna is not capable, if I recall correctly. But if any other witness wants to address that question at the appropriate time, you are free to do so.

Ms. Chancellor, before we turn to redirect by the Applicant, did the Board's questions lead you to want to ask any more questions on cross so we can try to do this in one round rather than several?

MS. CHANCELLOR: Yes, they did, Your Honor. I should proceed?

JUDGE FARRAR: Yes.

MS. CHANCELLOR: I thought I had a few more minutes. Just a second.

MR. TURK: May we take a few minutes?

MS. CHANCELLOR: May we take a quick break?

JUDGE FARRAR: It's 10:28. Let's come back at 10:40.

MR. TURK: The Staff will have follow-up on questions, also, and we propose to introduce the SRP the witness has been referring to. We have copies here.

(A break was held.)

JUDGE FARRAR: Ms. Chancellor, we were going to give you a further opportunity at cross to follow up on the Board's questions before we go to redirect.

MR. TURK: Were you going to do the Staff's cross, also, or do you want us to come back after the stat?

JUDGE FARRAR: No. This was, given the nature of our questions, we were inserting another step to let the State do further initial cross, and then we would go to the redirect and the Staff's recross.

MR. TURK: Thank you.

JUDGE FARRAR: Apparently we have one more question that you'll want to wait for.

Judge Kline: During the break we got into a discussion as to the definition of "turnover", and I'm wondering if you can just give us that.

THE WITNESS: Overturning --

JUDGE KLINE: Overturning, yes.

THE WITNESS: Overturning of a structure would be a global type failure where it rotates as a rigid body about one of the edges of the structure.

Judge Kline: So when you use that term in connection with a pad, would that mean rolling the pad completely over?

THE WITNESS: I'm afraid that's what it means. That's why I say the pad with the casks on it is not going to overturn. The casks may overturn. The casks certainly would overturn before the casks on the pad could all overturn.

Judge Kline: Then it does open a different question which is, what aspect of your analysis accounts for just a tilting of the pad during a seismic event?

THE WITNESS: The dynamic bearing capacity shows that the factor of safety is adequate to prevent the bearing failure under the pad; that if you had low enough factor of safety from a bearing point of view you would get tilting of the pad.

Judge Kline: And you are confident that the pad would remain horizontal after the event, after a design-basis event.

THE WITNESS: I believe so. Because the soils out there are not subject to dynamic compaction that is one form of dynamic settlement that you would see, for some sites.

JUDGE Kline: Well, you are sounding a little uneasy about that. And let me just lay our concern out on the table. Did you hear the testimony of Dr. Singh and Dr. Soler.

THE WITNESS: I heard much of that testimony.

JUDGE KLINE: And isn't it so that their simulations all were performed on what would be essentially a horizontal pad.

THE WITNESS: Okay.

JUDGE KLINE: Do you agree with that? Is that correct.

THE WITNESS: I believe that's correct.

JUDGE KLINE: That was my understanding, too. So that the issue is if there's some doubt that the pad remains horizontal in a seismic event, would that not disturb the whole task stability analysis that they did?

THE WITNESS: If I'm not mistaken, some of the Holtec analyses were done with two casks, one cask, and it's my understanding that those analyses start with the deflected shape that would include a tilt to the pad just from the static loads of the pad. That's my understanding.

JUDGE KLINE: Are we going to get a

further exploration of this somewhere? I don't know if you are the right person to be talking to about it.

THE WITNESS: I'm not familiar with the Holtec analyses.

MR. GAUKLER: Dr. Soler could talk to that and my understanding is, like Mr. Trudeau says, you know, that --

JUDGE KLINE: Well, it appears that the issue is coupled in the sense that Dr. Trudeau knows about tilting pads and Dr. Soler would know the impact on the cask stability if the pad were to tilt.

MR. GAUKLER: My understanding is where the analysis is done is that you have a soil spring under the pad and when the earthquake with the soil spring, what happens to the pad and the cask happens to the pad and the cask.

JUDGE KLINE: Well, if that's his answer.

THE WITNESS: I seem to recall some of Wen Tseng's testimony that there were deflections accounted for in his design of the pad that amounted to, like, 3/8th's of an inch over this 67-foot long pad.

JUDGE KLINE: But those were -- I thought that referred to rigidity.

THE WITNESS: I understood that those were due to their soil structure, their dynamic analysis that included the forces from Holtec structure and interaction analysis.

JUDGE KLINE: I guess we have gone as far as we can at this point. But we would be interested in getting an answer.

MR. GAUKLER: We can bring Dr. Soler back so the Board gets information in that respect. We would be glad to do that.

JUDGE FARRAR: All right.

Ms. Chancellor, you may proceed.

CONTINUED CROSS EXAMINATION

BY MS. CHANCELLOR:

Q. Mr. Trudeau, isn't it true that Holtec treated the pad as a rigid body?

A. That's my understanding, yes.

Q. And when Dr. Soler ran the model, the simulation, isn't it true that he reset the model at the beginning of each run so that the pad was completely horizontal?

A. I believe he reset --

MR. GAUKLER: That's not correct.

JUDGE FARRAR: Wait. That was a question to the witness, Mr. Gaukler. I'll be happy to put you on the stand, but you don't want that.

Q. Oh, I'd love it.

A. I saw the movie; I was at that part of the testimony. And my understanding of what I heard was that they reset the simulation at the start to specifically account for the deflections that would be associated with the dead loads of the casks on the pad. And so the resetting that I heard was not to reset it to be horizontal, but rather to reset it to reflect the deflections that the system wanted to do under the dead load of the casks that were being analyzed at that time.

Q. Dr. Lam was asking you questions about the factors of safety with the 1.1 or the ten percent margin. All your calculations were based on a peak or the design basis case where peak ground acceleration is approximately .7g; correct?

A. Correct.

Q. And for a 10,000 year event, the peak ground accelerations in the horizontal and vertical direction would be 1.1 to 1.3g. Isn't that correct?

A. I believe that's correct.

Q. With respect to soil cement, you testified that soil cement has been used elsewhere for quite sometime. I don't know if you said it but dam stabilization, for example?

A. That's one application.

Q. And road embedments. None of these cases in the use of soil cement are for resistance to sliding; is that correct?

A. Those are not, no. There are some others such as coal slots constructed in seismic areas where the resistance at the toe of these coal slots is very similar to a sliding resistance, in my estimation.

Q. I beg your pardon, I didn't mean to cut you off.

A. What I meant to say was that the material at the toe has to provide a large horizontal force similar to the passive resistance that we are relying on for the canister transfer building, the soil cement apron around the canister transfer building.

Q. You haven't mentioned coal slots before, have you, Dr. Trudeau, in any of your depositions?

A. That one was in one of the papers that was handed out at the deposition we had just a month ago. It was in Dr. Bartlett's deposition that that paper was handed out. And it was not discussed in great detail.

Q. And you also mention that soil cement is being done at PFS for environmentally sound practices?

A. That's correct. It is so that we can avoid a lot of truck trips of other construction materials.

Q. Isn't it correct that Dr. Wickers has testified that you may need a central plant to bring all the eolian soil to and then make your soil cement there and then bring it back to the site?

A. It is expected that that batch plant will be constructed on site, in my estimation. I don't --

Q. But there are no firm plans how PFS is going to make its soil cement and where it is going to make it? Is that correct?

A. I believe that's correct. I have not seen any plans for where this batch plant will be. But it is my expectation that it is going to be on site.

MR. TRAVIESO-DIAZ: Your Honor, I may ask, we have had ten questions on how you do soil cement, where you build it. Do we need to go this way?

MS. CHANCELLOR: Following up, Your Honor.

JUDGE FARRAR: That objection is overruled. He answered the Board's questions and raised this subject of environmental benefits from this alternative. And that's one of the reasons we gave Ms. Chancellor another chance to cross. Anything he raises in response to the Board's questions is then part of his testimony and available for cross-examination. So the objection is overruled.

Q. In answer to Judge Lam, I believe you said that soil cement will act as a glue to glue the soil cement to the Bonneville clays; is that correct?

A. Yes.

Q. So the super glue, is this going to be part of your DeGroot bond interface testing?

A. That's correct.

JUDGE FARRAR: To keep your counsel from

harassing the Board, when she uses the term "super glue", you are free to disavow that. If you don't, Counsel will tell me that he doesn't like that term. So tell us whether you -- feel free, when you don't agree with the premise or characterization in a question, to say so.

MR. TRAVIESO-DIAZ: If I could ask the Board's indulgence in instructing Counsel to try to avoid characterizations that are not necessarily for her questions. To all counsel. I don't mean to pick on Ms. Chancellor.

JUDGE FARRAR: That is a fair request. And without criticizing Ms. Chancellor's questions, that's one of our ways to save time is where a characterization is unnecessary to the question, to avoid it.

Ms. Chancellor, could you reframe that question without the "super glue"?

MS. CHANCELLOR: Certainly. But I will use "glue" because that's what Mr. Trudeau used.

JUDGE FARRAR: That's fair.

Q. (Ms. Chancellor) Isn't it correct that you said that the soil cement will act as a very strong glue in bonding the Bonneville clays with the soil cement?

A. That is what I believe. I don't know if those were the exact words I used.

Q. And the testing that needs to be done to determine the strength of the bond between the interface of the soil cement and the Bonneville clays, that is called DeGroot testing?

A. This will be direct shear tests of those, the interfaces between those materials. And it will be similar to the testing that is described in that DeGroot paper on bonding of soil cement interfaces.

Q. And this DeGroot type testing is essential to establish whether you do have this firm glue or bond between the soil cement and the Bonneville clays; correct?

A. Yes. That is the testing that is done.

Q. And that testing will not be done until after PFS or if and when PFS gets a license. Is that correct?

A. Well, I guess I can't really say. It will take three or four months. So if it takes longer to get the license, then yeah. It won't be done after. But --

Q. There are several steps that need to be done, in your testing program, before you get to the bond interface testing?

A. That's correct.

Q. Isn't it true that the cement-treated soil is in a different category than the soil cement with respect to the compressive strength that it needs to meet?

A. Yes.

Q. And the compressive strength for the cement-treated soil is 75,000 psi?

A. No. That's not a compressive strength.

Q. Isn't it true that the cement-treated soil has to meet a Youngs modulus of 75,000 psi?

A. It has to be less than that value.

Q. Less than, right. And that this will also be part of PFS's soil testing program?

A. The soil cement testing, yes.

Q. And the meeting the 75,000 psi is critical to meet Holtec's cask tipover analysis? Is that correct?

A. That's correct.

Q. And isn't 75,000 psi on the outer limits of what has been done in the past to meet Youngs modulus? Lower limits of Youngs modulus? Not the outer limits but the lower limits?

A. I don't have any information on Youngs modulus for these cement-treated soils. So we will determine that in our testing program.

MR. TRAVIESO-DIAZ: Your Honor, I apologize for raising objections, but we are way beyond any question the Board asked. I don't think it relates to anything the Board asked.

MS. CHANCELLOR: That was my last question on soil cement, Mr. Travieso-Diaz.

JUDGE FARRAR: Whether or not Counsel is correct, let's -- did we have an answer? Had you answered?

MS. CHANCELLOR: He wasn't sure about Youngs modulus. That's fine, Your Honor.

JUDGE FARRAR: Let's have the question read.

(The record was read as follows:

"Q. And isn't 75,000 psi on the outer limits of what has been done in the past to meet Youngs modulus? Lower limits of Youngs modulus? Not the outer limits but the lower limits.

A. I don't have any information on Youngs modulus for these cement-treated soils. So we will determine that in our testing program.")

JUDGE FARRAR: The objection is moot.

Q. (Ms. Chancellor) In response to Judge Kline's question about the pad tilting, was tilting of the pads due to long-term settlement considered by Holtec in its cask sliding analysis? Do you know?

A. No, I don't know.

Q. In response to questions by, I think it was Judge Farrar and also maybe Judge Lam, where you talked about earthquake cycling and what Judge Lam referred to as impulse loading, in your testimony, if you would turn to Page 4, Answer 9. In the third sentence, "In addition because of the cyclic nature of seismic loading," from that sentence to the end of the answer, does this answer relate to earthquake cycling and impulse loading?

A. Yes.

Q. And do you rely on PFS Exhibit WW to form the basis of your answer in Answer 9?

A. As an example of what I'm trying to describe, yes.

Q. And then if you would turn to Page 16, the last paragraph of Answer 28. You specifically refer there to Exhibit WW. And you mention that at every point of the time history, the factors of safety will exceed a certain value. And you talk about the duration of the earthquake, and you rely on W W. Do you rely on WW for all of your answer in Answer 28 or just for this last paragraph?

A. Well, certainly the last paragraph.

Q. How about the rest of 28?

MR. TURK: The entire three-page answer?

MS. CHANCELLOR: That's correct, Mr. Turk. It's two and a half pages.

A. On Page 15 you can see the way those factors of safety are calculated that are plotted in WW.

Q. And that's -- is that for the part that starts, "Time history of forces --"? I'm trying to find out where in your answer you are relying on WW. You talk about 4.67 seconds into time history where you would have --

A. That discussion leads up to the equation of factor of safety against sliding is the sum of the resisting forces over the driving forces.

Q. Right.

A. And in the equation immediately below that is the calculation of the factor of safety of 1.25 for the maximum force from the Holtec time history. That is at that 4.675 seconds shown on WW as being down near the 1.1, but still above the 1.1 factor of safety criterion.

Q. So all of Page 15 to some extent relies on the Holtec force time histories?

A. I believe that's correct, yes.

Q. So is it fair to say that -- how about on Page 14; anything there you rely on the force time histories?

A. I don't think so. Not on Page 14, no.

Q. So 15. And so all of 15 and the --

A. And Page 16, I'd say on A 28.

Q. And Exhibit WW is from Holtec, and it's a part of their force time history or it's got factors of safety in time and seconds on the X axis and factors of safety on the Y axis. Correct?

A. That is correct.

MS. CHANCELLOR: Your Honor, we would like to renew our objection. This force time history is the force time histories we were talking about yesterday, the CD ROM that we had just received from Holtec. And if Mr. Trudeau is relying on Holtec's force time histories then we would like the opportunity to cross-examine him again specifically on his use of the Holtec force time histories after we have had the opportunity to review those force time histories that we just received from Holtec.

JUDGE FARRAR: Go ahead.

MR. GAUKLER: I believe that these are different force time histories than the ones that were the subject of the CD which were force time histories that were given to ICEC for the design of their pad. This is something I believe entirely different.

MS. CHANCELLOR: If these are different, Your Honor, we would like a copy of those, too.

JUDGE FARRAR: Can the witness shed any light on this in terms of the different histories?

THE WITNESS: These are time histories that I received from Holtec that represent the pad with the casks on them. I understand that CEC was provided an different set of time histories that represent the load of the cask on top of the pad. So these include the pad in the time histories that CEC was provided with, in my estimation.

JUDGE FARRAR: And how did the time histories that went into the animation fit into this picture?

A. I honestly don't know how that fits together. That's an Alan soler question, I'm afraid.

MR. GAUKLER: You are asking what, Your Honor?

JUDGE FARRAR: In other words, there are different time histories provided for different problems, provided by different people. And we are talking about several of them. And Ms. Chancellor is saying she wants a chance to cross-examine witnesses based on the right time or the time histories they used, after she has had time to review those time histories.

MR. GAUKLER: I believe this is a time history right here, Your Honor, the representation. And for example, such as in the MM, we provided time histories, Exhibit MM which was the angle of the -- referring to the degree of ways to the extent they were not vertically propagated, we had a set of time histories from a particular point in the cask. And those were provided as part of that calculation on MM. So you have different time histories at different points for different purposes. And this is a time history as Mr. Trudeau has described it. It is what it is. And there is nothing more to provide, is my understanding.

MS. CHANCELLOR: Your Honor, we have been chasing various time histories. That's been part of the problem with trying to analyze the work in this project. There are segregated areas here and here and there. And it is very difficult to put this web together, not knowing what we are missing. And I know we are missing the force time histories from Holtec. I don't know where this force time history, whether this is part of the CEC calculation, part of what Geomatrix gave Holtec, part of what Holtec gave CEC, part of what CEC gave to Mr. Trudeau. There are -- I mean, if you drew a flow chart you would have arrows going off at various directions.

MR. GAUKLER: Your Honor --

MS. CHANCELLOR: I haven't finished yet.

JUDGE FARRAR: Wait. You can say that to Mr. Gaukler, but you can't say that to me.

MS. CHANCELLOR: Point well taken.

JUDGE FARRAR: What are you waiting for from Dr. Soler? Didn't we give him a homework assignment that was going to take him some time? He was going to provide --

MR. GAUKLER: He is working on that right now.

MR. TRAVIESO-DIAZ: If I may explain on that. The State received a CD ROM yesterday morning with information they requested and they were asking for what the files in that CD ROM meant and how it could be used.

JUDGE FARRAR: And that was provided by Dr. Soler.

MR. TRAVIESO-DIAZ: Yes.

JUDGE FARRAR: As backup to his animation.

MR. TRAVIESO-DIAZ: No. That's a different thing. They had trouble with the CD ROM so I copied it to them and gave it to them yesterday. And they have had it at least since yesterday.

If I could make a suggestion, I think some of this confusion could be alleviated if Ms. Chancellor asked Mr. Trudeau a few questions of how he generated this document, what it represents, what the source is, and maybe Mr. Trudeau can go some distance in trying to explain this. I don't know if there's that much of a mystery. But that would be my suggestion.

MS. CHANCELLOR: Without knowing all of the force time histories out there -- and getting a copy of the CD ROM on the evening after Dr. Singh and Dr. Soler testified. I understand what you are saying that Exhibit WW, PFS is representing that this is different than the force time histories that Holtec gave us. What I am saying is that we get force time histories at the end of the day and we don't have a chance to analyze it, we can't understand how all of this integrates together, and I don't think that I should have to establish, through Mr. Trudeau, where he got his force time histories because I don't know how the Holtec force time histories relates to Exhibit WW.

MR. TRAVIESO-DIAZ: If I could answer that.

JUDGE FARRAR: Go ahead.

MR. TRAVIESO-DIAZ: Mr. Trudeau, I believe, just testified that these are not the force time histories you are talking about. But perhaps if you ask him the question just as to establish where did he get these and what it represents, you might be able to get some of your questions answered. I don't think that there is any mystery here that anyone is trying to hide.

JUDGE FARRAR: Let me say, without making a ruling, that the Board is sympathetic to

Ms. Chancellor's concerns. And maybe they are readily solvable, maybe not. But you recall, before there was any questioning yesterday I asked Mr. Trudeau how his work fit in with the different witnesses and that was part of the same question with the Board trying to have the same comprehension of how all this fits together that Ms. Chancellor is talking about. And we have had, in our rulings, allowing questioning that may not be directly mentioned or maybe colateral to one witness's testimony simply because of this interweaving notion. So we are sympathetic to the concerns.

Let me ask the witness if, having heard this discussion, he is able to shed any light on the misery, or lack thereof.

THE WITNESS: It's my understanding that Holtec did a soil structure interaction analysis of the pad with the casks on top of it. And for the 2000 year seismic event, that analysis was reported in their report number H I-2012640, which is titled "Multi-cask response at PFS ISFSI from 2000 year seismic event, rev. 2." Now, it's my understanding that from that analysis, Holtec generated a set of time histories that represented the forces at the base of the casks that they provided to CEC who performed the design of the pad based on supporting those loads on top of soil properties that I provided to CEC. In generating this time history plot of factor of safety --

JUDGE FARRAR: "This" meaning WW.

THE WITNESS: Exhibit WW. I needed to have a similar time history of forces that Holtec provided to CEC at the top of the pad generated for the pad plus the casks, so that we could see whether or not my use of the PGA for determining the effects of the earthquake on the pad underestimated the loads that were used. So Holtec provided me with a set of time histories of forces that represented the pad in the casks from this analysis that is described in this report. And I generated this factor of safety plot using the very simple equation shown on Page 15, I believe it is.

MR. TRAVIESO-DIAZ: As a point of clarification, Mr. Chairman, the report he is referring to was introduced by the state as Exhibit 173 yesterday so they certainly have the report.

MS. CHANCELLOR: Your Honor, if I may, Exhibit 173, the first set of time histories that Mr. Trudeau talked about that were given by Holtec to ICEC, they were the force time histories that we obtained on the CD ROM yesterday. Or was it the day before?

MR. TRAVIESO-DIAZ: If I may respond.

MS. CHANCELLOR: I haven't finished, Mattias.

MR. TRAVIESO-DIAZ: I'm sorry. I thought there was a period there.

MS. CHANCELLOR: No. It was a pause.

Mr. Trudeau testified that there were similar force time histories that he requested of Holtec for the top of the pad, for the pad and the cask. And it was from those Holtec force time histories that he developed this plot.

And in speaking with Dr. Bartlett, it is my understanding that the State does not have that set of force time histories that Mr. Trudeau asked Holtec to perform at the top of the pad and from which he developed Exhibit WW. So it turns out that we are actually missing a set of force time histories. This is not a force time history, WW. It is a plot. So we would request that we be given the force time histories that Mr. Trudeau obtained from Holtec from the top of the pad that includes the pad and the cask.

MR. TRAVIESO-DIAZ: The comment I was going to make - and I apologize for interrupting, I thought she was finished - was that I think the witness just testified that the force time histories that Holtec provided to ICEC which they just received a few minutes ago, or days ago, hours ago, have nothing to do or are different time histories than the one he used. I think he made it very clear that they were time histories supplied by Holtec based on this report which is in evidence that reflect the condition of both the pad and the casks, not the force and function on the pads. I think he testified to that. So I think we are mixing here apples and bananas. They are two entirely different sets of documents, one of which they currently have in their possession. Another one is based on a report they have had for quite some time.

JUDGE FARRAR: Before I confer with my colleagues. Does the Staff have a position?

MR. O'NEILL: I just conferred with the Staff and they agreed with Mr. Trudeau's characterization. The Staff's review was based on the set of time histories that he described.

MS. CHANCELLOR: Your Honor, whatever

the case, we would like a second bite whether it's the apple or the banana.

MR. O'NEILL: And he cited the correct Holtec report.

(Discussion off the record.)

JUDGE FARRAR: Let us make a general ruling on this that we are leaving to you all to apply. Without saying whether the Staff's and the Applicant's specific position on the immediate matter is correct or incorrect, there's a larger general issue here. And as I said, we sympathize with Ms. Chancellor's concern because it is one that our rulings and our questions have shown that we share, the interrelatedness of all this.

We are going to have at least a four-week hearing on seismic matters. As we have said, and as you all know, we don't care who wins and who loses but we care that everybody in that four weeks has a fair chance to present their case. Because as much as we are enjoying it here, we don't want to come back a year from now and retry the case. Out thought is that part of that - and again this is general, not specific - is that each party is entitled to have everything they think they need from the other parties, a fair opportunity to review it, and then a fair opportunity to cross-examine the witnesses. So as a general matter, we will rule only that Ms. Chancellor has what seems like a valid point.

Having said that, I think it is something that you all can work out. What does she need to have, when does she need to have it? Whether or not it specifically relates to what a specific witness says he relied on, there's a lot of interaction going on among the various witnesses. That was the point of our questions at the very outset of Mr. Trudeau's testimony. And you, over on this side of the room, pointing to the Staff and the Applicant, may have a perfect comprehension of all the little boxes that each little piece of evidence and thought process fits in. I'm not -- the Board certainly doesn't have that. And I think what Ms. Chancellor is saying is that she doesn't have that. Maybe you think she should have had that by now. But given how our thought processes work, we are not going to hold her to that standard at this point.

So, having said that, let me ask the Applicant what they think they can do to make sure this case moves forward so that at the end of the case, whatever the decision is, everybody believes they had a fair chance to be heard.

MR. TRAVIESO-DIAZ: I would like to consult with my colleagues and of course with the client. I believe that perhaps the Board will benefit since Mr. Soler from Holtec is going to be explaining to us a little bit more as to what he meant, what conditions he meant with respect to the pad, and so on. He can take that opportunity also to examine the various sets of histories, how they were generated, distributed. Do you believe that would be helpful to you?

JUDGE FARRAR: It would be helpful to us. But the question is whether that would be, even if it is helpful to Ms. Chancellor, whether that alleviates all her concerns in trying to present her case and challenge the case you all have put forward.

MR. TRAVIESO-DIAZ: If I may, my point was that after we understand what the universe is, we can establish what she has, what she doesn't have, and what she needs. It is very hard for me perhaps to understand what she needs to cross-examine on something that appears to be different from what she is concerned about. That's my whole point.

JUDGE FARRAR: But I think you all understand the interweavings better than anybody else does. That's our concern.

MS. CHANCELLOR: I'd like to make the point, it is true that when I was cross-examining Mr. Trudeau I did think the answers that are in his testimony A9 and A28, I did think that that was the force time histories from the CD ROM. But I now understand that it isn't those force time histories. It is a different set of force time histories that we were unaware of, that we don't have. And it's the force time histories on the top of the pad that Mr. Trudeau requested that Holtec gave him and from which this Exhibit WW is plotted.

I'll be happy to work with the Applicant and try and figure this out rather than waiting for Dr. Soler to get on the stand. Because if he starts testifying about all these different force time histories, I'm still not sure whether I know whether the State is going to have all those force time histories or not. So maybe the first start is to just talk with the Applicant, figure out what we do and don't have, and see if we can get whatever is missing.

MR. GAUKLER: I think we can work that out, Your Honor.

MR. TRAVIESO-DIAZ: We will be happy to do whatever is required to clarify the State's information that is available.

JUDGE FARRAR: Ms. Chancellor, we will, for the moment, preserve your right to call witnesses back for further cross after you're satisfied.

MS. CHANCELLOR: I think that would be an excellent idea. Thank you.

JUDGE FARRAR: And we will preserve the Applicant's right when you try to exercise your right to argue against it. But I think you all get the drift of where we are going. And I think this is a prudent step for everyone to take. I'd rather take a little longer at this hearing than have to repeat things later. With that, Ms. Chancellor, are you through with this witness for now?

MS. CHANCELLOR: Yes, I'm through for now, Your Honor.

JUDGE FARRAR: Okay. Then we would --

MS. CHANCELLOR: I do have one more question. Can we get the heat turned up?

JUDGE FARRAR: Donnell is not here, so

no. The answer is no. I don't know what he is do will.

We will they have the Applicant's redirect. How long do you expect it will take?

MR. TRAVIESO-DIAZ: I'm going to need a few minutes to try to digest the happenings of the last half hour. I suspect it will take about 15 minutes to figure that out and the questions will probably take on the order of a half hour, at the most. That's my best guess.

JUDGE FARRAR: So we have some hope of finishing this witness before lunch?

MR. TRAVIESO-DIAZ: As far as the Applicant is concerned, that is probably true.

JUDGE FARRAR: How about Staff?

MR. O'NEILL: We might have 15 minutes or so.

JUDGE FARRAR: Let's see if we can't get it done before lunch. The fewer questions you ask, the fewer questions Ms. Chancellor gets on her final turn. And then are we still trying to do Mr. Ebbeson this afternoon?

MR. TRAVIESO-DIAZ: Mr. Ebbeson is patiently waiting for his turn.

MS. CHANCELLOR: I think we will get

done with Mr. Ebbeson today, Your Honor.

JUDGE FARRAR: In terms of Mr. Ebbeson, I have the State's cross-examination plan. Do I have the --

MR. O'NEILL: No, you haven't. I can give it to you.

JUDGE FARRAR: At the next break. All right. Then you need some time?

MR. TRAVIESO-DIAZ: Ten to fifteen minutes.

JUDGE FARRAR: Let's come back at quarter to twelve and see if we can wrap this up before lunch.

MR. TRAVIESO-DIAZ: Thank you.

(A break was held.)

JUDGE FARRAR: The Board realized during a break we never set an official lunch hour earlier this week when we tried to go on a regular schedule. If you do the math, an hour lunch would come between 12:30 and 1:30 to give you a three and a half hour morning and a three and a half hour afternoon session. So for the future, let's try to work toward that. We departed from this yesterday to make sure we got through with a witness who had a plane flight, but let's try to keep to that. So let's see if we can come reasonably close to that today.

MR. TRAVIESO-DIAZ: I'll do my best. I'll talk fast. I don't think you want that.

CROSS EXAMINATION

BY MR. TRAVIESO-DIAZ:

Q. Mr. Trudeau, are you ready?

A. I believe so.

Q. Starting with some of the Board's questions, I believe that Judge Lam asked you several questions about the significance or the conservatism implied in having 1.1 factors of safety.

A. Yes.

Q. Could you elaborate how the factors of safety are arrived and what is included in them?

A. In our stability analyses, we have many conservatisms that we build in to the analysis to effectively result in a higher margin than the 10 percent that we have been talking about. One of the prominent ones for this type of soil that we have at the site has to do with the well-known phenomenon that the dynamic strength of these clayey soils in response to rapidly cycling loads is 50 to 100 percent greater than the static strength. So right there we've boosted the margin that's available by not taking credit for that dynamic strength of soil. We are using the measured static strength. And in these analyses we don't use a median or mean, generally. We are using the lower bound strengths that we have measured in these tests. So that also provides some additional conservatisms.

Q. Are there other conservatisms implicit or included in your analysis?

A. There are others, but I can't think of them right now.

Q. Okay. To tie this discussion of safety margins to a series of questions that was asked of you by Judge Kline, he was concerned about the at tilting of the pads and the bearing capacity. Is it true that the bearing capacity for the base case is safety factor of 1.17, the minimum bearing capacity margin?

A. I don't recall, but I think -- I'll just check here for a minute.

Q. Without attempting to lead the witness, if I could draw your attention to Page 12 of your testimony.

A. Yes. And that was for a case that is beyond what would normally be allowed based on that 100/40/40 rule that we discussed yesterday, because that case included the full hundred percent of the horizontal, both horizontal components acting at the same time.

Q. Are you saying that there is an additional conservatism in that number in that, instead of using 40 percent force, you are using the whole 100 percent?

A. That is correct.

Q. And do you have any idea how much the factor of safety would increase if you used 100/40/40 rule?

A. Yes. It is shown in my testimony there as 1.6.

Q. So you start with 1.6. Now, as you testified a moment ago if you removed some of this extra conservatisms, your factors of safety increase somewhat? Is that correct?

A. I'm sorry. I misspoke on the previous response.

Q. Okay.

A. That number goes from 1.17 to 2.

Q. All right. Have you computed how the

bearing capacity factor of safety would increase if you removed the conservatisms that you mentioned a moment ago in tensile, using the static strength as opposed to using the dynamic value?

A. I think I have.

Q. And again without trying to lead the witness, trying to move this along, if you turn your attention to --

MS. CHANCELLOR: Your Honor, I know that Mr. Travieso-Diaz is trying to move it along, but I think the witness should be responsible for finding, in his own testimony that he has written, where these factors of safety are.

MR. TRAVIESO-DIAZ: I'm trying to make the lunch hour, Your Honor.

JUDGE FARRAR: Ms. Chancellor, I was just thinking, however, that while in ordinary cases you are not supposed to lead the witness, in a case with prefiled testimony, I'm sure there's been such an abundance of leading the witnesses that I think this effort to point a witness to something is legitimate and as a time-saving measure. And for any witnesses in the room, and for this witness, that's just an aid to guide you to something. If that is not or if what your lawyer points you to is not responsive, please say so and go look for it somewhere else.

MR. TRAVIESO-DIAZ: Absolutely. I'm not telling you what to say. I'm telling you where to find it.

A. I see where it is, I'm sorry.

MS. CHANCELLOR: Could I have a page number, please?

A. It's at the top of Page 7 of my testimony.

MS. CHANCELLOR: Thank you.

A. And you can see for the static strength the factor of safety was 2.1 for the worst case. And increasing the strength by 50 percent to account for the dynamic strength increased the factor of safety, it goes up to 3.63.

Q. Would a factor of safety of 3.63, in your mind, be considered what you have seen in other industries or other parts of your practice outside of this area?

A. It is much greater than what is normally required for loadings that include seismic loadings for bearing capacity.

Q. Judge Farrar asked you a question about cost considerations and he was prompted by a question in which Ms. Chancellor characterized the decision to use soil cement as cheap. Could you tell the Board and the parties whether, in fact, cost was a consideration in your decision to use soil cement?

A. No. I mean, we were not instructed to go find a cheap way to make this work. But it is a very cost effective way to use a good product to make the stability of these foundations work.

Q. But what I'm trying to understand is your thought process. Was your thought process first to find something that worked and then if it was cost effective it was a good thing, like the cherry on top of the sundae?

A. I guess that's a fair statement, yes.

Q. Thank you. Ms. Chancellor in some of her questions implied that there was a link in the timetable for your completion with the soil cement test program and the decision on the licensing of this facility. Is there such a link?

A. No, not that I'm aware of. The soil cement testing is on hold because of my needing to be involved in preparation for these hearings and these ongoing hearings. Once it gets underway, it will progress and be completed in a period of months. I said three or four months, but it may take six months or so to complete, in my estimation.

Q. Would it be fair to say that it would take whatever it takes independently from how long the licensing takes?

A. Correct. PFS is committed to doing the testing. It will get done.

Q. Let me have a few slide questions, as well. At the very beginning of your testimony you refer to something that you call ASCE 4, and we have heard various other witnesses refer to ASCE 4-86 and 4-98. Are all these the same standard?

A. Yes. And the particular section that I was referring to has the same discussions regarding this 100/40/40 rule.

Q. There was a series of questions which, quite frankly, I had trouble following myself by Ms. Chancellor on the specific assumptions that went into your overturning calculation, and the possibility that the results might change if you assume the worst known sliding. Do you remember the questions?

A. Yes.

Q. And you answered that if you assumed the there was no sliding, in other words that the force was a little less than the 696 kips in the calculation, something different would happen.

A. Yes, I recall that.

Q. Could you give us additional information on that particular issue?

A. On the surface, it seemed like that was the right way to be looking at it. But I think that it would be -- I think that was a hasty decision on my part, that perhaps I need to revisit that and see just what the correct thing is.

Q. So you are not prepared to have an answer at this moment on the hypothetical I posed to you?

A. Correct. I should look at that further.

Q. Okay. While we are on the topic of calculations, she asked you several times whether the analysis that was your testimony had been submitted to the NRC.

A. Yes.

Q. Do you submit the analysis that you prepared to the NRC?

A. I do not. Not personally.

Q. And was the purpose of the analysis that you presented in testimony to be used to be submitted to the NRC for review?

A. Ordinarily our calculations are not submitted to the NRC unless they request them and the project decides to send them. That's my understanding, anyway.

Q. You also were asked a number of questions about the case in which there is an assumption made that there is a large layer of cohesionless soils eight to ten feet beneath the pads, and you computed the effects of having the behavior of that layer affect the behavior of the pads. Is that right?

A. Correct.

Q. Realistically, first does such a layer exist?

A. Well, we have seen that in some of the borings, especially as we get closer to the mountains where the canister transfer building is located. But even there it doesn't appear to be continuous everywhere.

Q. So the first clarification you'd like to offer, then, is that it is conservative to assume that you have a continuous cohesionless layer eight to ten feet below the surface?

A. That's correct.

Q. Now, in your judgment, is there a possibility that a layer such as that eight to ten feet below the surface could have an effect on the pad on the surface?

A. I have conservatively addressed that, placing that layer right at the base of the pads and showing that the displacement of the pads would be very small. If that material is, indeed, down at that depth, it's unlikely, in my estimation, that the earthquake forces will get up through that material to affect the pads anyway. So the ground motions will probably be reduced somewhat because of the large shear strengths trying to get through that cohesionless material.

Q. Well, if you assume that the layer was eight to ten feet below, why do you place it just below the pad?

A. To simplify the analysis.

Q. So it will be a much harder analysis to actually compute the displacement and the effects at the actual layer at which --

A. It is more difficult computationally, yes. But it is also more conservative to assume that it is higher in the profile.

Q. So, in fact, moving the application or the location of that layer to being next to the pads adds an additional element of conservatism to that calculation?

A. Correct.

Q. All right. Now, there were a number of questions about your Newmark analysis.

JUDGE FARRAR: Could I interrupt for a question? You said a moment ago or the question -- no, you said as you get closer to the mountains with the canister transfer building. You are only talking 300 or 400 feet.

THE WITNESS: No. I'm talking about the canister transfer building being closer to the mountains than the pad and placement area.

JUDGE FARRAR: But it's only 300 or 400 feet closer; isn't it?

THE WITNESS: Geologically there was a shoreline, one of the Lake Bonneville shorelines was closer to the southeastern area by the canister transfer building than where the pads are. So that resulted in these sandy silts that we are referring to. Silty sands.

JUDGE FARRAR: Okay. Thanks.

Q. (By Mr. Travieso-Diaz) I'm very grateful that the Chairman asked that question because now it places in my mind the new question which is, if you are talking about the potential existence of cohesionless soil being near the canister transfer building, why do you choose to make that assumption that it applied to the pads?

A. The silts -- there are some silts under the pad area that are less clayey than the upper Bonneville clay deposits and the lower layer of silty clay that is in the pad and placement, as well.

Q. Okay. Ms. Chancellor asked you a number of questions about your Newmark analysis. Could you please first explain what case that was. Is this the real case or hypothetical, or what were the assumptions?

A. That was done for this case where the cohesionless soils were moved up to the base of the pad. And it was also done for the hypothetical case where all the cohesion of the silty clay layer was assumed to not exist. And we assigned a friction angle of 17 degrees to the clayey soils. So both of those cases use that Newmark method to estimate displacements of the pads.

Q. She asked you specifically, among other questions, to what acceleration was the Newmark analysis, performed by Newmark, normalized. Do you remember that?

A. Yes. That was the .5g value.

Q. Right. And at least with me that was left with the implication that it wasn't appropriate to use it for the PFS where the acceleration is higher. What is your opinion on that?

A. Well, the Newmark method is a simplified method. The charts were developed based on the time histories normalized to .5g. However, as I believe I said in my testimony yesterday, Holtec did a more rigorous analysis of the sliding of the pads for that same friction angle of 17 degrees and determined that the estimated sliding of the pad was 3.4 inches compared to, for that hypothetical case where the friction angle was 17 degrees, two to six inches I think is what my calculations showed based on the simplified Newmark method.

Q. Are you saying the more detailed and perhaps accurate calculation that Holtec performed came up with a result that is within a range that is within a few inches of yours?

A. Yes.

Q. Would that, in your mind, satisfy you that your own Newmark analysis, the assumptions that were made were consistent with the result that you get from the more realistic situation?

A. Yes.

Q. Ms. Chancellor asked you a series of questions to which you answered no and I'm not going to go through all of them, but a few of them that I was able to catch. And I'll ask you to elaborate a little on the basis of why you said no to the questions.

For example, the one that stuck most in my mind is she asked you whether you consider the potential affect on your sliding analysis of imperfections on the surface of the pad. And you said you didn't consider that. Could you elaborate as to why you did not consider it?

A. Well, perhaps that wasn't even the right answer. But because I think that that kind of an effect is included in the coefficient of friction value of .8 that is used for this design. So specifically addressing the deformations, the irregularities of the top of the pad, no. But in my estimation, the high value, the upper bound value of coefficient of friction incorporates that kind of a detail.

Q. As I said, I have a list here. But in the interest of time I will move on to a different question here. Ms. Chancellor asked you whether, two things, first whether the design of the casks was such that they were allowed to slide on the pad. Do you remember that?

A. I remember discussions of designs and sliding and --

Q. And you agreed that, in fact, the casks are so designed that they are allowed to slide?

A. That's my understanding. Not that they are designed to slide, but that they can slide for this design.

Q. And I was going to get to that this way: She also seemed to imply or perhaps asked directly, I don't recall, whether it was required in order for the Holtec design to work, if you will, that the pads do slide. Is that what you intended to say?

JUDGE FARRAR: Did you say pads?

Q. That the casks, I mean casks actually slide. I meant the casks.

A. My calculations take credit for the fact that the casks slide at points in the analysis where the forces are great enough to exceed the coefficient of friction values.

Q. But in order to provide stability against sliding, that is not a necessary assumption in your analysis, is it?

A. There are portions of my calculation where I take credit for the .8 times the normal force. So I would say yeah, if they don't slide, my analysis doesn't look at it with them not sliding.

Q. If I can have a moment.

Now I'm going to ask the witness to help me find something. Mr. Trudeau, Ms. Chancellor asked you several questions about your Exhibit UU which is the stability calculation for the cask storage pads, and specifically she asked you about an equation in which you compute resisting forces versus applied forces. And one of the terms in that equation is an acceleration called A sub V.

A. Correct.

Q. Would you help me find where that particular question is so I can ask you a question about it.

I think I found it. Could you turn to Page 13 of your stability calculation. That's Exhibit UU.

A. Correct.

Q. I believe this is the one that she was asking you about. A sub V on the second equation of the page, and there you are using a value of -- well, why don't I ask you, what is the .695 percent?

A. That's the peak vertical ground acceleration.

Q. She asked you to assume that that vertical ground acceleration approached 1. To what would that correspond in reality?

A. A big earthquake. Vertical acceleration of 1g.

Q. Okay. Are you saying that this formula gives you negative values if you have an earthquake in excess of 1g?

A. It says that the resisting moment goes to zero, yes.

Q. All right. And based on your understanding of --

JUDGE FARRAR: Wait. Does it go to zero or does it go -- in the real world can it go negative? According to the equation, it can go negative.

THE WITNESS: No. It goes to zero.

Q. (By Mr. Travieso-Diaz) So you are saying there is a range of values for which this equation applies?

A. The resisting moment is based on the weight of the structure. So if the vertical acceleration is greater than 1, the effective weight is reduced to zero.

Q. And in a situation like that, would it mean that the cask, for example, or the pad, the element would lift off?

A. No. Well, it will lift off but I guess it wouldn't necessarily tip over. You'd have to add some kind of a rotational force to it at this point.

Q. So what you are saying is that up to acceleration of 1g, essentially, this equation will give you the resistance of the system, if you will, to the applied forces? And beyond that point you will no longer have resistance?

A. Correct.

Q. So whether you have 1 or 1.1 or 1.5, what you have is that this equation, as such, would no longer apply?

A. That's correct.

Q. Now, do you have any evidence that leads you to suggest that the value of A sub V for this site on the design basis earthquake will experience a vertical acceleration of 1?

A. It's my understanding that the vertical ground acceleration for the design earthquake is .695g.

Q. So that one assumption was just that; a hypothetical?

A. It's a beyond-the-design-basis case.

Q. And you took it to be a hypothetical?

A. Correct.

JUDGE FARRAR: In answer to the question a moment ago, you said 1g vertical is a big earthquake. What do you mean? Does that ever happen?

THE WITNESS: There's been situations where peak ground accelerations greater than 1g have been recorded.

JUDGE FARRAR: Do any come to mind that we would know of.

THE WITNESS: One that is famous is the Pequoima Dam record. But it's questionable whether that was really a reflection of accelerations in excess of 1g or whether that was just some movement in the abutment that caused the record, where the recorder was situated, to record greater than 1g. But there are some other more recent events that have recorded higher than a g, but I don't remember the names right now.

JUDGE FARRAR: Okay.

JUDGE LAM: So 1g basically means the mass is escaping gravity.

THE WITNESS: Correct.

MR. TRAVIESO-DIAZ: I will quit so we can make our lunch hour, unless something prompts further questions.

JUDGE FARRAR: Okay. Then Staff can do recross.

MR. TURK: Thank you, Your Honor. Mr. O'Neill will do the recross. The first thing that Staff is going to do is introduce or proffer a document. I'm going to pass that out now.

JUDGE FARRAR: And you'd like this marked as what?

MR. O'NEILL: Proposed Exhibit EE.

MR. TURK: This would be Staff proposed Exhibit EE.

JUDGE FARRAR: Let's have the reporter do that for identification.

(EXHIBIT-EE WAS MARKED.)

JUDGE FARRAR: Go ahead.

RECROSS EXAMINATION

BY MR. O'NEILL:

Q. Mr. Trudeau, do you recognize this document?

A. Yes.

Q. Would you please identify it.

A. This is Section 3.8.5 Foundations, of the Nuclear Regulatory Commission Standard Review Plan. The entire book or report is NUREG 0800.

Q. Formerly NUREG 75/087?

A. Correct.

Q. Is this the document that you had referred to in response to one of Judge Lam's questions?

A. That's correct. This one shows the factor of safety values of 1.1 for the design basis seismic event.

Q. Referring you to Page 3.8.5-7. Do you see Paragraph or Section 5?

A. Yes.

Q. Structural Acceptance Criteria?

A. Correct.

Q. Which combination would be the

applicable combination for design basis earthquake? If you don't recognize it immediately --

A. I'm just confirming that the E prime is really the design basis earthquake.

Q. If you could turn to page 3.8.5-6 it might help you. Paragraph 3, Loads and Load Combinations. You see part C?

A. Yes. Load Combinations. Under 3, Loads and Load Combinations, part C is dead load, plus H plus E prime. H is the lateral earth pressures, I believe. And the E prime is the seismic loads due to the design basis earthquake.

Q. And as you mentioned, the minimum factors of safety for that particular load combination against both overturning and sliding are 1.1; correct?

A. That is correct.

Q. Okay. Thank you.

JUDGE FARRAR: Counsel, you mean as found on Page 7?

MR. O'NEILL: Yes, on Page 7.

JUDGE FARRAR: For case C, or combination C.

MR. O'NEILL: I'd like to offer this exhibit into evidence.

MS. CHANCELLOR: What is number, Mr. O'Neal?

MR. O'NEILL: Staff's Exhibit EE.

JUDGE FARRAR: Does the State have any objection?

MS. CHANCELLOR: No objection, Your Honor.

JUDGE FARRAR: Applicant?

MR. TRAVIESO-DIAZ: No.

JUDGE FARRAR: Then Staff's EE will be admitted.

(EXHIBIT-EE WAS ADMITTED.)

Q. (By Mr. O'Neill) Mr. Trudeau, I'd like to ask you a couple of follow-up questions. Actually they are follow-up questions to a question posed by counsel for PFS, and they do pertain to your Exhibit UU, specifically the overturning stability analysis for the cask storage pads. And I understand that you requested that you might like some additional time to consider the State's hypothetical case, involved consideration of the moment --

A. Correct.

Q. The driving force of less than .696. But I'm going to pose a couple of questions, and I will respect your request if you want more time. Is it drew that a free-standing cask like the ones you have at PFS, the proposed PFS site, cannot transmit moment to the pad?

A. That's my belief.

Q. That is your belief?

A. There is no moment connection between the bottom of the cask and the pad.

Q. In addition, do you believe the weight of the cask would maybe counter the moment due to the horizontal inertial forces?

A. Yes.

Q. Yes? Okay.

JUDGE FARRAR: Counter what moment?

MR. O'NEILL: Moment due to the horizontal inertial forces.

Q. (By Mr. O'Neill) Did you understand it that way?

A. Yes.

Q. I'm going to move on to a different subject here. I thought at some point you may have indicated that you did not consider soil structure interaction in your pad stability analysis. I'm not sure, I hope I'm not mischaracterizing that. In light of that, I'd ask you to turn to Exhibit

UU, tables 2.6-7 and 2.6-8, on Pages 107 and 108 of Exhibit UU.

A. Yes, I see that.

Q. Could you briefly describe what these tables represent and the differences between the two tables?

A. The first table was developed to determine the allowable bearing capacity for the pad cask system based only on inertial forces. That was calculated based on peak ground acceleration. The table 2.6-8 on 108 was the allowable bearing capacity that was developed using the forces, the maximum cask dynamic forces that I received from CEC.

Q. And would those forces have taken into consideration soil structure and --

A. Yes, those did. I thought I made that clear in my earlier testimony. It's just the forces of the pad that the PGA had been used to calculate the dynamic forces of. The cask forces were generally from the soil structure interaction analysis, generated forces that Holtec gave to CEC.

Q. Thank you for clarifying that. With respect to the CTB, would you agree that the soil cement buttress was designed to withstand dynamic forces from the building itself?

A. That is correct.

Q. Okay. One more question. I think in response to a question from Judge Farrar, you indicated that you will make the design work. My understanding -- you had used those particular words.

A. I don't recall.

Q. But my understanding is that it is Applicant's responsibility to develop, submit a design that satisfies regulatory requirements for site- specific engagements. Correct?

A. That's correct.

Q. I have no further questions at this time. Thank you.

A. Thank you.

JUDGE FARRAR: Thank you, Mr. O'Neill. Ms. Chancellor, does anything in the Applicant's or the Staff's questioning --

MS. CHANCELLOR: Just one or two, Your Honor. Very, very briefly.

RECROSS EXAMINATION

BY MS. CHANCELLOR:

Q. Discussing Page 13 of your calculation,

Exhibit UU, Mr. Trudeau, Mr. Travieso-Diaz, I believe, stated that if the vertical force is greater than 1g then the equation does not apply. Do you recall that?

A. I recall that.

Q. Isn't it true that the equation does apply but, in fact, the resistance goes to zero?

A. I believe that's what I said; that the resistance goes to zero. The cask would, indeed, lift off at that point but would not necessarily overturn. It would need additional rotational moment of some form in order to make it overturn. It would -- what would in reality happen, in my estimation, is that the pad would move down, away from the cask. And the cask's inertia would keep it suspended in place for that brief moment in time.

Q. Suspended animation.

A. For .005 seconds.

Q. So there's no resisting moment for overturning. Is that correct?

A. There obviously must be. Well, wait a minute. This is a beyond-design-basis case, in my estimation. Because our vertical acceleration isn't 1g anyway.

Q. So greater than 1g you consider this to be a hypothetical case; is that correct?

A. Correct.

Q. And isn't it true that for the 10,000 year earthquake the predicted vertical acceleration for the PFS site is 1.23g?

A. The accelerations for the 10,000 year earthquake are greater than 1g. I don't remember the numbers. It's a beyond-design-basis case, in my estimation, that I haven't done a lot of work for or with.

Q. But it's greater than 1g. Is that correct?

A. Yes. I understand that it is greater than 1g.

Q. Thank you. I have no further questions.

MR. TRAVIESO-DIAZ: May I have a few, one question for clarification?

JUDGE FARRAR: Yes.

FURTHER RECROSS EXAMINATION

BY MR. TRAVIESO-DIAZ:

Q. I believe that you, in your answer to one of Ms. Chancellor's questions a moment ago, you indicated that you would consider a vertical acceleration of 1g or greater to be beyond the design basis. Isn't it a more accurate statement to say that any acceleration in excess of .695g is beyond your design basis?

A. Yes. That's correct.

Q. Do you want to modify your previous answer to say that?

A. I didn't think that was necessary. We were talking about a particular one greater than 1g, and that's greater than .695 so in my estimation it is a beyond-the-design-basis event, that I know is .695g vertically.

Q. Okay. Thank you. That's all.

(Board conferred off the record.)

JUDGE FARRAR: The Board wants to make sure we are clear on something. Some of the evidence so far uses two as the design basis, which we -- 2000 year return period as the design basis, which we understand is in accordance with the exemptions that the Staff ran. Some of the evidence says that the Applicant is still okay at a 10,000 year return period. We want to make sure that everyone is thinking along the right lines legally.

As we understand the State's challenge

to the exemptions, they say two is no good, 2000 year return period is no good. It should be 10,000 year return period. But in any event, it ought to be something more than 2000. We want to know from the Staff and the Applicant whether the proof that they are deducing at this hearing, whether they are trying to prove that it would or that the design is adequate at a 10,000 year return period. In other words, do we have just a legal issue that if the State wins and it is a 10,000 year period, they win; at least for the moment? Or are you trying to make a case that even if the State is right -- even if the State proves the Staff shouldn't have granted an exemption and therefore we should be at a 10,000 year return period, is the Applicant's evidence intended to say it is okay at 10,000 years?

MR. GAUKLER: Our evidence is twofold, Your Honor. First of all, the design basis is 2000 years. And we have to meet all the NRC requirements for designing that design basis. Beyond that, we say we have, by virtue of designing to the 2000 year design basis earthquake, there are conservatisms and sufficient margins such that the facility could with- stand a much larger earthquake with no radioactive release. And it's our position that we could with- stand an earthquake in the range of 10,000 years or more with no radioactive release.

JUDGE LAM: To be more specific, is Mr. Trudeau's testimony only testifying to the 2000 year earthquake?

MR. TRAVIESO-DIAZ: If I could address that. You take a look at the early pages of his testimony, starting with page -- essentially starting on Page 5. I don't want to walk through it. But I think it provides information in his testimony and the testimony he gave today to the effect that not only is the design capable of meeting the 2000 year design basis earthquake, but there is enough conservatism and enough margin in the design that it could withstand, even though it is not designed to, it could withstand accelerations of forces in excess of that design basis and therefore corresponding to a larger earthquake.

MR. GAUKLER: In terms of the overall presentation of Applicant's testimony, we have in Mr. Trudeau's testimony, Mr. Ebbeson's testimony, as well as in Dr. Singh's and Dr. Soler's testimony, testimony about the conservatism of the design and the capability of the design to withstand an earthquake up in the range of 10,000 years without radioactive release and then Dr. Cornell and his testimony where he talked about the reasonableness of the exemptions refers to the margins that are described in Mr. Trudeau's, Mr. Ebbeson's, and Dr. Soler and Dr. Singh's testimony.

JUDGE LAM: My question is more specific than that, Mr. Gaukler. Mr. Trudeau's testimony testified that the pad would not slide, would not overturn, so does the canister transfer building. But what I just heard from Ms. Chancellor and from Mr. Trudeau that the acceleration in the vertical direction can approach 1 for the beyond-design-basis earthquake. If that is the case, then the issue of the pad or the building would not overturn would only be limited to the design basis earthquake because beyond-design-basis earthquake there's no demonstration here.

MR. GAUKLER: I believe that it is discussed in the testimony of Mr. Trudeau and Mr. Ebbeson that neither the building nor the pad would overturn. We would expect neither the building nor the pad to overturn at the beyond-the-design-basis earthquake of up to 10,000 years in the range of the 1.2g, et cetera. I believe that's in their testimony.

JUDGE LAM: May I ask the witness to refer me to that? Where in your testimony, Mr. Trudeau, do you testify for 10,000 year earthquake the pad would not slide, the building would not overturn, and then there's no bearing capacity failure.

THE WITNESS: My analyses do not address the 10,000 year earthquake. What I have in my testimony identifies additional margin that is available over the analyses presented for the 2000 year design basis earthquake. And you can see that in many cases the amount of margin is enough to get up to the 10,000 year earthquake. But I have not specifically looked at all of the -- like, for instance, I don't think I even looked at the overturning case for the 10,000 year.

JUDGE FARRAR: So if you were doing a 10,000 year design basis earthquake, you would do more than you have presented here.

THE WITNESS: I believe that's a correct statement, yes.

JUDGE LAM: That's the answer I'm

looking for. Thank you.

JUDGE FARRAR: Staff wants to be heard on this?

MR. TURK: Yes. Briefly, Your Honor. The NRC regulation requires that a design basis be established for a facility. The choice of a 2000 year earthquake has been approved. I'm sorry. The ground motions associated with a 2000 year return period earthquake has been approved and will be the design basis for this facility.

JUDGE FARRAR: By the Staff.

MR. TURK: Yes. The Staff has conducted an evaluation and has determined that this facility does, in fact, meet the 2000 year ground motion design basis earthquake. That's the basis upon which we would propose licensing the facility.

Now, beyond that, the State has raised a contention that says 2000 year return period ground motions are not conservative enough. So for purposes of litigating that contention, we have conducted our own confirmatory analysis as shown by Dr. Luk, and have evaluated the consequence of a larger return period earthquake at this specific site. And our analyses and evaluation conclude that in terms of protection of public health and safety there is no consequence adverse to the public if you do reach earthquakes as large as the 10,000 year return earthquake.

JUDGE FARRAR: Suppose we were to hold on the exemptions issue -- and this is purely hypothetical, no one should take comfort or lack of comfort from it. We are just trying to make sure we know where we are. Suppose we were to hold that there was something faulty with the Staff's grant of the exemptions and the proper design basis earthquake should be 5000 years, and let's assume that that determination were upheld by the reviewing authorities, would the Staff then argue that the evidence presented thus far, and to be presented, satisfies that requirement or would you say, "Wait, we have to go back to the drawing board and we have to do an official design basis 5000 year --"

MR. TURK: If the design basis was selected to be higher than what the Staff has approved, you would need to have further demonstration from the Applicant that it meets whatever the higher standard is. And we would evaluate that demonstration.

JUDGE FARRAR: You wouldn't point to

this evidence and say, "It is okay."?

MR. TURK: There would be a time in which we probably would refer back to it and say we are satisfied with respect to consequences, but we have not evaluated design against the 5000 year earthquake and we would need to do that.

JUDGE FARRAR: Ms. Chancellor, in light of that do you have any comment you'd like to make?

MS. CHANCELLOR: Just a couple of things, Your Honor. The State starts from the standpoint that the PFS design is not conservative and then we build our case from there. With respect to the seismic exemptions, the arguments have been, "Provided there aren't those consequences, then it's okay." I don't believe that's what we are looking at. We are looking at the exemptions standards, which is, does it meet health and safety requirements and is it in the public interest. And I'm pleased to note that the Staff would require further demonstration by PFS for a hypothetical case of a 5000 year earthquake because the design basis earthquake does assume ground motions at about 0.7g and the 5000 year earthquake would have higher ground motions.

MR. TURK: I would note in that case it

would not be a hypothetical earthquake. It would be the design basis earthquake.

JUDGE FARRAR: Right. And again, draw no conclusions from this question. This was just -- I wanted to make sure that the record or that we know for what purposes this record is being developed and what its strengths and limitations are. And that's all we wanted to make sure of.

If no one has any questions, it is 12:40. Let's come back at 1:40 with a new witness. This witness is excused.

THE WITNESS: Thank you, Your Honor.

JUDGE FARRAR: The record will reflect that was said with deep sincerity. But I believe you will back. So let's take an hour lunch; 12:40 to 1:40.

(The lunch break was held.)

JUDGE FARRAR: Let's begin the afternoon session. Mr. Turk.

MR. TURK: Thank you, Your Honor. Over the lunch break, Ms. Nakahara asked me for a copy of the time histories that were used by Dr. Luk in his analysis, and I do have those on a CD Rom disk. This is my own CD Rom disk copy. I'm handing that to Ms. Nakahara now.

JUDGE FARRAR: So you need that back at some point?

MR. TURK: Yes, please. This is my own CD.

MR. TRAVIESO-DIAZ: Mr. Chairman, we would like to make two announcements, as well. Are we on the record.

JUDGE FARRAR: Yes.

MR. TRAVIESO-DIAZ: First I want to announce that Mr. Donnell is now in the room, so he is available for any assistance that we may need.

JUDGE FARRAR: We needed him this morning.

MR. TRAVIESO-DIAZ: And second, I would like to state that again, thanks to the good office of Mr. Trudeau, we were able to locate the time histories that were used in generating his Exhibit WW, and I have handed the State counsel at the break a copy of those time histories in this case.

JUDGE FARRAR: Okay. Then we'll give the State some time to look over these new materials and take that up at the appropriate point. You're now ready to present Mr. Ebbeson?

MR. TRAVIESO-DIAZ: Yes.

JUDGE FARRAR: Mr. Ebbeson, would you

stand, sir and raise your right hand.

BRUCE EBBESON,

called as a witness, for and on behalf of the Applicant, being first duly sworn, was examined and testified as follows:

DIRECT EXAMINATION

BY MR. TRAVIESO-DIAZ:

Q. Would you please state your name for the record.

A. Bruce Ebbeson.

Q. Mr. Ebbeson, do you have before you a document entitled Testimony of Bruce E. Ebbeson on Section D of Unified Contention Utah L/QQ?

A. Yes I do.

Q. Dated April 1, 2002?

A. Yes, I do.

Q. Thank you.

Was this document prepared by you or under your direct supervision?

A. Yes.

Q. Do you have any corrections to make to this document?

A. I have no changes to make to this

document, but I would like to make a change to Exhibit MM, which was submitted with Dr. Wen Tseng's testimony. I prepared that document.

MR. TRAVIESO-DIAZ: With the Board's indulgence, may we proceed?

JUDGE FARRAR: Yeah, now this has already been admitted?

MR. TRAVIESO-DIAZ: Yes.

MR. EBBESON: It was referred to in my testimony.

MR. TRAVIESO-DIAZ: If you remember, a calculator was used to solve the dispute.

JUDGE FARRAR: Right.

MR. EBBESON: I forgot my calculator.

Q. (By Mr. Travieso-Diaz) Would you, Mr. Ebbeson, turn to Exhibit MM. Do you have a copy?

A. Yes, I do. You go to the last page of Exhibit MM, there are two typographical errors I'd like to correct.

JUDGE FARRAR: Is that Page 5 or --

MR. EBBESON: It's Page 5.

JUDGE FARRAR: Okay, right.

MR. EBBESON: About halfway down the page is an equation that says A equals square root

67 times 30, parentheses, parentheses divided by four is equal to 22.4 feet. The second bracket should be moved to the other side of the four.

JUDGE FARRAR: Right.

MR. EBBESON: And that makes the number come out correct. The number is right, I just typed it wrong. And on the last sentence on the page, there's an A in there that should be deleted. It should say see attachment A for details.

Q. (By Mr. Travieso-Diaz) Do you have any more corrections?

A. That is all.

MR. TRAVIESO-DIAZ: I would like to note for the record that a copy of Mr. Ebbeson's testimony has been distributed to the parties and the Board, and I would like at this point to move that his testimony be admitted into evidence.

JUDGE FARRAR: And this is no different than what we had prefiled a month ago?

MR. TRAVIESO-DIAZ: Not at all. It's identical.

JUDGE FARRAR: Huh?

MR. TRAVIESO-DIAZ: It is identical.

JUDGE FARRAR: Any objection by the State?

MS. CHANCELLOR: No, Your Honor.

JUDGE FARRAR: Mr. Turk?

MR. TURK: Mr. O'Neill.

JUDGE FARRAR: Mr. O'Neill? Wait. No objection?

MR. TURK: No, Your Honor.

MR. O'NEILL: No, Your Honor.

JUDGE FARRAR: Then Mr. Ebbeson's testimony will be bound into the record at this point as though read.

(Prefiled testimony of Bruce Ebbeson follows:)

April 1, 2002

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
	)	
PRIVATE FUEL STORAGE L.L.C.	)	**Docket No. 72-22**
	)	
(Private Fuel Storage Facility)	)	**ASLBP No. 97-732-02-ISFSI**

TESTIMONY OF BRUCE E. EBBESON
ON SECTION D OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESS BACKGROUND

Q1. Please state your full name.

A1. Bruce E. Ebbeson.

Q2. By whom are you employed and what is your position?

A2. I am a Senior Lead Structural Engineer with Stone & Webster, Inc., a Shaw Group Company ("S&W"), in Cherry Hill, New Jersey.

Q3. Please summarize your educational and professional qualifications.

A3. My professional and educational experience is described in the *curriculum vitae* attached to this testimony. Briefly summarized, I have approximately thirty years of experience as a Civil/Structural Engineer, specializing in the structural design and analysis, including seismic analysis, of nuclear facilities. I am currently the supervisor of the structural division for S&W's Cherry Hill office and serve as structural engineering consultant on various projects performed by S&W in its Cherry Hill, Boston, Denver and Taiwan offices. My experience has included assignments as Principal Structural Engineer on many nuclear facility projects. I have, among other activities, performed and supervised the performance of original designs and design modifications for those projects, as well as safety evaluations to meet licensing requirements. I have also performed independent design reviews of nuclear facilities at various stages of their licensing and operation.

Q4. What is the basis of your familiarity with the Private Fuel Storage Facility?

A4. S&W is the Architect/Engineer for the Private Fuel Storage Facility ("PFSF") under contract with Private Fuel Storage, L.L.C. ("PFS" or "Applicant"). As such, it coordinates the facility design activities, including the studies needed to characterize the PFSF site and establish its suitability. I have been involved in the design of the PFSF since June 1998. My duties include planning and supervising the preparation of calculations and drawings for the facility and responding to questions posed by the U. S. Nuclear Regulatory Commission ("NRC"). In particular, I am responsible for the seismic analysis and structural design of the Canister Transfer Building ("CTB") for the PFSF.

Q5. What is the purpose of your testimony?

A5. One of the purposes of my testimony is to describe the structural design of the CTB and the ability of the building to withstand the seismic loadings imparted by the 2,000-year return period earthquake and other, more severe seismic events. My testimony demonstrates that there are significant margins beyond the design basis requirements in the designs of the CTB and the important-to-safety structures, systems and components ("SSCs") it contains that will enable them to survive earthquake ground motions much greater than those of the 2000-year design basis earthquake. My testimony will also respond to certain allegations raised by the State of Utah in Part D of Unified Contention Utah L/QQ with respect to the seismic analysis and design of the CTB and its foundation.

II. FUNCTIONS AND CONSERVATIVE DESIGN FEATURES OF THE CANISTER TRANSFER BUILDING AT THE PFSF

Q6. What are the design functions of the CTB?

A6. As discussed in Section 4.7.1 of the PFSF Safety Analysis Report ("SAR"), the CTB provides physical protection and shielding for the canisters containing spent fuel during their transfer from the shipping casks in which they are brought to the site to the storage casks used to store them at the PFSF. The CTB is a reinforced concrete structure with thick walls providing tornado-generated missile protection and radiation shielding.

The main function of the CTB is to facilitate the safe performance of canister transfer operations at the PFSF. Specific CTB functions include:

- Load or unload spent fuel shipping casks from railcars or heavy-haul tractor/trailers.
- Provide weather and tornado protection for performing the canister transfer operations.
- Provide the support structure for the single failure-proof cranes required for the transfer operations.
- Provide radiological shielding during the transfer operation.
- Store potential low-level radioactive waste from health physics surveys.
- Provide storage and laydown space for transfer and shipping equipment.
- Provide a staging area for storage casks.

The important-to-safety SSCs in the CTB include a 200 ton overhead bridge crane, a 150 ton semi-gantry crane, seismic support struts, the spent fuel canisters, shipping and storage casks, and transfer casks used during the canister transfer operation.

Q7. What are the main NRC regulatory and industry guidance documents used in the seismic design of the CTB?

A7. PFS follows the criteria specified by the NRC in the Standard Review Plan ("SRP") for independent spent fuel storage installations ("ISFSIs"), NUREG-1567, for the seismic design of structures such as the CTB. In addition, the criteria used for the seismic design of the CTB are those used to meet the safe shutdown earthquake loads in accordance with the NRC Standard Review Plan for nuclear power plants, NUREG-0800, to the extent those criteria are pertinent to ISFSIs such as the PFSF. Both NUREG-1567 and NUREG-0800 provide load combinations and acceptance criteria which, for the loads applicable to the PFSF, are very similar, and provide similar degrees of conservatism.

The seismic analysis and design of the CTB are performed in accordance with the standards set forth in nuclear industry standard ASCE 4-86 (relevant sections of which are included as PFS Exhibit XX), an accepted standard widely used and accepted in the seismic design of nuclear power plants and ISFSIs, which provides comparable levels of conservatism to those in the SRPs.[1]

The concrete portions of the building are designed for the appropriate load combinations, as described in Section 3.2.11.4.1 of the SAR. The strength capacity of a concrete cross-section under the seismic load combinations was determined using the guidance in the ACI 349 Code. Use of this standard is called for under SRP guidelines for nuclear facilities, including both nuclear power plants and ISFSIs.

For structural steel portions (primarily roof beams and girders), the allowable stresses are computed using the applicable load combinations for normal and shear stresses, as described in Section 3.2.11.4.1 of the SAR. The allowable steel stresses are determined following the guidance in the AISC N690 code, another standard code used in nuclear power plant design. The N-690 code is more stringent than the AISC code "Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design," which is endorsed by both NUREG-0800 and NUREG-1567.

To the extent pertinent for ISFSIs, the load combinations and acceptance criteria for the CTB under seismic loadings are those specified in NUREG-0800 for the safe shutdown earthquake loadings for nuclear power plants.

Q8. Would you please describe the main features of the design of the CTB and its foundation that provide protection against the forces resulting from an earthquake?

1 In his deposition, State witness Dr. Farhang Ostadan acknowledged that ASCE 4-86 and its subsequent revision ASCE 4-98 are very important standards used in the seismic design of nuclear facilities. See Ostadan Tr. at 72-73.

A8. The CTB roof consists of an eight-inch thick reinforced concrete slab supported on structural steel beams spanning in the N-S direction, which are in turn supported by plate girders spanning in the E-W direction. There are studs on the beams and girders to prevent the roof slab from uplifting during a design basis tornado. The beams and girders are designed as simply supported members, with no consideration of composite behavior. The roof has been designed for a vertical acceleration of 1.84 g at the roof center.

The CTB is supported by a heavily reinforced concrete foundation mat. The foundation mat is 240 ft. in the E-W direction, 279.5 ft. in the N-S direction, and 5 ft. thick. A reinforced concrete key, 1.5 ft. deep by 6.5 ft. wide, will be constructed around the perimeter of the foundation mat. The purpose of this key is to ensure that the full shear strength of the clayey soils beneath the foundation is available to resist sliding of the structure due to the loads from the design basis ground motion.

The CTB foundation design calls for soil cement to be placed around the base mat to help resist earthquake sliding forces. Soil cement will thus surround the foundation mat and will extend outward from the mat to a distance equal to the associated mat dimension; i.e., approximately 240 ft. out from the mat in the E-W direction and approximately 280 ft out in the N-S direction. Existing soils will be excavated to a depth of approximately 5 ft. 8 in. below grade, mixed with cement, and placed and compacted around the foundation mat. The soil cement placed around the CTB foundation mat will be 5 ft. thick and have a minimum unconfined compressive strength of 250 psi. The top 8 inches will be filled with compacted aggregate, similar to that used in the pad emplacement area.

Q9. Are there conservatisms embodied in the codes and standards you referenced and in the manner you applied them in developing the structural design of the CTB?

A9. Yes.

Q10. What are some of the main conservatisms?

A10. A major conservatism that is incorporated in the applicable codes and standards is that stresses resulting from the design basis earthquake are required to be limited to levels below the specified yield point of the materials. It is well known that concrete and steel structures have substantial deformation capacity above and beyond the point of first yielding. Codes used to design conventional buildings, such as the Uniform Building Code, recognize this fact and increase allowable seismic loads for ductile structures. The CTB and the SSCs of interest in it are generally ductile and have significant deformation capacity beyond yield.

Q11. Are there additional design elements that provide further conservatisms?

A11. Yes, the criteria recommended by the NRC in the SRP for ISFSIs (NUREG-1567) for use in the seismic design of structures such as the CTB provide large additional margins against building failure in an earthquake. The CTB is designed to resist lateral force through a series of reinforced concrete shear walls. This design is highly effective in resisting earthquake forces. The conservatisms built into the design of the CTB can be illustrated by comparing the design of the CTB to a structure that would fulfill similar functions designed under conventional structure codes. The use of conventional building codes would result in a structure designed for much lower seismic forces.

Q12. How does the design of the CTB compare to a similar structure that has been built under conventional building codes?

A12. If one were to design a building of the same general design as the CTB in accordance with the Uniform Building Code (1994) ("1994 UBC") (which was the version of the UBC in effect at the time the license application for the PFSF was filed) under the most conservative assumptions possible, i.e., if it were located in the most severe earthquake area in the continental US (Seismic Zone 4, $A_v = 0.40$); the location had the worst soil conditions (Soil Profile Type S_4); and the facility had the highest Seismic Importance Factor ($I = 1.25$, hazardous facilities), the combination of these conditions would require that the building be designed for a base shear force of 0.23 times the building weight above the base. By contrast, the PFSF CTB has been designed for a base shear force of approximately 1.17 times the weight above the base. In other words, the CTB has been designed for seismic forces 5 times those for which a conventional structure would be designed, assuming that structure was subject to the most severe seismic design requirements possible under the 1994 UBC.

Q13. How would the design of the CTB compare to a similar structure designed under conventional building codes for Utah?

A13. Since Utah is located in Seismic Zone 3 (and 2), the CTB at the PFSF has been designed for seismic forces almost 7 times those for which a conventional structure located in Utah would have been designed for under the 1994 UBC and previous codes.

Q14. Are conservatisms incorporated into the designs of other components in the CTB?

A14. Yes. The applicable seismic load combinations for the cranes within the CTB are described in Section 3.2.11.5 of the SAR. Allowable stresses on the cranes are conservatively limited to the allowable levels of ASME NOG-1.

These cranes are designed to the same design codes as a crane that would be installed at a nuclear power plant and are, therefore, the same, to the extent applicable, as those specified in the NUREG-0800, the SRP for nuclear power plants.

Q15. Is there reserve capacity in the CTB and the structures it contains that would allow them to resist seismic loadings beyond those from the design basis earthquake?

A15. Yes. Reserve capacity exists due to many factors, including, but not limited to: a redistribution of stresses from highly stressed areas of the structure to adjacent areas which occurs after yielding; the fact that the actual material yield strength (for concrete, the compressive strength) exceeds the nominal yield strength values; and the fact that the materials' ultimate strength is significantly greater than its yield strength. Additionally, the seismic loads are of short duration and reverse direction several times each second. Thus, even where some yielding occurs, the load will likely reverse direction before significant distortion can occur and the stresses will return to the elastic range.

Q16. Are there also conservatisms incorporated in the seismic design of the foundations of the CTB?

A16. Yes. A number of conservatisms are incorporated into the design of the CTB foundations. Those conservatisms are evidenced in the building's safety factors against dynamic sliding, overturning, and bearing capacity failures, as described in Calculation 05996.02-G(B)-13, Rev. 6, *Stability Analyses of Canister Transfer Building* (July 26, 2001) ("CTB Stability Calc. Rev. 6") and in the testimony of Paul J. Trudeau on Part D of Unified Contention L/QQ, filed simultaneously herewith.

Because of these conservatisms, there is no concern about potential overturning of the CTB under beyond-design basis earthquake loadings. The CTB is a very stable structure, exhibiting a factor of safety of 1.95 under design basis (2000-year return period earthquake) loadings. Even if the factor of safety against overturning were reduced to less than 1.0 in a beyond-design basis earthquake, the building would not overturn. There could be some lift-off, but the building would tend to return to contact with reversals of the ground acceleration, thus there would be no safety consequences from the lift-off.

This conclusion can be demonstrated by comparing the CTB to the casks on the storage pads. The casks have a tendency to tip (i.e. lift off) during the 2000-year earthquake, but do not overturn. Because of its more stable configuration (240 feet wide and less than 100 feet high, with much of the mass concentrated at the bottom) the CTB is inherently more stable than the casks, and exhibits no such tendency to tip during the 2000-year earthquake.

Holtec has performed analyses of the pads and casks to evaluate their response to a beyond-design basis, 10,000-year return period earthquake. The analyses indicate that the casks will not overturn in such an earthquake. Since the CTB is more stable than the casks, it can be safely predicted that the CTB will not overturn during a 10,000-year earthquake.

Q17. How would the seismic loadings on the CTB change from the 2,000-year return period design basis earthquake to a 10,000-year return period earthquake?

A17. My understanding, based on accelerations corresponding to a 10,000-year return period earthquake provided by PFS's consultant, Geomatrix Consultants, Inc., is that the free-field ground motion due to the 10,000-year return period earthquake has a peak acceleration estimated to be 70-90% greater than that due to the 2,000-year return period earthquake (depending on the direction of motion).

Q18. Would such an increase in peak earthquake acceleration result in a corresponding increase in building accelerations for the CTB?

A18. No. The building accelerations will not necessarily increase in the same proportion as the free-field ground motion. This is due to several factors. First, the soil strains will be higher under higher accelerations. This will result in a reduction in soil shear modulus and increased soil damping. The seismic analysis of the CTB (Calculation 05996.02-SC-5) clearly shows that the building accelerations decrease considerably as the soil stiffness decreases (based on examination of results from the best estimate, lower bound and upper bound soil cases). Both reduced soil stiffness and increased damping will reduce building accelerations.

Furthermore, at the high ground acceleration levels produced by a 10,000-year return period earthquake, the CTB will exhibit non-linear behavior, with the building sliding on and separating from the soil for brief periods of time. Since the vertical acceleration will at times exceed 1.0 g, it is obvious that there will be times that the building will not be in contact with the soil. These non-linear effects will significantly reduce the building accelerations, similar to the manner in which a base-isolated structure behaves under seismic loadings, resulting in no adverse safety consequences.

Q19. What effect do these conservatisms in the design of the CTB have on its ability to withstand a potential building collapse in the event of a beyond-design-basis earthquake?

A19. While the CTB is designed – in accordance with NRC guidance – to withstand the forces resulting from a 2,000-year return period earthquake, due to the structural factors and the mechanics of a beyond-design-basis earthquake, the CTB has a large additional reserve capacity. Given this reserve capacity, the CTB would be expected to survive a much more severe earthquake than the 2,000-year return period earthquake.

A primary concern with respect to building collapse is the potential for collapse of the CTB roof during canister transfer operations. However, the CTB roof has the capacity to withstand accelerations well in excess of those produced by the design basis, 2,000-year return period earthquake for the following reasons:

- The bending moment capacity due to downward loads of a typical girder is 9598 ft-kips, based on N-690 code allowable stresses. The maximum calculated moment is only 6861 ft-kips (71% of capacity).
- The roof bending moment capacity of 9598 ft-kips is based on the N-690 code allowable stresses. The ultimate moment capacity based on the plastic section modulus and minimum material tensile strength is approximately 14,800 ft-kips (54% higher).
- While the studs on the beams and girders have not been designed to provide full composite action, the existing design provides some increase in strength. Fully composite behavior would allow for a vertical acceleration of up to 4 g. I estimate that the existing design can resist a vertical acceleration of at least 3 g.
- The girders are assumed to be simply supported at their ends, where they attach to the N-S walls. Since the girders are connected to the roof slab, and the roof slab is integral with the walls, rotation of the girder will be restrained at the walls, reducing the bending moment at midspan. I estimate that this arrangement increases the load carrying capacity by about 30%.

Due to these factors, the CTB roof should be capable of withstanding accelerations significantly greater than those produced by a 2,000-year return period earthquake without failing.

Q20. Would other SSCs in the CTB also be capable of withstanding a beyond-design-basis earthquake?

A20. Yes. The structural members of the cranes that handle the spent fuel casks and canisters have the same type of reserve capacity as the CTB's structural steel elements. I have learned from a consultant to Stone & Webster with more than twenty years of experience in the design of cranes, including those for nuclear power plants, that the CTB cranes' mechanical components have additional design margins to accommodate increases in seismic loading. The ultimate strengths of mechanical component materials subject to tension and compressive loads are designed such that the ultimate strength of the material is five (5) times that required to support the lifted load. Additionally, if failure of a mechanical component could cause the load to drop, the design of the component is then increased such that the ultimate strength of the material is (10) times that required to support the lifted load. For example, the hoisting cables, as addressed in ASME NOG-1, have a "maximum allowable load" under the design basis earthquake that is less than 40% of the rope's breaking strength. Thus, the cranes to be used in the PFSF CTB would be able to withstand the forces resulting for an earthquake with a return period significantly greater than the 2,000-year return period of the design basis earthquake.

Similar margins exist in the design of the seismic support struts (restraints) for the casks used during canister transfer operations. Those restraints are designed to ASME NF criteria and, therefore, meet the same standards to which comparable nuclear power plant safety-related components are designed. Thus, under code acceptance criteria, the nominal capacity of the seismic struts is 400 kips. The maximum strut load due to the 2000-year return period earthquake is 395 kips. While this would seem to push the capacity of the struts, there is additional conservatism built into the design. Based on an evaluation of the critical components of the seismic strut assembly (tie rods, tie rod welds, strut pins, strut pipe strut pipe end welds, and bracket welds), the ultimate strut load capacity is at least 571 kips. Thus, the seismic struts used in the PFSF CTB will be able to withstand the forces resulting from an earthquake with a return period significantly greater than the 2,000 years of the design basis earthquake.

Q21. What is your conclusion about the survivability of the CTB and the important-to-safety SSCs it contains in the event of a beyond-design-basis seismic event?

A21. The CTB and all important-to-safety SSCs it contains possess far greater seismic loading capacities than those for which they were nominally designed. In addition to the margins that can be explicitly calculated (as discussed above), there are also margins which are known to exist but which are not easily quantifiable. For example, as discussed above, steel structures have reserve capacity above the onset of yielding due to, among other things, the redistribution of stresses -- from highly stressed areas to adjacent areas -- which occurs after yielding. This combination of quantifiable and non-quantifiable margins provides a great degree of assurance that the structures will be able to perform well beyond their design limits. Consequently, there is no doubt that the CTB and the important-to-safety SSCs it houses can withstand acceleration levels well in excess of those associated with the design basis earthquake and have a high likelihood of surviving without loss of safety function in an earthquake with a return period significantly greater than the 2000 years of the design basis earthquake.

III. RESPONSE TO STATE CLAIMS IN SECTION D RELATING TO THE DESIGN OF THE CTB AND ITS FOUNDATION

Q22. In Paragraph D.2 of Unified Contention Utah L/QQ, the State alleges that several deficiencies exist in the seismic design of the CTB and its foundation. Are you familiar with those allegations?

A22. Yes.

Q23. What is your general response to the State's allegations?

A23. The claims raised by the State are hypothetical and are contrary to the guidance in applicable industry standards. Moreover, any potential adverse effect on the seismic performance of the CTB resulting from the factors raised by the State is within the limits of accuracy of the analysis. Any such adverse effect is also made up by other features of the analysis and the design, and is readily absorbed by the factors of safety that exist in the design codes and standards.

Q24. In Subparagraph D.2.a(i) of Unified Contention Utah L/QQ, the State asserts that PFS's calculations incorrectly assume that the CTB mat foundation will behave rigidly during the DBE, an assumption that is alleged to lead to a significant underestimation of the dynamic loading to the mat foundation. How do you respond to these assertions?

A24. Assuming that the CTB mat is rigid is appropriate. Because of its five foot thickness and the stiffening provided by the shear walls connected to the mat, the mat can be assumed to behave rigidly in an earthquake. This is consistent with Section 3.3.1.6 of the industry code that governs the structural design of the CTB, ASCE 4-86, which states: "The effect of mat flexibility for mat foundations and the effect of wall flexibility for embedded walls need not be considered in the SSI analysis." See PFS Exh. XX at 26.

I have reviewed the CTB basemat displacement results of Stone & Webster Calculation Nos. 05996.02-SC-6, "Finite Element Analysis of Canister Transfer Building", Revision 1, which is in the final stages of completion. That calculation shows that, for the loading combination with the full vertical earthquake acting downward (40% of each of the horizontal earthquakes acting, in addition to dead and live loads), the maximum variation of displacement along the centerline of the building in the N-S direction is .164 inches over the length of 279.5 ft. (less than 0.005%) deflection. The maximum variation of displacement in the E-W direction is. 334 inches over the length of 240 ft. (about 0.01% deflection). These small differential displacements further demonstrate the appropriateness of treating the CTB base mat as rigid in the PFS seismic analyses. See PFS Exh. YY.

Q25. State witness Dr. Ostadan testified as follows with respect to the potential flexibility of the CTB mat in his deposition taken on March 8, 2002 at p. 136: "As I indicated before, I would not have raised this issue if we had a good margin under the sliding conditions. I think Holtec or Stone & Webster is on the record that the factor of safety for sliding would be less than one if we do not include soil cement. And then they rely on the soil cement, that we have a number of issues with, to provide the passive resistance. So that, to me, is a slim margin that we have, be it safe or unsafe. Now you talk about the mat being rigid or flexible enough to increase the seismic loads. If it was flexible, it becomes important. Even though the general guidance is not to worry about it. I think it should be viewed in light of the overall design and the margin." How do you respond to Dr. Ostadan's position?

A25. Dr. Ostadan has produced no evidence to suggest that the CTB base mat does not behave rigidly, and he has in fact acknowledged that he knows of no such evidence (Ostadan Dep. Tr. at 137). Industry practice, as reflected in the above referenced ASCE standard, endorses treatment of mats such as this as rigid and the results of the SC-6 calculation discussed above demonstrate that the assumption of rigidity is appropriate. Also, the allowable factor of safety against sliding to which Dr. Ostadan refers as slim is actually 1.1, which in itself represents a 10% design margin, since the onset of sliding will not occur until the factor of safety goes below 1.0. (This factor of safety is set in accordance with the guidance in NUREG-0800, the SRP for nuclear power plants.) Thus, the potential effect of mat flexibility is accommodated by the factor of safety applied in the seismic stability calculations. Ultimately, of course, whether the CTB slides is inconsequential, since the building is free-standing and there are no safety-related components connected to it which could be affected by the sliding of the building.

Q26. In Subparagraph D.2.a(ii) of Unified Contention Utah L/QQ, the State asserts that PFS's incorrect assumption that the CTB mat foundation will behave rigidly during the DBE leads to an assumption that is alleged to lead to an overestimation of foundation damping. What is your response?

A26. As I indicated, the assumption of rigid mat behavior is appropriate and consistent with industry practice. However, even if such an assumption led to some overestimation of foundation damping, there is sufficient margin in other areas of the CTB foundation design to compensate for it.

I would also note that if the frequency-dependent properties of a structure change due to a change in the structure's flexibility, both the stiffness and the damping components of the impedance change. It is not appropriate to look at one aspect (damping) of the impedance without the considering the other (stiffness). The effects of a lessening of the foundation damping would tend to be offset by the effects of the simultaneous reduction that would occur in the structure's stiffness. PFS Calculation 05996.02-SC-5 Rev. 2, "Seismic Analysis of the Canister

Transfer Building," shows that the accelerations experienced by the CTB during a seismic event tend to decrease as the soil stiffness is reduced.

Q27. Dr. Ostadan goes on to testify at p. 137 of his deposition that, if the CTB mat is flexible instead of rigid there will be less radiation damping of the structure, which will result in higher vertical loads. Do you agree?

A27. The significance of mat flexibility hinges on the relative stiffness between the mat and the surrounding soil. Analyses in the literature[2] show that the frequency dependent values of stiffness and damping of the structure are significantly different from the rigid case values only if the ratio of mat-to-soil stiffness is very low, which is definitely not the case for the CTB mat. In particular, for vertical radiation damping, which is the parameter of interest, there is little difference between the rigid case and one in which limited mat flexibility is present. Therefore, even if one assumed that the CTB mat was somewhat flexible, there would be no discernible increase in the vertical loads on the structure.

In addition, a study I performed for the storage cask pads (referenced in Dr. Tseng's testimony) demonstrates that the effects of pad flexibility on the impedance functions are not significant. See PFS Exh. MM. Because of the greater thickness (five feet) of the CTB mat and the stiffening effect of the interior and exterior shear walls, I would expect the effect of potential flexibility on impedance to be of even less significance for the CTB base mat than it is for the pads.

Q28. In Subparagraph D.2.b(i) of Unified Contention Utah L/QQ, the State claims that the PFS calculations ignore the presence of a much stiffer, cement-treated soil cap around the CTB but that the presence of this soil cap impacts the soil impedance parameters. First of all, what are the soil impedance parameters?

A28. They are the frequency-dependent spring and damping parameters that are used to characterize the soil in soil-structure interaction analyses.

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2 M. Iguchi and J. E. Luco, " Dynamic Response of Flexible Rectangular Foundations on an Elastic Half-space," Earthquake Engineering & Structural Dynamics, Vol. 9, No. 3, May - June 1981, Figs. 4 and 5.

Q29. Would the presence of a soil cement layer around the CTB affect the soil impedance parameters?

A29. It might, but any impact would be minimal and can be disregarded in accordance with standard industry practice.

Q30. To what industry practice do you refer?

A30. The soil cement around the CTB is no different than soil backfill, except for being somewhat stiffer. Section 3.3.4.2.4 of ASCE 4-86 states: "For shallow embedments (depth-to-equivalent-radius ratio less than 0.3), the effect of embedment may be neglected in obtaining the impedance functions, provided the soil profile and properties below the basemat elevation are used for the impedance calculations." See PFS Exh. XX at 29. In talking about embedments, the standard is referring to the portion of the soil that surrounds the foundations. The standard is saying that the effect of the soil layer around a foundation can be disregarded in computing the soil impedance for soil structure interaction analyses, if certain conditions are met.

We have complied with those conditions. The depth-to-equivalent-radius ratio for the CTB is less than 0.04, which is much less than 0.3; and actual soil properties below the basemat elevation were used in the impedance calculations. Therefore, the effect of the soil cement around the CTB can be disregarded.

Q31. Dr. Ostadan testified at his deposition (Tr. at 225-31) that the fact that soil cement is present around the CTB foundation makes a difference in the values of soil impedance parameters such that the code guidance does not apply. How do you respond?

A31. I do not believe the distinction Dr. Ostadan is trying to draw is a sound one. The guidance in the ASCE standard would allow us to ignore embedments even if we had 40 feet of compacted backfill around the building. Five feet of soil cement should have less impact on the impedance calculations than 40 feet of backfill. For that reason, it is appropriate to ignore the contribution of the soil cement layer around the CTB foundations in the soil impedance calculations.

Considering the issue from the physical standpoint, the main interface between the CTB and the subgrade occurs at the base of the foundation mat. Energy radiates downward and outward into the soil at this interface. The presence of a soil-cement cap around the CTB has no effect on this energy-dissipation mechanism, which is directed downward and not in the horizontal direction.

Looking in particular at the vertical and rocking components of motion, I cannot envision how the presence of soil cement would have any impact at all on those components of motion. As State witnesses have pointed out, the settlement of the CTB relative to the soil-cement will cause vertical cracks at the mat-soil cement interface. After that happens, the soil cement will not be able to influence vertical and rocking movements of the mat, i.e., the mat will be able to move up and down relative to the soil-cement.

Finally, I would note that the SSI analysis is done with three sets of impedance functions to cover possible variations in soil properties, and the most conservative (least favorable) results are used for design of the CTB. This enveloping technique accounts for any minor variations in soil impedance, caused by soil cement or other conditions.

Q32. In Subparagraph D.2.b(ii) of Unified Contention Utah L/QQ, the State claims that the PFS calculations ignore the presence of a much stiffer, cement-treated soil cap around the CTB but that the presence of this soil cap impacts the kinematic motion of the foundation of the CTB. In the State's response to Interrogatory No. 14 in the Applicant's Eighth Set of Discovery Requests, the State explains this concern as follows: "The soil-cement and the concrete mat foundation will have significantly different stiffnesses and such contrasts in stiffness (or impedance parameters) will cause kinematic interaction between the soil-cement and the CTB mat foundation. This interaction may lead to overstressing and cracking of the soil-cement placed immediately adjacent to the CTB and renders it ineffective in performing its intended function used for CTB analysis." Is this a valid concern?

A32. No. The input to the CTB seismic analysis includes the free-field motion and the strain-dependent soil properties, both of which were developed by Geomatrix Consultants, Inc. ("Geomatrix"), and I understand Geomatrix included the presence of soil cement in developing these inputs to our analysis. In our dynamic analysis of the CTB, we use the free-field motion as an input located at the top of the profile. However, the applicable motion to use would be the motion at the base of the building foundation, which is at the top of the clay layer. Based on my review of the results of the Geomatrix analyses that developed the strain-dependent soil properties, the seismic motion at the base of the mat (the top of the underlying clay layer) is slightly lower than at the surface; hence, the input provided by Geomatrix is conservative, and so is our seismic analysis.

The issue of the potential cracking of the soil cement and its effect on stability analyses for the CTB is discussed in the testimony of Paul Trudeau on Section D of Unified Contention L/QQ, filed simultaneously with my testimony.

Q33. In subparagraph D.2.d of Unified Contention Utah L/QQ, the State claims that Applicant's calculations unconservatively assume that only vertically propagating in-phase waves will strike the CTB and its foundations, and fail to account for horizontal variation of ground motion that will cause additional rocking and torsional motion of the CTB and its foundations. Should the effect of non-vertically propagating waves have been taken into account in the CTB seismic calculations?

A33. No. This is essentially the same claim raised with respect to the seismic design of the cask storage pads in subparagraph D.1.a of Unified Contention Utah L/QQ. That claim is refuted in the testimony of Robert Youngs and Wen-Shou Tseng on Unified Contention L/QQ, which is being filed simultaneously herewith. The evaluation performed by Drs. Youngs and Tseng led them to conclude that the angles at which seismic waves would impinge the PFSF site are, for all practical purposes, vertical.

In addition, the Commentary to Section 3.3.1.2(a) of ASCE 4-86 Code allows the seismic analyses of structures such as the CTB to assume incoming seismic waves to be vertically propagating waves provided a mass eccentricity factor of 5% is incorporated into the actual design of the structures to address the effects of inclined and incoherent waves. See PFS Exh. XX at 66. S&W is following this recommendation in the design of the CTB, so there is no reason why it would need to account in the seismic analyses of the building for non-vertical propagation of seismic waves.

Q34. Do you know whether the State witnesses agree with your position?

A34. Yes. State witness Dr. Ostadan testified in his March 8, 2002 deposition at p. 78-79 that if an accidental mass eccentricity factor was included in the design of the CTB, there was no need to consider the potential for non-vertical propagation of seismic waves.

Q35. Does that conclude your testimony?

A35. Yes, it does.