RAS 4529

Official Transcript of Proceedings

NUCLEAR REGULATORY COMMISSION

Title: Private Fuel Storage, LLC

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

Location: Salt Lake City, Utah

Date: Wednesday, May 1, 2002

DOCKETED
USNRC
2002 JUN 13 PM 12: 43
OFFICE OF THE SECRETARY
RULEMAKINGS AND
ADJUDICATIONS STAFF

Work Order No.: NRC-281 Pages 5977-6226

NEAL R. GROSS AND CO., INC.
Court Reporters and Transcribers
1323 Rhode Island Avenue, N.W.
Washington, D.C. 20005
(202) 234-4433

Template=SECY-032

SECY-02

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
Shearton Hotel, Wasatach Room
Salt Lake City, Utah 84114

On May 1, 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

APPEARANCES

FOR THE STATE OF UTAH:
Denise Chancellor, Esq.
Connie Nakahara, Esq.
James L. Soper, Esq.
ASSISTANT ATTORNEYS GENERAL
Office of the Attorney General
160 East 300 South, 5th Floor
P. O. Box 140873
Salt Lake City, Utah 84114

FOR PRIVATE FUEL STORAGE, LLC:
Paul Gaukler, Esq.
Mattias Travieso-Diaz, Esq. — Blake Nelson Esq.
SHAW PITTMAN
Attorneys at Law
2300 N Street, N.W.
Washington, D.C. 20037

FOR THE U.S. NUCLEAR REGULATORY COMMISSION:
Sherwin E. Turk, Esq.
Martin O'Neill, Esq.
OFFICE OF THE GENERAL COUNSEL
Mail Stop - 0-15 B18
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

I N D E X

E X A M I N A T I O N

E X H I B I T S

9:00 a.m. May 1, 2002

P R O C E E D I N G S

JUDGE FARRAR: Good morning, everyone. It's 10 after 9:00. We were here ready to start, but the parties were engaged in what appeared to be a major discussion about some evidence. We wanted to let that discussion take place because, as we've seen, the parties are often able to work out things cooperatively better than we could by ruling on them. If you all want to tell us what that was all about.

MS. CHANCELLOR: Certainly. Last night when I got back to the office about seven o'clock I found this CD-ROM in a transmittal letter from Holtec, and the transmittal letter relates to PFS seismic analysis pad interface forces, and it's a transmittal letter transmitting this CD-ROM which has a whole bunch of zip files, a whole bunch of data. These Holtec force time histories were developed in the 2,000-year cask stability analysis and these are four time histories that we have requested from Dr. Soler and Dr. Singh during their deposition in March.

MR. GAUKLER: Requested from Dr. Wen Tseng.

MS. CHANCELLOR: Oh, Dr. Tseng, Dr. Wen Tseng. There are too many "sings." We requested it from Dr. Wen Tseng and it is a critical calculation. It's a calculation that Holtec did and gave to Dr. Wen Tseng for the design of the pad. There is no -- the only piece of paper we got was a transmittal letter. Mr. Gaukler tells me there's like a three-page description of what is on these -- on the CD-ROM. And it's going to take our experts some time to review this data, look at the -- where Holtec produced the forces, on what part of the cask and the pad and how Dr. Tseng applied those forces in his calculation.

So I think the most expedient resolution of this is to allow us to cross-examine -- we'll certainly need to cross-examine Dr. Soler as to how he produced these force time histories. We may also need to cross-examine Dr. Wen Tseng as to how he applied them. And what I suggest is that we continue on today and that they come back. And I would also request that we be permitted to supplement our direct testimony if necessary. It would be the testimony of Dr. Bartlett and Dr. Ostadan for dynamic analysis.

JUDGE FARRAR: Supplement it to include

their response to this information?

MS. CHANCELLOR: Supplement it solely with respect to this information. "This information" being the CD-ROM and any accompanying documents that Mr. Gaukler gives me.

JUDGE FARRAR: Mr. Gaukler?

MR. GAUKLER: Yes. I just want to state a little more background. In general, the type of data that is on the CD-ROM is described in Dr. Wen Tseng's ICEC calculations and there are some simple plots there from the data, just for background purposes, but I agree that they got the CD last night. For various reasons I won't get into it. So we don't have any objection to their looking at the data. Right now I have tentative Dr. Wen Tseng and Dr. Soler here for the cross-examination of Dr. Khan and possibly Dr. Bartlett, and we'll see how their review is. Hopefully they'll be done by that time frame to follow up at that point in time with Dr. Singh and Dr. Soler. And I would want to be convinced that any additional testimony is really -- was due to the new information in the CD. Assuming that to be the case, I wouldn't object to that. But obviously --

JUDGE FARRAR: You can't commit to that

right now.

MR. GAUKLER: Without reviewing it. And also -- yeah. We might also put it in in rebuttal too, is another possibility.

JUDGE FARRAR: Does the Staff have any position on this?

MR. TURK: No. We view this as a matter between the parties.

JUDGE FARRAR: Speaking of when witnesses are showing up, are we looking at plan A or plan B.

MS. CHANCELLOR: Plan Z, your Honor.

JUDGE FARRAR: If you haven't arrived at anything then let's not go over it.

MR. GAUKLER: Yeah. I would suggest that -- well, we have some stuff to talk to the Board about. I would suggest, given Dr. Singh's schedule, that we defer to that until we're done with the testimony of Dr. Singh and Dr. Soler, if that's okay with the State.

MS. CHANCELLOR: That's fine.

JUDGE FARRAR: Then we'll go ahead with the continuing State's cross-examination. As we discussed yesterday, we do have a witness availability problem. And while that's not paramount, we will try during the course of the proceeding to extend the same courtesy to any party's' witnesses. Before you start, Ms. Nakahara, Mr. Turk, do you have something?

MR. TURK: Yes, your Honor. Just a housekeeping matter. I would like to introduce to the Board two other members of the Staff who were here yesterday and are with us again today. I would ask them simply to stand as I introduce them. First Mr. E. William Brach, who is Director of the NRC Spent Fuel Project Office. And with him is Mr. Stephen Baggett who is a project manager on the Spent Fuel Project Office with current responsibility, while other facilities or projects, the Diablo Canyon project.

MS. CHANCELLOR: Your Honor, just to clarify, I assume that you did rule that that was acceptable that we supplement our testimony and that we have the opportunity to cross-examine Dr. Singh and Dr. Soler to the extent we need to based on our review of this late information?

JUDGE FARRAR: What we intended to rule was that you certainly have time to review that to cross-examine later, bring the witnesses back and cross-examine them whenever you're prepared to do.

So on the matter of whether you can supplement your testimony, in theory you can do that. I think Mr. Gaukler was just saying he would want to make sure that your supplement was limited to the new information. But certainly if it were, then we would be inclined -- we would let you do so.

MS. CHANCELLOR: Okay. Thank you, your Honor.

JUDGE FARRAR: In other words? Wait a minute.

MR. GAUKLER: Where there was testimony that was giving new information as opposed to what they already had. One minor thing, I think the current instruction letter is on the CD itself, but we'll get you a hard copy as well.

MS. CHANCELLOR: At ten o'clock last night that was just a blow.

MR. GAUKLER: I understand that.

JUDGE FARRAR: Well, then let's get going. We promised Dr. Singh, what was it, two o'clock, to get him out of here?

DR. SINGH: Yes.

JUDGE FARRAR: All right. Go ahead, Ms. Nakahara.

MS. NAKAHARA: Thank you, your Honor.

CROSS-EXAMINATION (Resumed)

BY MS. NAKAHARA:

Q. Good morning, your Honors. Good morning Dr. Singh and Dr. Soler.

DR. SINGH: Good morning.

DR. SOLER: Good morning.

Q. I'm Connie Nakahara. I apologize for my voice. I represent the State of Utah.

Dr. Singh, if you'll turn to your response to Answer 19, isn't it true that you state that ASME, the American Society of Mechanical Engineers Boiler and Pressure Vessel Code governs the design of pressure vessels for safety-related applications at nuclear power plants?

DR. SINGH: In general that is correct.

Q. All safety-related pressure vessels at nuclear power plants are rigidly connected to the foundation; isn't that correct?

DR. SINGH: No, that's not correct.

Q. What examples do you have?

DR. SINGH: Well, examples, actually, are quite contextual to these proceedings. All safety-related equipment where spent fuel is stored, almost all of that equipment at the present time in the United States and most countries overseas has spent fuel -- I mean the equipment that is used to store spent nuclear fuel in the spent fuel pools are free-standing and they are safety-related, they are Seismic Category 1, and I mean practically all of it.

JUDGE FARRAR: Dr. Singh, would you pull the microphone a little closer, please?

Q. (By Ms. Nakahara) Are spent fuel racks classified as pressure vessels?

DR. SINGH: No, they are not pressure vessels. They are safety-related and they are seismic Category 1.

Q. So isn't it true that there are no pressure vessels, safety-related pressure vessels at nuclear power plants that are not rigidly connected to the foundation?

DR. SINGH: Well, realize that the HI-STORM is not a pressure vessel either, okay? If you're inferring that HI-STORM is a pressure vessel, it is not.

Q. But in Answer 19, isn't it true that the HI-STORM meets the code governing design of pressure vessels?

DR. SINGH: Perhaps you're confused. Let me attempt to explain. The ASME Boiler and

Pressure Vessel Code has many subsections. The subsection NF that applies to linear structures, that is not a pressure vessel, even though it's under the umbrella of the ASME Boiler and Pressure Vessel Code. There are codes within the ASME code that pertain to non-pressure vessels. Subsection NF, NF in quotes, is not a pressure vessel code, it's a structural code. And the HI-STORM system is engineered to meet the stress limits of the NF code.

Q. Isn't it true only high density racks are unanchored, spent fuel racks?

DR. SINGH: I would not make that bold of a statement. I cannot say that for the universe of equipment used at nuclear power plants.

Q. If you'll turn to your response to Question 27, yesterday Mr. Soper asked you some questions with respect to your seismic analysis at the facilities you list in your Answer 27. With respect to the site-specific analysis conducted, isn't it correct that the storage pads at Diablo Canyon will be embedded in soft bedrock?

DR. SINGH: I didn't hear the last few words.

Q. Isn't it correct that the storage pads at Diablo Canyon will be embedded in soft bedrock?

DR. SINGH: I would say they are founded on soft rock, to my knowledge. I did not -- we did not design the ISFSI pad for Diablo Canyon, but my understanding is that the pad is founded on rock.

Q. Are the storage pads at Dresden supported by soil cement on a clay layer?

DR. SINGH: No, there is no soil cement at Dresden.

Q. What about at Columbia Generating Station?

DR. SINGH: No, no soil cement is used at either Dresden or at Columbia Generating Station, to my knowledge.

Q. What about at Fitzpatrick?

DR. SINGH: Again, to my knowledge, soil cement has not been used.

Q. And what about at Sequoyah Fuels?

DR. SINGH: Sequoyah's pad has not been established yet, but the design of the pad which we have produced does not call for use of soil cement.

Q. And is the site at Sequoyah, could it be classified as a clay site, a clay-layered site?

DR. SINGH: I couldn't tell you for sure. After a while they all go together in my mind.

JUDGE FARRAR: Speaking of going together, we're losing some of your words. If you could put the microphone maybe between you and the witness -- or between you and counsel so you could look at her and talk into the microphone at the same time.

DR. SINGH: All right. Thank you for the direction, Judge.

Q. (By Ms. Nakahara) Dr. Soler, if you have in front of you PFS Exhibit 86 -- do you have the Exhibits from yesterday?

DR. SOLER: Unfortunately I gave them back so just give me a minute. Okay.

Q. Do they also have State's Exhibit 173, the Multi Cask Response?

DR. SOLER: You had better give me the other one too.

JUDGE FARRAR: And while we're doing that, Ms. Nakahara, PFS Exhibit 86, would you say for the record here what that is.

MS. NAKAHARA: It's the PFS Beyond Design Basis Scoping Analysis.

JUDGE FARRAR: And that's the document that was marked and then retrieved yesterday. So this might be a good time, I think Mr. Gaukler has already passed out the new copies, to have the reporter mark that. It turns out she already has. And just so all this is on the record in one place, we're going to talk about PFS Exhibit 86 and State Exhibit 173. Both of those are Company proprietary documents. We have made no ruling on whether they are, in fact, proprietary until someone challenges it, we just take them as that way.

But what happened yesterday was the Board sensed that the witnesses were waiving the claim of proprietary because they thought they had to in order to bring it into this proceeding and we advised them that they did not have to do that, that there were procedures we could invoke to continue with our proceeding and to protect the proprietary nature of the documents. They took that opportunity.

So as far as we're concerned, these have the same claim of proprietary nature that they did before we started 24 hours ago. So on that basis everyone will protect these copies of the documents that they have. The agreement that counsel reached was that while the documents would be protected, the discussion about them in the transcript did not have to be sealed or kept in confidence and we'll proceed on that basis.

Q. (By Ms. Nakahara) Dr. Soler, isn't it true that you are the sole author of PFS Exhibit 86?

DR. SOLER: That's the Beyond Design Basis?

Q. Yes.

DR. SOLER: That is correct.

Q. And, Dr. Soler, isn't it true that you are the current author identified in State's Exhibit 173, Multi Cask Response at PFS ISFSI for 2,000-Year Seismic?

DR. SOLER: That is correct.

Q. Is it also true that Revision 0 author to that document is Chuck Bullard?

DR. SOLER: That is correct.

Q. Dr. Soler, isn't it true that you are the sole author identified in the 1997 Multi Cask Response, Seismic Response of the PFS ISFSI?

DR. SOLER: I don't have that in front of me but I believe, to the best of my knowledge, that I'm identified as the author.

Q. Thank you. In Answer 41 in your response to Question 41, isn't it correct that you state the effect -- I'll wait. I apologize. Isn't it correct that you state that the effect of soil, soil cement foundation was modeled by appropriate springs and dampers characterizing the soil resistance in deflection and rotation?

DR. SOLER: That is correct.

Q. By modeling the soil springs and damper, is that how you accounted for the soil structure interaction?

DR. SOLER: That is how we included the soil in our dynamic model, yes.

Q. And is it also correct you use the soil spring and dampers provided in the Multi Cask Response at PFS ISFSI, States's Exhibit 173, the soil --

DR. SOLER: We were provided with the moduli of the soil appropriate to the earthquake. The formulas that we used to develop the springs were not provided to us by Geomatrix.

Q. Yesterday Mr. Turk asked you questions about the HI-STORM 100 CoC, Certificate of Compliance. I would just like to clarify an issue. Because the CoC is a generic license, isn't it correct that soil structure interaction effects are not included in that license?

DR. SOLER: That is correct.

Q. Is the soil spring and damper concept that you use in your model -- strike that.

Is the soil spring and damper concept that you use in your model for the Multi Cask Response at PFS ISFSI for the 2,000 year the same method you use in all the Holtec seismic analysis?

DR. SOLER: Would you clarify what you mean by "all the Holtec seismic analysis."

Q. The ones you listed in I believe it was Answer 27, which would include Diablo Canyon, the site-specific cask stability analysis.

MR. GAUKLER: What page is that?

MS. NAKAHARA: I'm looking. It's Answer 27.

DR. SOLER: Okay. Now what was your question again?

Q. (By Ms. Nakahara) Is this the same model, the same concept to account for soils -- strike that.

The soil spring and damper concept you used in the Multi Cask Response for 2,000 Year at PFS, is that the same concept you used in these analyses to account for soil structure interaction effects?

MR. TURK: I'm sorry, in which analysis?

MS. NAKAHARA: In Answer 27, which would be the Diablo Canyon. Let's just start with Diablo Canyon.

DR. SOLER: Soil springs were used in the TVA, Sequoyah, analysis. Diablo Canyon did not use -- the SSI analysis was not done by us. The time histories were provided to us. The fact that the pad is thick and founded on rock negated the use of soil springs for any of our analysis. The dynamic analysis of the Dresden plant, we had specified accelerations there at the base of the pad. And I believe, to the best of my recollection, that was also true for Energy Northwest and J.A. Fitzpatrick.

Q. (By Ms. Nakahara) Do you know what methodology was used to derive the accelerations at the pad for Dresden?

DR. SOLER: To the best of my knowledge, I don't know the methodology, but it satisfied the appropriate SRP, Standard Review Plan.

Q. Do you know the methodology that was used to derive the accelerations at Fitzpatrick?

DR. SOLER: We don't.

Q. And do you know the methodology that was used to develop the accelerations at Diablo?

DR. SOLER: No, I do not.

Q. Dr. Soler, will you describe your experience in analyzing soil dynamics in foundation design.

DR. SOLER: Other than the work that I have done for these ISFSIs, my expertise is not in the design of foundations and soils.

Q. Will you describe your experience in analyzing the effects of soil structure interaction.

DR. SOLER: My experiences on this project?

Q. In general.

DR. SOLER: In general and on TVA, Sequoyah, and then an internal report that I wrote generic to work we were going to do, but that is the extent of my work in soils.

Q. Thank you. Does Mr. Bullard have experience -- is it Mr. Bullard or Dr. Bullard?

DR. SOLER: Mr.

Q. Does Mr. Bullard have experience in analyzing soil dynamics and foundation design?

DR. SOLER: Mr. Bullard did not need that experience to do what he had to do in the project.

Q. Does Mr. Bullard have any experience in analyzing the effects of soil structure interaction?

DR. SOLER: What he gained on these projects that he worked on is the sum total of his experience.

DR. SINGH: Can I supplement the answer, if you will allow me?

MS. NAKAHARA: Yes.

DR. SINGH: I think the questions you're asking and the answers may not quite illuminate the matter properly. We do not -- we are not soil people in the sense that we do not develop properties, we do not run tests and characterize soils and develop properties of soils. We do, however -- we do, however, once we get the soil characterized by appropriate people, analyze the interaction between the soil and the structure attached to it using classical mechanics techniques.

You had asked earlier NRC did not include the soil structure interaction in the certificate. That is true. But there is in our FSAR the statement that we recognize the effect of soil structure interaction and when we began work on studying the interaction between the soil, again, characterized by others, we simply make the mathematical model for it.

In doing that work, we wrote to the NRC in a separate letter informing them of our broad methodology and we used that approach in the projects that we do. Because after all, the FSARs, every document is finite in the amount of information it has. And in this period of open communication with the NRC, when we have an area where there has not been in-depth discussion with the regulators, we would like a letter or we use some means, official means to communicate as to the work we do. And that's what we did in the area of soils structure interaction.

Q. With respect to the methodology to develop soil springs and dampers in ASME, isn't it correct that's an approximation method to account for soil structure interaction effects?

DR. SINGH: Practically all methods --

MR. GAUKLER: Just for clarification of the record I think counsel meant to say -- you said ASME and I think you meant to refer to something else, ASCE.

MS. NAKAHARA: ASCE, thank you.

DR. SINGH: It should be ASCE. The procedures given in industry standards typically are conservative, they are approximate. And I will further go and tell you that if anyone tells you that a solution is exact using computer programs they are misstating the fact. Every solution has approximation built into it. That is the nature of engineering solutions.

Q. (By Ms. Nakahara) Dr. Soler, if you have a non-sliding condition -- or for a non-sliding condition, have you compared the reaction loads applied at the cement-treated soil interface for your soil springs and damper with a soil structure interaction code such as SASSI?

DR. SOLER: SASSI would not be applied, could not be applied to this project.

Q. But it could be applied to a non-sliding situation, correct?

DR. SOLER: The sliding or non-sliding is immaterial. It still couldn't be applied to this project because the casks and the pad interaction is nonlinear and the codes that I know of that are devoted to soil structure interaction in the manner which you're alluding are linear codes.

Q. But couldn't you use SASSI to verify your forces you're seeing at the interface between the pad and the cement-treated soil?

DR. SOLER: In my opinion, no.

Q. Then how do you know the loads that you've applied are an accurate representation of soil interaction effects?

DR. SOLER: The loads that I apply in the analysis are simply the inertial loads from the earthquake. They're applied at the base of the soil springs and propagate through the soil springs into the pad and then into the casks, in a simplified description. So the only loads that we apply are the seismic inertial loads, the accelerations due to the earthquake.

Q. And you're talking about the ones you received from Geomatrix, correct?

DR. SOLER: Geomatrix gave us the time histories, that is correct.

Q. How do you know that the soil springs and dampers that you calculated adequately account for the soil structure interaction effects?

DR. SOLER: In our opinion, they did. Using, in my opinion, using a code that doesn't apply to try to verify a problem solves nothing. Had I rigged up a problem and used SASSI and gotten results that agreed or disagreed, it would have provided no additional information to me. In my opinion, because of the nonlinearity of the problem we were dealing with, the only way to treat it would have been with the soil springs we were using, plus our model of the pad and the casks in terms of lump mass spring systems.

Q. And this is based on your engineering judgment, correct?

DR. SOLER: My judgment and roughly about 40 years of experience.

Q. But didn't you just tell us that the only experience you have with soil structure interaction is this case?

DR. SOLER: That is true. But in the tenor of the analysis we were doing, the effect of the soil was simply masses and springs and dampers. So it's no different than any other dynamic problem.

Q. Do you have any working knowledge of the SASSI code?

DR. SOLER: I have looked through the manual, but that is the extent of my knowledge of it.

Q. Thank you. Dr. Soler, isn't it true in your 10,000-year analysis, I think in response to Answer 39, isn't it true this 10,000-year analysis did not account for the effects of soil structure interaction? And take the time answering, will you?

DR. SOLER: It accounted for soil structure interaction in the same manner that the DYNAMO analysis accounted for it by simulating the effect of the soil by a set of springs. Oh, we are talking about the --

Q. I was going to clarify.

DR. SOLER: The Beyond Design Basis or no?

Q. Is that the 10,000-year you were talking about in this response?

DR. SOLER: I have to -- there are two reports. The first one, you are correct, the first one stated explicitly that the analysis we were doing for that report assumed that the time histories were applied, were driving the pad. So there was no soil structure interaction model in that report.

Q. For the record, and I apologize for not having a copy for you, does the first analysis, the title sound like, or based on your recollection, the Dynamic Response of Freestanding HI-STORM 100 Excited by 10,000-Year Return at PFS --

DR. SOLER: That sounds --

Q. -- dated November --

DR. SOLER: Actually, that references Reference 11-1 in the Beyond Design Basis Scoping Analysis.

Q. And that is the correct title?

DR. SOLER: That is the correct title, yes.

Q. Thank you. If you'll turn to your response to Question 55, isn't it correct you discuss the State's concern that Holtec used a static, and you refer to it as a small strain Young's modulus in this response?

DR. SOLER: No, you've got it backwards. The static modulus is a large strain modulus.

Q. You're correct. Our concern is that you used this -- no, strike that. Anyway, does this response deal with Young's modulus?

DR. SOLER: Yes, it does.

Q. Thank you. Isn't it true in this response you state the soil strain calculated in

Holtec's tip-over drop analysis is 1.93 percent?

DR. SOLER: That is correct.

Q. Isn't it also true the soil strain calculation assumes a non-mechanistic tip-over?

DR. SOLER: That is also correct.

Q. And isn't it also true PFS has no cement-treated soil test data for either a small or large strain?

DR. SOLER: That I cannot attest to, but I -- that may be true. I do not -- I'm not involved with what soil testing has or has not been completed at this stage.

Q. And isn't it true that you did not quantify the effects of cask response in using a small strain Young's modulus on cask response?

DR. SOLER: Which response, the drop and tip-over response?

Q. No, the cask sliding tip-over -- cask sliding stability?

DR. SOLER: In the stability analysis the moduli were given to us. To my understanding, that moduli included that effective modulus for us to calculate soil springs included a layer of soil cement.

Q. If you'll look at your response to

Question 57, is it true towards the bottom half of that response you state, "One accepted methodology for bounding the seismic -- one accepted methodology for bounding the effects of non-vertical seismic waves is, in fact, to deliberately induce a 5 percent loading eccentricity into the model to account for rocking and torsion effects"?

DR. SOLER: That is correct, yes.

Q. Is it your opinion that Holtec introduced eccentricities by analyzing the seismic response of less than 8 casks on a pad?

DR. SOLER: Yes, it is.

Q. Did you account for eccentricities in any other way in the cask response?

DR. SOLER: Did I or could I?

Q. Did you.

DR. SOLER: In the DYNAMO runs the only eccentricities to the global model were those from using different numbers of casks on the pad, different simulations.

Q. Holtec did not induce an additional 5 percent load -- strike that.

At any one time wouldn't it be normal operations to have between 1 to 8 casks loaded on a pad?

DR. SOLER: I believe that any time during the course of a loading campaign you could have less than 8 casks on a pad.

Q. So Holtec did not induce an additional 5 percent loading eccentricity above that expected for normal operations, right?

DR. SOLER: Well, to me that 5 percent that is stated is not really meant to cover this situation. Generally speaking, you're talking about a building where you may want to introduce a 5 percent irregularity in some manner in that building model to account for these effects. I don't think my opinion, that that statement ever anticipated this kind of a problem.

Q. Isn't it true you have thought quantified the actual effects of non-vertically propagating waves on cask response?

DR. SOLER: I would say we bounded them probably by our Beyond Design Basis Analysis with all the conservatisms that we put in them.

Q. But you have not, in fact, quantified them; is that correct?

DR. SOLER: I have not been given a set of time histories which included non-vertical waves, no.

Q. Thank you. Dr. Soler, in response to -- excuse me. Dr. Soler, in response to Question 60 to 61, is it correct you discussed the analysis of Tennessee Valley Authority's Sequoyah Nuclear Power Plant?

DR. SOLER: I was busy turning the page there. I hate to make you repeat the question, but--

JUDGE FARRAR: Lanette, would you read that back, please?

(Pending question read.)

DR. SOLER: That is correct.

Q. (By Ms. Nakahara) Dr. Soler, what is the zero period acceleration at Tennessee Valley?

DR. SOLER: At the top of the pad, and this is from memory only, at the top of the pad I believe it was in the order of .45 to .55. I can't be sure.

Q. And I may not use this as an Exhibit, so if we can withhold from marking it right now. And this is also a proprietary document.

Are the TVA storage pads located on a clay-layered site?

DR. SOLER: I do not recall exactly how

the soil is characterized there.

Q. Isn't it correct the soil structure -- strike that. The document that Ms. Chancellor just handed you, is it correct these are various pages from Revision 1 of HI2012727(3), which includes a cover page, Page 17, 20, 21. It also includes pages from Revision 2 of HI2012727(4), which includes a cover page, Page 17, 21, 22, 23, and pages D-9 through D-24 from the Appendix D, Rev 2 of HI 2012727?

DR. SOLER: That's correct.

Q. Are you familiar with this document?

DR. SOLER: Yes.

Q. Isn't it true this document or somewhere in this document the soil structure interaction effects at the Tennessee Valley site are modeled as competent bedrock?

DR. SOLER: The time histories are imposed at competent bedrock, but soil spring effects were included between competent bedrock and top of ground -- or top of pad.

DR. SINGH: The bedrock varied between 46 feet to --

DR. SOLER: 46 to 80 -- actually, 33 to 83 feet I believe is the correct number.

DR. SINGH: Below the pad.

Q. What are the estimated long-term settlement of the storage pads at the Tennessee Valley site, if you know?

DR. SOLER: I do not know.

Q. Isn't it true that the pads are not supported by soil -- by cement-treated soil?

DR. SOLER: As was stated earlier, the pads have not been finally designed yet so I can't answer that.

Q. Are you aware of whether pad deflection, whether the pad deflection was determined using a finite element analysis? Strike that. Let me start over. Are you aware of whether pad deflection was determined at the Tennessee Valley site?

DR. SINGH: Let me answer that. The pad was analyzed, the pad was analyzed as a finite -- using a finite element program and the soil was simulated by a set of springs to analyze the pad. We do not as a practice make an attempt to predict long-term settlement. We can only conjecture. Even the concept in operating ISFSIs is that the long-term behavior of the pad is part of monitoring program for the ISFSI operator. And the settlement, we concern ourselves with making sure that the settlement, if it were to exceed a certain value, then the plant operator will take remedial action. It is not a stress evaluation effort.

Q. Do you agree that long-term settlement may deform the surface of the pad?

DR. SINGH: Over long term there may be some settlement depending on the manner in which the pad would be loaded, depending on the geotechnical conditions at the site, it may occur. Soil, as we all know, does creep under stress and it's possible for limited settlements to occur.

MR. GAUKLER: I want to put an objection on the record in terms of being beyond the scope of the direct.

JUDGE FARRAR: The same ruling as usual.

MR. GAUKLER: I understand. I just want to put it on the record, your Honor.

JUDGE FARRAR: That's fair. We'll overrule the objection. Let me interrupt here. Miss Nakahara, we can't help but notice you've been struggling with your voice for the past half hour. I seem to remember an obscure provision in the rules that says on occasion experts are allowed -- expert nonlawyers are allowed to conduct questioning. That's an option. Since you are being advised by some experts that's an option we would make available to you under the physical circumstances of your voice problems. If you were to say you wanted to accept that option we would then be forced to listen to arguments from the other side, from the other parties about whether that was legitimate or not. But let's go off the record and discuss how we might handle this.

(A recess was taken.)

JUDGE FARRAR: Let's go back on the record. During the break we discussed how best to move forward and counsel for the State is willing to soldier on and apparently does not feel as bad as her voice sounds. So we will go forward on that basis.

MS. NAKAHARA: Thank you, your Honor. It's harder on everybody else to listen to me.

Q. (By Ms. Nakahara) Dr. Soler, in conducting your cask sliding analysis, will you describe the type of surface you used for the top of the pad? For example, was it perfectly flat?

DR. SOLER: I need a little clarification as to your question. Are you talking about the numerical analyses as such in the Multi

Cask Response Report when you're talking about cask sliding analysis?

Q. Yes.

DR. SOLER: There we assume in our system that a coefficient of friction exists between whatever points of the cask are in contact with the pad. We -- the pad, of course, starts out at a certain initial condition in terms of its position and orientation depending upon how many casks we have placed on the pad for that run.

Q. So based on that response, if you have less than 8 casks, did you consider any pad tilting?

DR. SOLER: Initially, yes. The initial position of the pad, given the soil springs that are in place under the pad and given the dead weight of the casks on top of the pad at their respective locations, the initial equilibrium position of the system is calculated, and that includes an initial downward deflection of the pad plus two rotations, which of course are small, but they are included.

Q. Did that initial condition reflect any deflections due to settlement?

DR. SOLER: No. They just reflected the

dead weight. In other words, the pad was started in a condition where they had a dead weight on them in a certain location and a reaction from the soil springs.

DR. SINGH: Small settlements, in our technical opinion, will have a second order effect on the response of the casks.

Q. Have you ever quantified that for the PFS site?

DR. SINGH: Well, there are settlement guidelines in the codes that provide the acceptable settlement of structures based on their plane form dimensions. And I don't remember -- I have not memorized them, but those are the restrictions one uses in designing pads also.

Q. When you're talking about codes, what codes are you talking about?

DR. SINGH: That would be ASCE, ACI. I can't tell you off the top of my head, but there are specified acceptance limits for settlement of foundations and we recommend that those be met in pad designs.

Q. Do you know if those codes applied to the cask? Are the free-standing objects allowed to slide?

DR. SINGH: Yes. There is no reason why those restrictions -- they are good, sound, experienced-based limitations on permitted settlement. There is no reason why they wouldn't apply to free-standing. After all, many buildings could be considered freestanding structure. As Dr. Soler explained yesterday, they are on vibration isolation pads, they are not necessarily anchored.

Q. How did those codes apply to a nonlinear analysis such as your cask stability analysis?

DR. SINGH: I don't see any conflict between the two. The nonlinear analyses that we perform capture the essential elements of the structure, essential characteristics of the structure to get a response with a high level of reliability with respect to its accuracy. The data, such as small settlement, flexibility considerations of the pad, they are second order effects. They do not modify the response of the casks in a significant manner.

Q. Dr. Singh, let's go back to the Tennessee Valley documents you provided us, selected pages, and that analysis, the flexibility of the pad. Refresh my memory, did you say that the overall pad deflection was determined?

DR. SOLER: Are you directing that question to me? Are we back on Answer 60 and 61?

Q. Yes.

DR. SOLER: Okay. Now --

Q. Refresh my memory, did one of you say that you quantified the maximum amount of pad deflection when you designed the pad?

DR. SOLER: Our analysis is a two-part -- well, first of all, there's a three-part analysis that underlies all of this. First of all, there is a quantification of the soil properties at the site. That was not under our supervision. We were provided with appropriate data sufficient for our dynamic analyses such as shear moduli, Young's moduli, variations with respect to the soil had different depths. From that we developed the soil springs that we were going to use in our model.

To reflect the effect of the fact that there were varying depths of soil, we looked at both extremes of a deep soil and a shallow soil, carried out a number of evaluations using lower bound properties, best estimate and upper bound properties, and developed our dynamic model to predict the interaction forces between the pads and the casks, and then took those forces and applied them to a finite element model of the pad itself to evaluate the stresses in the pad.

Q. When you evaluated the stresses in the pad, did you determine a maximum deflection?

DR. SOLER: The pad was treated as an elastic body in that case, but the rigid body deflections from the dynamic analysis were not included in that analysis. It assumed a pad sitting on a foundation subject to the maximum values of the forces that we have found from our dynamic analysis.

Q. And this is for the Tennessee Valley site, correct?

DR. SOLER: Yes, this was for TVA.

MS. NAKAHARA: Your Honor, I would like to renew the State's objection to strike answers 60-61 and the first sentence of 64 as unreliable. The document that PFS provided us last Thursday is a partial -- if you recall our argument last Thursday, our argument that the document provides only selected portions of a larger document and based on the witness' knowledge of the actual soil properties at the site it's impossible for the State to even probe its relevance in the pad flexibility argument in this case.

JUDGE FARRAR: Mr. Gaukler?

MR. GAUKLER: We believe that the excerpts of the document we provided the State are sufficient. They provide the results of the analysis insofar as flexibility is concerned, which was the focus of the testimony. It also provided the methodology by which flexibility of the pads were incorporated into the Tennessee Valley Authority analysis. So we believe there's sufficient information for them to cross the witnesses on.

I have tried to get ahold of the TVA counsel, but he's been out of the office too and we've traded telephone calls over the past couple of days and I have not been able to further resolve it. But we think there's sufficient information here for the State to cross-examine on the method of flexibility that was used to incorporate the flexibility of the pads. And she's cross-examined in terms of differences in the site and things of that sort really with no limitation to focus on differences between TVA and PFS.

JUDGE FARRAR: Does the Staff have a position on this?

MR. TURK: Just I would note that we

think that the cross-examination is adequate to test the witness' ability to rely on the TVA analysis.

JUDGE FARRAR: You say you think it is an adequate?

MR. TURK: It is adequate.

(The Board conferred off the record.)

JUDGE FARRAR: We will deny the motion to strike. Again going back to the first document this Board wrote in the case back in December, we disfavor motions to strike from any party, particularly in a case this complex, because all this evidence is woven together. So we will adhere to that general philosophy in this specific instance. We think that there has been a sufficient basis to cross-examine and that the success of that cross-examination goes to the weight rather than whether this evidence remains in the record. So on that basis we'll deny the motion.

Q. (By Ms. Nakahara) Dr. Soler, if you'll look at your response to Question 78.

DR. SOLER: Got it.

Q. You essentially conclude that in variation the coefficient of friction between the bottom of the cask and the storage pad to the local deformation is negligible?

DR. SOLER: Coefficients of friction are really defined by the two materials that are sliding and whether or not the bodies are moving. Effects due to local deformations due to the load applied by one of the bodies on the other are generally considered second order. So our use of bounding values of either .2 or .8 in modeling the cask to the pad interaction were there to emphasize either sliding, the potential for sliding of the casks relative to the pad or the potential for tipping of the casks relative to the pad.

As I indicated yesterday in the couple of the Beyond Design Basis analyses, we actually treated what I consider as a real case where the coefficient of friction is likely to vary in a random manner at every instant of time due to local nature of the surface because there's more cement or more concrete or the reinforcement may be nearer to the surface or there may be a slight irregularity in the surface. We used random coefficients of friction to simulate that on every cask and really found no significant difference. So we feel that our bounding use of .2 and .8 in all of our analyses was sufficient to capture the potential for sliding and the potential for tipping.

Q. In your random case, where did you apply the coefficient of friction loads?

DR. SOLER: The coefficient of friction is, of course, an input to the program and you either input it as .8 or .2 or any number in between, if that is your desire. There is also a function built into the VN code which allows you to generate a random number between 0 and 1.

Q. Your random situation, the coefficient of friction was constant across the base of the pad for that particular value, correct?

DR. SOLER: No. In those simulations --

Q. Go ahead and answer.

DR. SOLER: In the simulations that we showed yesterday, as you will notice from the movies, every cask at every instant of time, once the excitation became strong, when these casks began to tip or move around on the pad as they were inclined, in general, every cask was moving in its own particular manner and the random cases that we ran, at every instant of time, at every point that happened to be in contact on each of the 8 casks, that number was randomly changing between .2 and 1.

Q. Let me clarify. In that situation for a single cask that has more than one point in contact with the surface, would those coefficient of friction values be the same or would they be different for a particular point in time?

DR. SOLER: You mean those let's say two points on a cask?

Q. Yes.

DR. SOLER: I am not competent enough to know the interworkings of that code to answer you either yes or no on that point. From the limited investigation that I did, where it was somewhat difficult for me to determine how many points were in contact at any given instant, I can only surmise that the one thing I was able to determine was that the coefficient of friction was changing with time. But I could not do it point to point.

Q. So for a particular point in time for one cask with more than one contact point, you do not know whether the coefficient of frictions were varied, correct? That's correct?

DR. SOLER: Across the two points I do not know. I have an opinion, but I do not know. Would you like my opinion?

Q. No. Thank you.

DR. SINGH: Can I provide you additional information on the surface condition of the pad?

MS. NAKAHARA: If we're going to get you on the flight --

DR. SINGH: I would not blame you for half a minute.

MS. NAKAHARA: Go ahead.

DR. SINGH: Contrary to -- unless you work with the equipment, see the equipment, it may not be clear to everyone, I want to clarify this. The bottom of the cask, the bottom surface is crowned at the ends. In other words, it's not a sharp edge. You know, as you see in the diagram you see a sharp edge against the surface of the pad. In reality, the cask is crowned. It's the edge has been removed. And that is the way we make all equipment that is freestanding.

So in case there's a surface imperfection, in case there's a localized settlement there will not be the digging. You know, the hardware would not dig into the surface. In the spent fuel pools, the fuel racks sit on liners, lines of thin stainless steel plates. They are by their very construction not deflected, they have some bow, some ripples in them. Just in the act of installing them there are imperfection. And because of that, all hardware that we design and produce, it's always designed to be tolerant of local variations on surfaces. If that allays your concern about settlement, then these 30 seconds were worth it.

Q. Dr. Soler, if you'll turn to response Question 73.

JUDGE FARRAR: Is this a new subject?

MS. NAKAHARA: Yes.

JUDGE FARRAR: Let me ask a question then. If you would look at your Answer 92 on Page 52.

DR. SOLER: Okay.

JUDGE FARRAR: On coefficients of friction, if I remember learning long ago, 0 is purely theoretical. You can imagine glare ice but there is no 0 in the real world. Is 1 also purely theoretical?

DR. SOLER: We used the value of 1 simply because Dr. Ostadan suggested that that would be the approach.

JUDGE FARRAR: No, that wasn't my question. This is a conceptual question.

DR. SINGH: 1 means complete binding, right, which is theoretically possible. If you take two materials like austenitic stainless steel I said yesterday and you apply pressure, large pressure, you can get 1.

JUDGE FARRAR: You can get 1.

DR. SINGH: But we don't have stainless on stainless here.

JUDGE FARRAR: No, that was just a -- so you can get 1. All right. Then I have two questions. Here you say that .8 is, for your purposes, essentially the same as 1, but you just said in the previous discussion that when you did the random analysis you went as high as 1. Help me on that. Why conceptually is .8 the same as 1?

DR. SOLER: Well, we were attempting, as I said yesterday, to provide some overall analyses that perhaps would bound every contention that was brought up at various times. So since the number 1 had been mentioned as perhaps a more suitable upper bound, we used 1.

JUDGE FARRAR: In your random --

DR. SOLER: In those random analyses.

JUDGE FARRAR: Why in Answer 92 do you say that .8 is essentially the same as 1? Help me with that conceptually.

DR. SINGH: Let me answer. If you assume a friction coefficient, be it .8, .7, .6, what have you, and if you perform your analysis and the cask, the bottom of the cask doesn't slide, then if you use a larger value it isn't going to matter because essentially the interface had sufficient friction to keep the cask from sliding.

JUDGE FARRAR: Under the forces you were applying?

DR. SINGH: Exactly.

JUDGE FARRAR: If we got to much bigger forces then .8 and 1 would not be the same?

DR. SINGH: That's correct.

JUDGE FARRAR: Go ahead.

Q. (By Ms. Nakahara) Dr. Soler, in your response to Question 73, you state the effect of soil cement adjacent to each pad on the casks' response was neglected because the effect would likely be negligible. Isn't that correct?

DR. SOLER: That is correct.

Q. Isn't it also true that you have not, in fact, quantified the effects of the soil cement on the pad loadings?

DR. SOLER: I have not quantified it

officially.

Q. In your response to Question 74, isn't it true you state that "this postulated closure of the soil cement pad gap would lead to horizontal impacts not included in the current analysis"?

DR. SOLER: That is correct.

Q. When you refer to the current analysis, are you referring to your cask stability analysis for the 2,000-year earthquake?

DR. SOLER: Any of the analysis, either Beyond Design Basis or the 2,000.

Q. In your response to 75, isn't it true you state that loads resting from the abutment of the pads and the soil cement would continue to be negligible?

DR. SOLER: Could you repeat that?

(Pending question was read back.)

DR. SOLER: That's addressed to me so I'm going to answer it.

DR. SINGH: All right.

Q. It's the second sentence.

DR. SOLER: I'm reading my answer for a second so that I can answer you correctly. The pad, of course, moves back and forth under oscillation -- under the earthquake loadings. The case of a pad sliding in one direction and then hitting, say, a rigid object, if you want to assume for the moment that the adjacent soil cement were rigid, that would provide an impact as well as an oscillating pad which hit the same impact. In our opinion, both of these loads would be insignificant, they would insignificantly affect the response of the system.

Q. That's based on your opinion, correct? Isn't it true you haven't quantified the actual effects?

DR. SOLER: The statement I will make, that there is no analysis on the record here that quantifies that effect at this present time.

Q. In response to Question 95, isn't it true you base your opinion that soil cement between the pads would not affect the casks response in any material manner, in part on Paul Trudeau's estimate of the amount of the pad sliding under the 2,000-year Design Basis earthquake?

MR. TURK: I'm sorry, I could not hear the question. Could I ask that it be repeated?

(Pending question was read back.)

DR. SOLER: At the time this testimony was written, that is correct.

Q. (By Ms. Nakahara) Isn't it true you haven't quantified the effects of that sliding on the casks' response?

DR. SOLER: There is no analysis on the record that I have done. If you're asking me have I ever done an analysis off the record, I will answer that in the affirmative.

Q. For the PFS site?

DR. SOLER: Yes.

JUDGE FARRAR: Let me interject a moment here. Two answers ago, read that back to me, would you please, Lanette, the testimony that said something about at the time of the test, when it was filed.

(Answer was read back.)

JUDGE FARRAR: That raises for everyone in the room why the limitation to when it was written. Has something happened since it was written?

DR. SOLER: I have done some additional analysis on my own which I've been careful to state is not on the record.

MS. NAKAHARA: Has counsel turned that over? Have you turned those documents over?

MR. GAUKLER: We're in the process of

doing that analysis and will on redirect or rebuttal. We will turn it over when it's done.

MS. NAKAHARA: Your Honor, may we have a moment?

(Discussion off the record.)

MS. NAKAHARA: Thank you, your Honor.

JUDGE FARRAR: How much longer do you think, in terms of when we take our break, how much longer do you have?

MS. NAKAHARA: It will be another hour.

JUDGE FARRAR: Why don't we then take a break for 15 minutes, let the court reporters switch. Be back in 15 minutes from now.

(A break was held.)

JUDGE FARRAR: All right. I think everyone's here. Let's resume the State's cross-examination.

Off the record.

(A discussion was held off the record.)

Q. (By Ms. Nakahara) Yesterday you discussed the NRC validating or accepting certain aspects of the DYNAMO code.

DR. SINGH: That's correct.

Q. Isn't it true that the NRC has not validated DYNAMO for specific contact stiffness values?

DR. SINGH: A validation of a program is not done for specific contact stiffness values. Contact stiffness value's an input data, and the analyst has the burden to ensure, in running the program, that he's using the correct, appropriate value.

Q. Dr. Soler, if you'll refresh my memory, yesterday you stated the maximum rotation for -- that you observed for a 10,000-year event at the PFS site. What was that?

DR. SOLER: It was based on the fact that I observed -- well, let me -- there are two reports. Are we talking about the initial report or the beyond design basis report?

Q. Let's talk about both. Let's start with the beyond design basis report.

DR. SOLER: Could I start with the other one?

Q. Okay. That's fine.

DR. SOLER: The initial report considered only one cask, so there was nothing to impede the rotation of the cask if it wanted to rotate away from vertical. In that case, we observed the maximum deflections of that cask were on the order of, I recall -- well, I don't recall the exact number, but the angle corresponding to them was about 10 degrees.

Q. And when you refer to --

DR. SOLER: Now, in the --

Q. Sorry.

DR. SOLER: -- in the cases that I showed yesterday --

Q. Can you stop?

DR. SOLER: Okay.

Q. When you refer to the initial report, what report are you talking about?

DR. SOLER: That was the initial report on the 10,000-year earthquake that is reference 11-1 in the beyond design basis.

Q. Okay. Thank you.

And that's the report that did not account for soil structure interaction?

DR. SOLER: That is correct.

Q. Go ahead.

DR. SOLER: In that report we, from our observations of the results, calculated a maximum angle of rotation to be approximately 10 degrees with the coefficient of friction .8.

In the analyses -- in the beyond design

basis report, there were other casks involved, and therefore, the maximum angle is probably somewhat less than that. But we did not do any specific calculations to determine what that maximum angle is since our original intent of that analyses was to visually observe all of our results.

Q. And you determined a maximum deflection for the 2,000-year?

DR. SOLER: The maximum deflection we certainly have determined for the 2,000-year earthquake, and that is reported in a summary table, in each direction.

Q. Do you agree that under a certain threshold value, the maximum tilting of the cask axis increases rapidly as the zero period acceleration increases?

DR. SOLER: I do not agree with that, rapidly, your word "rapidly." Certainly, given the 2,000-year earthquake and these displacements, if you increase the strength of the earthquake, it is easy to surmise that the maximum displacements will increase. Your characterization of a rapid increase is one which I do not necessarily agree with.

Q. In your opinion should the maximum

rotation allowed for a cask to tilt be set at some number less than the actual angle that would cause tip-over?

DR. SOLER: Since it is commonly accepted that the condition known as cg-over-corner is a threshold value which most people understand from simple analyses is a dividing point between the onset of tip-over and not tip-over for a very simple case, our approach, when we're trying to set factors of safety, if you will, for cask rotation is to use a certain fraction of the angle that would cause -- that would reach the cg-over-corner, which for the HI-STORM cask is roughly 29 degrees.

Q. So is that the fraction or the angle?

DR. SOLER: No, that is -- at 29 degrees -- if you take the HI-STORM cask, tilt it over 29 degrees, the center of mass, the center of gravity of this cask, will -- if you draw a vertical line from that point, it will intersect the ground right at the point of contact of the HI-STORM with the ground. And if you -- in a static situation, if you -- if you imagine the cask positioned at that point, if you push it a little bit further, it will go over, if it doesn't quite reach that point, it would come back when you let it go.

So that is -- a very simplified measure of cask stability might be how far you are away from that particular angle. However, I must caution that that is a very simplified case, and it is quite conceivable that if you ever reach that angle in a dynamic situation, your cask will also be precessing. Whether or not it will tip over you could only tell from the solution.

Q. You mentioned that for a factor of safety you would set the angle at a fraction of the tip-over angle. What fraction would you set it at?

DR. SOLER: We -- we have -- in previous submittals where we characterized freestanding casks, we have generally tried to set, I believe here, that the position of the center of gravity of the cask at any time during the motion -- in other words, if you -- if you imagine for a minute that you track in the horizontal plane the position of the mass center of the cask, during that time that moves in that plane, and if you compare that plane to the diameter of the cask, which, in that case, is about 133, I believe we were setting that to be roughly about one-half the excursion -- if the excursion ever exceeded one-half of that radius, of the initial radius, then we considered that we would -- we would not accept that.

DR. SINGH: Now, realize that this is the calculated tilt. The actual maximum rotation that the cask would see, because of the huge conservatisms built in the dynamic model, would be much less.

So there are two layers of safety here. One is, Dr. Soler said, we set at 50 percent of the cg migration, and then the other built-in factor of safety is that the analysis procedure itself is extremely conservative. It's deliberately set to yield conservative results.

MS. NAKAHARA: I'd like to mark this as State's Exhibit 174.

THE COURT: All right. The reporter will mark this document we're being handed, Seismic Response Characteristics, as State 174 for identification.

(State's Exhibit-174 was marked.)

MS. NAKAHARA: Your Honor, can we take a break? I have the wrong pages in the exhibit.

JUDGE FARRAR: All right. Let's take a break, pretty much in place.

(A recess was taken.)

JUDGE FARRAR: I understand that the exhibit we just marked is missing a page. And so to allow us to proceed, the witnesses have been given the complete document, and the State will substitute the proper excerpt at an appropriate occasion.

MS. NAKAHARA: Yes, Your Honor.

JUDGE FARRAR: Okay.

Q. (By Ms. Nakahara) Dr. Singh and Dr. Soler, is this a publication or a paper you presented called "Seismic Response Characteristics of HI-STAR 100 Cask System on Storage Pads," by you, Dr. Singh, Dr. Soler and Martin G. Smith?

DR. SOLER: That is correct.

Q. If you'll turn to page 15, Dr. Soler, please read the last sentence, partial sentence on page 15 continuing through that paragraph, into the record.

DR. SOLER: It starts with the word "exploratory"?

Q. I think it starts with after.

DR. SOLER: Oh, okay. After a certain threshold value, the response is maximum tilting of the cask axis increases rapidly with increase in the ZPA level.

Do you want me to go on?

Q. Yes, please.

DR. SOLER: For this reason, it is recommended that the acceptable response parameters, e.g., maximum rotation of the cask axis, be set at 1/4 of the ultimate value at which the cask will tip over. In addition to kinematic limits, specific requirements on stress limits on critical cask contents are also proposed.

Q. Thank you.

JUDGE FARRAR: And for the record's clarity, what does ZPA level --

DR. SOLER: Zero period accelerations.

DR. SINGH: Same as PGA.

MS. NAKAHARA: And I won't move for admittance until we get the right document.

Q. Dr. Soler, yesterday you stated in selecting a contact stiffness that it must be -- a contact stiffness value, it must meet the no physical penetration test, correct?

DR. SOLER: That is correct.

Q. Define what no visible penetration means.

DR. SOLER: That is subjective. It depends on the problem. If -- if I stand up and --

on a wooden floor and someone else looks at my feet, they see no visible penetration. If they took a micrometer, a very fine micrometer, I would suspect that they would see some penetration.

Numbers that are, let's say, .0 something, .00 something I would characterize for an analysis of the type that we're doing as a sufficient representation of no visible penetration. Numbers that I could measure with a ruler, say on the order of a quarter of an inch or so, I would consider as visible penetration and not representative of the situation that I would see when I would put one body down on another hard body.

Q. So is it fair to say you would set the limit at a quarter inch?

DR. SOLER: No. Well, let me -- let me get some -- you mean if I said contact stiffness was picked that gave me a quarter of an inch, that's okay?

Q. Yes.

DR. SOLER: No. That's -- that's -- I used that simply as an example here.

In my mind an acceptable limit for, let's say, a finite element analysis or a dynamic analysis would be as small as I could get it without running into problems of having my solution difficult to get convergence.

DR. SINGH: Let me explain the fundamentals here if I may. Can I?

Q. No.

DR. SINGH: The contact stiffness has been asked in the depositions. It's kind of a key parameter in the -- in the evaluations we have performed. Strictly speaking, the contact stiffness is, in this particular problem, you have the cask and you have the pad. I assume that the pad is on a rigid surface, okay?

Now, you take the cask and press down with it, apply a force. The amount by which this cask would penetrate the pad with an applied force, if you measured that force, let's call it D, if you applied a force F, then the stiffness is F over D.

Now, you can intuitively see that if you have a 3-feet-thick pad and you try to press down with the cask, you're going to have a very large value of the contact stiffness. Now, in the numerical calculation work, if you use a large stiffness, then your computation gets very, very slow. You have to use a small time step in the time history solution, and when you use very small time steps, you -- computers don't carry infinite number of digits in their computations -- you begin to have numerical inaccuracy problems.

So what we do, as analysts, we see if we can reduce the value of the contact stiffness without altering the result, okay? Now, that is the same answer that I gave to -- to -- to the judge earlier. If the friction coefficient is high enough, it doesn't matter. Similarly, if you reduce your contact --

DR. SOLER: Contact stiffness.

DR. SINGH: -- your friction coefficient is high enough in friction problems, it doesn't matter if the cask did not slip.

The same -- same core process applies in the selecting the contact stiffness. As long as the stiffness is high enough that it does not corrupt the solution process, then you are okay. If you take your stiffness down so low where now it is under -- not only is unrepresentative of the structure, but it begins to interfere with the dynamic characteristic of the structure, then you end up getting results that don't make any sense. They will be absurd. Some of the low values used by the State fall in that latter category. The contact stiffness values are selected so low that they distort the problem beyond its original context, and you get absurd answers.

Q. What would be the contact stiffness in the upward direction?

DR. SINGH: Contact stiffness in the upward direction would be zero.

Q. Okay. You mentioned that you would get the contact stiffness as low as you could so it wouldn't affect your analysis.

DR. SOLER: High, as high as you could.

Q. No. Actually I think --

DR. SINGH: Your question is correct. The actual contact stiffness is indeed very high. We -- in the numerical solution, we can use the actual calculated contact stiffness and pay the price in terms of calculating for a long, long time numerically and possibly have the -- the negative effect of numerical errors because of a significant -- the number of significant digits the computers carry in the calculations.

The analyst, an experienced analyst such as Dr. Soler -- he's been doing it for 30 years -- he will use a contact stiffness which is high enough that it does not interfere with the natural vibration characteristics of the system, okay? But at the same time -- and, therefore, it does not corrupt the solution process. But he will use a number lower than the actual contact stiffness, which is a theoretical value you can calculate between two -- two bodies in contact.

Q. So if you're trying to find the correct high contact stiffness that is low enough not to corrupt your analysis -- have you run cases with varying contact stiffnesses for the cask stability analysis at the PFS site for the 2,000-year?

DR. SOLER: Yes.

DR. SINGH: Yes, we routinely -- we routinely do that -- he routinely does that to ensure that the value of contact stiffness is appropriate. That's a routine process in --

Q. What contact stiffnesses have you used as inputs?

DR. SOLER: For the 2,000-year return earthquake, we use the theoretical value that we computed, and I believe it was 4.54 times 10 to the 8 pounds per inch total for the cask in the vertical direction.

Q. And what other contact stiffnesses have you used?

DR. SOLER: In the analysis with the -- well, of course, we -- we got acceptable answers in the 2,000-year return earthquake, so there was no incentive for us there to lower the contact stiffness. But in the VisualNastran runs that we have done, we have lowered the stiffness to decrease the simulation times because of the vast amounts of data that we were collecting, and we have done a series of test runs where we established that the number we ended up with, which was roughly 4 -- 40 million pounds per inch, a factor of 10 less than in the 2000-year earthquake, does not significantly alter our answers.

So we're -- we're well in the range, and if you take that -- either of those values satisfy the test of physical reality in that if you calculate the -- if you take the dead load of the cask, which is 360,000 pounds, and divide by those stiffnesses, you get a very small number which is essentially the static deflection of the pad under the cask if you just put the cask down on the pad. And I do not -- in my opinion, anyway, I would suggest that most of the people in this room would consider that that pad deflecting 3/8 of an inch would not be physically meaningful for this kind of a problem.

Q. To clarify the record, you said that VisualNastran runs use a contact stiffness of 40 million pounds per inch?

DR. SOLER: I believe that that was exactly it. I will check the number if you want an exact number.

Q. No. I just want to know which ones you're referring to. Are you referring to the VisualNastran runs in the beyond design basis document?

DR. SOLER: Either the beyond design basis or the first report, the 10,000-year.

Q. 10,000-year. What was the contact stiffness you used in your very first cask stability analysis in '97?

DR. SOLER: I believe was the 4.54 times 10 to the 8 pounds per inch.

Q. If you'll refer to your response to 144, if you look at the second half on page 82, when you mentioned calculating the deflection for the dead load of the cask, is it correct for the 4.54 times 10 to the 8 pound per inch context that you obtained 0.00008 inches?

DR. SOLER: Well, that's what is written. If you want me to do the calculation, I better do it before I say yes or no.

Q. That's not the important point. That's okay.

DR. SOLER: Excuse me?

Q. That's not the important point. The --

DR. SOLER: The deflection associated with that stiffness would be 360,000 divided by 454,000,000.

Q. Okay. And in your opinion, .00008 inches meets the no visible penetration test, correct?

DR. SOLER: Assuming that that number -- that I correctly did the division, yes, that would be a number that meets my test.

Q. And you also calculated the deformation for a 40 million pounds per inch context, correct?

DR. SOLER: Yes, that the value .009 I would also consider as an acceptable value.

Q. What about for 0.01 inch?

DR. SOLER: 0.01 inch? I would probably reserve judgment until I actually ran the calculation, but it may be acceptable. It may be acceptable. I mean it seems -- you know, there you're starting -- 001 is pretty small, 01 is also pretty small. I would believe I'd be reasonably willing to accept as a good value as a static deformation any number with a .0 in front of it for these kind of problems.

DR. SINGH: Realize, as he says, we say in the response that it's calculated -- 40 million is calculated based on 33 hertz, which -- which is a way to assure that the solution will not be corrupted by the natural characteristics of the system. Earthquakes don't have frequency context over 33 hertz. So if you -- if you set the contact stiffness in the range which is outside of the potential range where the frequency characteristics of the system may interfere or the input may interfere, then you will get -- then you'll get correct results. If your stiffness is too low, and, therefore, the frequency you will calculate corresponding to that stiffness is too low, then you will end up corrupting your solution.

Q. Thank you, Dr. Singh, for reminding me I had a follow-up question.

Dr. Soler --

DR. SINGH: All right.

Q. -- you mentioned that you ran some test runs varying the contact stiffness. What contact stiffness did you --

DR. SOLER: In --

Q. -- perform those runs?

DR. SOLER: In the VisualNastran models I have -- I have run cases with 10 times the stiffness that I've used here, and I have also run some cases with about 1/8 of the stiffness that I've used, the 40 million.

Q. 10 times, then, you would have run approximately a 400 million pounds per inch contact stiffness?

DR. SOLER: I believe that I -- I ran a run with something of that order.

Q. And is that with the PFS 10,000-year design basis -- or 10,000-year return earthquake?

DR. SOLER: Yes.

Q. And the 1/8 stiffness, is that 1/8 of the 40 million?

DR. SOLER: Roughly speaking, yes.

Q. Thank you.

And that's also for the 10,000-year return --

DR. SOLER: No. Let me correct that. That particular one was the 2,000-year.

Q. Dr. Singh mentioned that the natural frequency -- strike that.

Dr. Singh mentioned a 33 hertz frequency. What is the cask rocking frequency?

DR. SOLER: Would you define -- well, what do you mean by the cask rocking frequency, that if I tilt it over and let it go, that --

Q. It goes back and forth.

DR. SOLER: That we have not calculated by simple techniques. I've not calculated that frequency.

Q. What contact stiffness would you expect if the cask rocking frequency is between 2 to 5 hertz?

DR. SOLER: I would -- I would not choose a contact stiffness on the basis of the global motion of the cask. The contact stiffness is a -- is an item, if you will, that is imposed locally to describe the potential for compression only contact between two surfaces.

If you look through a microscope, if you will, at those two surfaces, at that point of contact, that stiffness, if you will -- I want to ascribe some intelligence to that stiffness -- that stiffness doesn't know that the cask is actually rotating down. When it hits -- when it hits the surface, these two local points in contact are coming straight at each other. You -- you can't ascribe the fact that the -- you have a rotational motion globally of that cask with choosing a contact stiffness.

If you choose a correct -- and correct in parentheses here meaning no one number is necessarily correct. If you choose a contact stiffness, it should not be problem dependent. You should be able to analyze equally well, calculate the dead load static deflection of the cask on a rigid pad. You should be able to predict what happens if you drop the cask on the pad and watch it bounce back up in the air and continue to bounce until it comes to rest. And you should also be able to correctly predict what happens if you tilt the cask and let it bounce back and forth in, say, a simple two-dimensional problem.

The same contact stiffness ought to be applicable, because if you go beyond that, then you are starting to get in the realm of saying that the same pad and the same cask which is located in Salt Lake City should have one contact stiffness in its model and the same cask and the same pad that's located, say, in California should somehow have a different contact stiffness associated with that phenomena. And that is something you just should not be in the realm of doing.

That is -- the contact stiffness should not be a function of the input motion.

Q. You mentioned you ran a test run with 1/8 contact stiffness -- 1/8 of the 40 million pound contact stiffness for 2,000-year. Did you include the effects from the soil springs and dampers in that test run?

DR. SOLER: Yes, we did.

Q. What were your results?

DR. SOLER: There was some changes. If you -- if you compare the results with the same results for the 40 million, if you want to consider that as what we consider as our base case, there were some differences in the results. As I recall, the maximum deflection of Cask No. 1, which I was tracking, was slightly larger than it was for the case with the 40 million stiffness.

Q. When you say "slightly larger," can you approximate that?

DR. SOLER: I would say maybe a half an inch or so, maybe -- maybe as much as an inch. I can't be sure without, you know, looking.

Q. Did you model 1 cask on a pad or 8 casks?

DR. SOLER: No. The particular case we ran was 8 casks on the pad.

Q. Is the cask rocking amplitude a function of input motion?

DR. SOLER: I would say the cask rocking amplitude is a function of input motion, yes.

Q. In your testimony you talk about ANSYS gives guidance. Isn't it true that ANSYS has over 500 examples for solving problems?

DR. SOLER: Did you get that?

THE REPORTER: Would you read it to me?

(The question was read.)

DR. SOLER: They call them verification problems. I believe 500 is probably a good number.

Q. (By Ms. Nakahara) Do you know if there is any example in ANSYS 5.7 to solve the problem of pure sliding and uplift on objects under an earthquake condition?

DR. SOLER: No. I mean I know for a fact that there is not a -- either a real or artificial earthquake applied to either pure sliding, pure uplift or any combination of same in the ANSYS verification manual.

Q. Do you know how many example problems are in the verification manual for VisualNastran in selecting contact stiffness?

DR. SOLER: In VisualNastran that use contact stiffnesses -- there is an option in VisualNastran to either deal with coefficient of restitution or model the stiffness by a spring, as we have done. Your choice of which option to use depends upon what information you're looking for. As a result the VisualNastran verification manual provides 6 problems that they have documented against other sources in the literature. I believe -- and here I'm strictly going by memory -- that two of them definitely involve some kind of contact.

To those problems in our -- we, of course, in our validation manual repeated all of their sample problems to ensure that our computers were giving the same solutions that they indicated agreed with the exact solutions. We also added a problem that was directly attributable to studying whether or not VisualNastran could and correctly predict contact behavior.

That problem was a classical problem,

which you can find in, I suspect, high school physics books, of the dropping of a sphere onto a hard surface where you correlate the initial height to the final height and, in fact, to the various heights as this object bounces. It is usually done in terms of illustrating the concept of what is called coefficient of restitution.

We duplicated the expected behavior there, and we went one step further in our validation by noting that there is a correspondence between coefficient of restitution and the amount of damping that you would put in parallel with a spring, and we ran the same problem as part of our validation manual using the contact stiffness approach as opposed to the coefficient of restitution approach to model collisions. And we got the agreement that we expected.

Q. The dropping sphere problem, that's not for pure sliding and uplift, correct?

DR. SOLER: That is not to pure -- well, uplift has nothing to do with it, but it is certainly not for pure sliding. I mean --

Q. Is there --

DR. SOLER: I mean contact stiffness disappears as soon as you have separation.

Q. Is there any example in VisualNastran for an earthquake problem for pure sliding?

DR. SOLER: No.

Q. Have any of your contact stiffness results been validated compared to natural test data such as summary table data?

DR. SOLER: No.

Q. In your opinion is it possible to solve pure horizontal sliding problems without a horizontal stiffness?

DR. SOLER: Well, there are a number of simple problems where you can, because of the simplicity of the analysis, get away with assuming an infinite horizontal stiffness in your simulation. As a practical matter, as the ANSYS manual points out, if you try to do that in a reasonably complex problem, you're just asking for numerical problems. And they suggest that you should, you know, incorporate a reasonable horizontal stiffness kind of using the same guidelines that you would go about choosing a vertical stiffness for contact.

Q. For the simple problem -- have you tried to run a problem through your model without a horizontal stiffness?

DR. SOLER: Without a horizontal stiffness?

DR. SINGH: You have to have a horizontal stiffness. That's --

DR. SOLER: Numerically the problem gets very difficult to -- you can't really treat a problem numerically where, at the same deflection -- you can either -- you can have two forces, either -- either zero or a limiting force. You have to -- you have to simulate any kind of a stiffness with a finite value.

Now, in the classical friction problem, that finite value where you either have sticking or slipping looks to you like it's occurring instantaneously when you're observing it, but in reality there are small imperceptible deformations that occur prior to the actual slip. The -- how detailed you want to model those is a function of how much risk you want to take in convergence. You try to simulate an infinite stiffness horizontally with a finite stiffness. But, again, it requires judgment as to whether the stiffness you choose is appropriate.

Have I answered your question?

Q. I'm not sure, to be honest.

DR. SOLER: I could always let Dr. Singh elaborate.

DR. SINGH: I was hoping you would ask me.

Q. Dr. Soler, can you write an equation of motion of pure sliding without horizontal stiffness?

DR. SOLER: You can certainly write an equation of motion in a piece-wise linear manner for an idealized, say, problem of a block that is sliding either due to some force applied to it, or if you have a spring attached to it and then you stretch the spring and then let it go, I believe those two problems have been solved analytically. And with enough effort, you can solve them without regards to a horizontal stiffness. There may be others, but these are, again, very simplified problems.

Q. I just have -- I have a few more, if you'll look at PFS's Exhibit 86, the beyond design basis --

DR. SOLER: Okay.

Q. -- document. If you'll turn to page 15, for the record, is it correct that there's a schematic on this page?

DR. SOLER: Yes. There is a simplified figure on that page.

Q. At the attachment point of the spring -- I don't have the rest of my question. Strike that. Sorry.

If you'll look at page 17, it's Table 2, Summary of VisualNastran Analysis, correct?

DR. SOLER: Correct.

Q. For Case 8 and 11 -- Cases 8 and 11, have you evaluated what would happen if you choose as an oscillation frequency time from 3 to 7 hertz?

DR. SOLER: Well, in case -- now, we tuned Case 8, but tuning referred only to the soil springs. We did not tune Case 11 because that was using the values that were given to -- soil values that came to us from Geomatrix. Neither of those runs had different contact stiffnesses between cask and pad. That's stated -- both of those runs had the same contact stiffness.

Q. Is it possible to run Case 8 and 11 and tune the pad oscillation frequency from 2 -- from 3 to 7 hertz -- for a range of 3 to 7 hertz?

DR. SOLER: Let me see if I understand your question.

MR. GAUKLER: Can we have the question

reread first?

(The question was read.)

Q. (By Ms. Nakahara) Actually I read that wrong. Let me strike that.

To change the contact stiffness to tune the frequency from 3 to 7 hertz?

DR. SOLER: I can certainly change the contact stiffness. However, I would not buy into the fact that the value I change it to is in any way accurate.

Now, beyond that, the question needs a little elaboration because there are a number of stiffnesses involved in the simulation. There's, first of all, the stiffness between the cask and the pad, which is the contact stiffness we're talking about, and there is certainly the vertical component of the stiffness of the soil.

Now, if you think of this complex problem as a very simple problem involving a cask mass, a pad mass, a spring between the cask and the pad and then another spring between the pad and the rock, or whatever, you have a classical 2-degree-of-freedom system. We have two masses and two springs. Our contention is that one of those springs is very stiff, the contact stiffness, and the spring that we tuned to get what we call our tuned case was to deal with the -- I guess the -- some uncertainties that were raised by the State about whether or not our values of soil springs were accurate or inaccurate or omitted certain features. And, therefore, in attempt to, if you will, wash away those concerns, we chose the worst case soil stiffness that we felt was appropriate. If we, if you will, just simply chose contact stiffnesses between cask and pad to be tuned to a certain input, then we'd -- in our opinion, we'd be buying into a game that's not technically justifiable.

Q. How do you ensure that the 5 hertz you selected is, in fact, the worst case scenario?

DR. SOLER: Well, in deposition testimony, 5 hertz was raised by the State's expert witness as being a frequency at which there was predominant earthquake energy being input into the motion and pointed out that he saw some observable deflections of the pad from the CEC calculations. Therefore, I was willing to buy into an analysis in which I said regardless of the physical reality of the site, let me just assume that, unfortunately, I have soil stiffness that is going to give me what

I'll call simply a resonance condition in terms of a simple linear system.

MR. GAUKLER: And just for clarification of the record, you're talking about resonance system of the soil springs in that case, right?

DR. SOLER: Well, what I'm really talking about is imaging the cask and the pads as one body of a total mass and the soil spring vertically as one spring. So I have a mass and I have a K and an M. And the relation to the lowest natural frequency of a linear system consisting of a mass, M, and a spring constant, K, the lowest natural frequency in cycles per second is 1 over 2 pi times the square root of K over M. So, therefore, with a given mass which I certainly had and a frequency which I wanted to tune to, I could simply pick the soil stiffness.

Q. Did you tune your frequency in the vertical direction?

DR. SOLER: I not only chose it in the vertical direction but chose it the same in the horizontal direction as well. So all three linear directions had soil springs that were tuned to 5 hertz natural frequency based on 2 limits, either the pad and 8 casks as the mass -- yes, the pad and

8 casks as the mass or the pad and 1 cask as the mass. I have 2 sets of tuned stiffnesses, if you will.

But is there more to your question or have I --

Q. If you'll turn to Appendix A, page A-1 --

DR. SOLER: That's of the beyond design basis?

Q. Yes.

DR. SOLER: Okay.

Q. Is it correct under the column input that the cutoff frequency is 33 hertz?

DR. SOLER: That is the frequency we chose to develop the spring constant K0 in that report, which represented the contact stiffness between the pad and the cask at one of the 34 points.

Q. What is the basis for 33 hertz?

DR. SOLER: As Dr. Singh pointed out a little bit ago, earthquakes -- bodies under forces caused by earthquakes are generally considered as rigid bodies when their frequency -- natural frequency exceeds 33 hertz. That's because the major -- most earthquakes will have major energy content in the low frequency range. And if -- if you look at response spectrums, you can see that, generally speaking, above 10 to 15 hertz in the response spectrum at most sites, you will find that the response spectrum falls off and begins to approach the 0 period accelerations. And at 33 hertz your response spectrums are, in general, flatlined at whatever either the ZPA or the PGA is designated for that site.

So what we're basically doing by this method of choosing contact stiffnesses is to make sure that our contact stiffness doesn't give rise to another frequency that is below the 33 hertz cutoff point we ascribe to.

You can also relate that same frequency, be it 33 or any other number, to the static deflection of the object. In other words, there's a direct 2- or 3-line simple calculation which will get from you a statement, if this object, mass and a single spring, has a certain natural frequency, what is its static deflection when I put it down on a surface? There's a very simple relationship that you can derive which will tell you -- if you tell me the natural frequency, I'll tell you the static deflection and, conversely, the other way, if you want to choose an observable or an appropriate static deflection that corresponds to a certain natural frequency of this simple system.

Q. Thank you.

On page A-3 of the appendix where you perform a calculation to check on resonance, can you explain why you selected a soil damping value of 1 percent?

DR. SOLER: Again, we -- there had been some contentions by the State that the soil damping values that we were using corresponding to best estimate, lower bound or upper bound and calculated in accordance with the soil modulus -- moduli that we received and the formulas in ASCE 86 somehow included -- we were including more damping than by using those formulas than we might have in our system because of other effects that we had neglected. And, therefore, since this entire report beyond the design basis was an attempt to answer let's just take the worst case we can reasonably imagine, we also decided to lower the damping value to, hopefully, a value that would be uncontestable.

Q. In calculating the constitution -- the coefficient -- sorry, the coefficient of restitution on page A-3, you account for 1-percent damping --

DR. SOLER: Well -- well, first of all, the coefficient of restitution is not calculated on page A-3. I think you want A-2 for that.

Q. What's the calculation of -- under Check on Resonance, where C is equal to 2 times 0.01 times --

DR. SOLER: Okay. That equation, yes. I mean that's a calculation of a damping coefficient, not a coefficient of restitution.

Q. Oh, okay. I apologize.

Is the 0.01 factor in that calculation, is that the damping -- does that account for the --

DR. SOLER: That's -- that's the 1 percent, yes.

Q. So that when you look at --

MR. TURK: I'm sorry, Your Honor. I'm having a hard time understanding the question because it's being asked in such a quiet manner. Can I hear the last question back, question and answer please?

(The record was read as follows:

"Question: Oh, okay. I apologize.

"Is the 0.01 factor in that calculation, is that the damping -- does that account for the --

"Answer: That's -- that's the 1 percent, yes.")

Q. (By Ms. Nakahara) So if you look at your damping calculation on page A-2, is it correct you used a 40-percent damping value?

DR. SOLER: For our calculations involving VisualNastran for the design basis -- at the contact stiffness, we are using 40 percent of critical damping for the damping that represents the loss of energy at the pad/cask interface. The 1-percent damping refers to the soil damping.

JUDGE FARRAR: Ms. Nakahara, I think we're all right when you're whispering right into the microphone, but when you get even a few inches away from it, the sound system doesn't pick it up.

MS. NAKAHARA: Actually, if I could have one moment, I think I'm done.

(A discussion was held off the record.)

MS. NAKAHARA: That concludes my cross-examination. Thank you to everyone for putting up with me.

JUDGE FARRAR: Well, we admire your fortitude and tenacity at pressing forward on a difficult subject under difficult vocal cord conditions.

The Board has just a couple of questions. Let me start on page 66 of your testimony.

DR. SOLER: Okay.

JUDGE FARRAR: The summary chart appears to be nearly the same as Table 2 on page 17 of what's now PFS Exhibit 86; is that correct?

DR. SOLER: Except for the fact that -- I mean we -- we did make some corrections, but I do not see them appearing here in that table. Those are the ones I read out yesterday.

JUDGE FARRAR: Right. As I look at the corrections, it looks like the corrections in your testimony were largely intended to bring the testimony into conformance with the table.

DR. SOLER: That is correct.

JUDGE FARRAR: There are -- but in the remarks, some of the corrections bring it into conformity, but the remarks in Table 2 of the exhibit are not exactly the same as the remarks on page 66 of your testimony, although the differences may be only matters of style or phraseology as opposed to substance. Can you compare those and --

DR. SOLER: Let's see. I -- all right. I'm going to look at the corrected table.

JUDGE FARRAR: And just focus on the remarks.

DR. SOLER: On the remarks, the -- for Case 5, the testimony as I'm given says "Check sliding," and that is simply incorrect.

JUDGE FARRAR: No, no. That was changed to say "Check real configuration."

(A discussion was held off the record.)

JUDGE FARRAR: If the witness will hold his remarks or make sure, if you're talking to counsel, the reporter doesn't have to hear it.

5 says "Check real configuration," and that was --

DR. SOLER: That was --

JUDGE FARRAR: -- changed.

But on Case 1, for example, the table lists stiffness values per Appendix D which is omitted in the remarks.

DR. SOLER: Yes. The -- the report actually gives the appendix because our QA program requires that we give our reviewer enough information that tempts them to go there instead of just accepting the result.

JUDGE FARRAR: And on 6 and 7 where you -- in the remarks of your testimony it now says "low stiffness" and "high stiffness," that's the same as the lower bound tune stiffness and upper bound tune stiffness?

DR. SOLER: Yes, in the report I used the word "lower bound," but I didn't mean lower bound soil stiffness. So I wanted the table in the testimony to not use the words "lower bound" or "upper bound."

JUDGE FARRAR: Okay. The corrections you made in the stiffness column yesterday, they now -- the columns in the testimony and the columns in the report coincide, but explain to me again why in Case -- Cases 5, 6 and 7 the number of casks that you're starting with is not the same as the number of casks in your stiffness calculation.

DR. SOLER: The number of runs we were planning to do, we started out with one cask on the pad, and, therefore, I calculated what I'll call the low tuned stiffness on the basis of 1 cask plus the pad and a stiffness then chosen to give us 5 hertz.

At the other extreme of 8 casks on the pad, the mass was different, so the stiffness came out to be different but still tuned at 5 hertz. The --

JUDGE FARRAR: Wait, wait. Maybe --

DR. SOLER: The --

JUDGE FARRAR: Wait, wait.

DR. SOLER: Okay.

JUDGE FARRAR: Maybe I didn't make my question clear. Case 5 --

DR. SOLER: I was just getting there by using the two extremes.

JUDGE FARRAR: Okay.

DR. SOLER: For the intermediate cases, I balanced the time that I had available with the necessity for calculating a new tuned stiffness based on either 3 casks -- 2 casks, 3 casks or 4 casks, and I decided, since I really didn't see that much difference between the 1-cask case and the 8-cask case, that I was just going to arbitrarily decide that if I ever went beyond 3 casks, I'd use the high stiffness and, if I was doing any case below 3 casks, I'd use the low stiffness. I did not on a case-by-case basis tune the stiffness, which is one reason why I changed the words to low and high.

JUDGE FARRAR: But I want to make sure I

understand. So in Case 5 where you have 3 casks on the pad and you use a stiffness based on the mass -- mass of 1 cask, that's done for the reason you just said.

THE WITNESS: That is correct, yes.

JUDGE FARRAR: So the same when you have 4 casks, but you base it on a mass of 8.

DR. SOLER: Correct.

JUDGE FARRAR: That's all I have.

Dr. Kline?

JUDGE KLINE: Yeah. I want to refer you to Question and Answer 42 in your testimony.

DR. SOLER: Okay.

JUDGE KLINE: Okay. It deals with the confirmation -- confirmatory analyses done by Sandia. What impact on you did the Sandia report have? In other words, did you have to do anything or go back and recalculate anything or restructure your analysis in any way after seeing the Sandia report?

DR. SOLER: No. Our -- our analyses -- at the time we did our analyses, we did not have a copy of the Sandia report. We noted, however, that the summary they provided or that was provided to us did seem to give the same -- at the time it was provided to us, provided numbers that were, given the differences in the model, in agreement with ours to some extent.

JUDGE KLINE: Have you since seen the full report?

DR. SOLER: I have read the report but not in the detail I would read it as if I was a reviewer.

JUDGE KLINE: Okay. In your opinion does it constitute a genuine confirmation of what you did? That is to say, was it -- was it something that would give us confidence or should give us confidence in the analyses you performed?

DR. SOLER: Well, it is -- certainly includes some features that we did not include, like the flexibility of the pad, explicit differentiation between soil cement and underlying soil, the potential for sliding of the pad and the soil cement and, similarly, the potential for soil/soil cement sliding. It also modeled the soil/soil cement by a finite element representation, and it used a computer program that, in my opinion, would be capable of predicting large rotations.

In the 2,000-year earthquake, it

certainly seemed to confirm completely the results. In my observations of the 10,000-year, it appears that the results that we are getting, if you would like to consider Sandia's results as a base case, are overpredicting deflections, although you could equally well use the other one as a base case.

JUDGE KLINE: Well --

DR. SOLER: Both of them -- I'd like to point out, regardless of the differences in solution methodology, the difference in models, the difference in number of casks, they both predict that the cask will not tip over.

DR. SINGH: The Sandia -- the Sandia results, the report, indeed provides a definitive confirmation to the conclusions we have reached --

JUDGE KLINE: All right. I note that they considered only a single cask. Would you still consider it a global confirmation of what you did, or is it a limited confirmation?

DR. SINGH: No. Actually, it's a -- in my personal view, it provides a global confirmation. The evaluation of casks on pads and stability calculations historically have been done using a single cask. That's been the standard practice. In the case of PFS, to go the extra mile, to allay concerns of people who -- who might entertain thoughts that the cask might tip over, we decided to make a multicask model. But the conclusions, as we said earlier during the testimony, they don't change if you consider multiple casks or one cask.

JUDGE KLINE: In your view, did the Sandia analysis -- was it equally applicable to the VisualNastran simulation as well as the DYNAMO simulation?

DR. SINGH: Yes. They both -- both solutions -- both solutions, whether we use DYNAMO, we use VisualNastran or we used Abacus that Sandia used, they all fundamentally rely upon the same principle, Newton's equations of motion. It's just the details of the modeling, the details of the way the program you organized differs. So, yes, the answer is yes.

JUDGE KLINE: Okay. Just a general question on confirming one model with another model. What does it do to our level of confidence in one model when it's confirmed by another model? I mean could we not view -- view it just as well as that you confirmed Sandia's model?

DR. SINGH: Well, that would be -- that

would be honoring a private company against the National Laboratory --

JUDGE KLINE: All right. What is the standard that says Sandia confirmed you?

DR. SINGH: Well, we view this as -- as a peer effort by another organization --

JUDGE KLINE: Yeah, okay.

DR. SINGH: -- fully competent to perform -- fully qualified to perform such evaluations. And the fact that they performed these analyses -- we weren't even aware that the work was being done by Sandia, let alone have access to their results. That they independently came to the same conclusion is certainly -- we view this as an excellent confirmation of the work we have done.

JUDGE KLINE: Okay. Thank you.

JUDGE FARRAR: Mr. Turk, in light of Mr. Soper's cross-examination, would you be good enough to alert me when you put on a witness who can deal with this subject from the Staff point of view that Dr. Kline has just been mentioning?

MR. TURK: Yes, Your Honor. That would be the testimony of Dr. Luk and Mr. Guttman.

JUDGE FARRAR: Okay.

MR. TURK: As I understand it, we're currently expecting that Dr. Luk will be deposed over the weekend, and we hope that his testimony will be presented this coming Monday.

JUDGE FARRAR: Okay.

MR. TURK: And I will do some follow-up questioning in light of the Board's questions with these witnesses just to bring out some highlights.

JUDGE FARRAR: But on this question we can get only so far with these witnesses, and we'll want to talk about the Staff --

MR. TURK: Absolutely.

JUDGE FARRAR: -- review process.

MR. TURK: Yes. And Dr. Luk and Mr. Guttman are here in the room, and I'm sure they're listening with great interest to the question.

JUDGE FARRAR: And, Mr. Soper, in light of Dr. Kline's questions, do you want to do any further examination of these witnesses, either right now or before we excuse them?

MR. SOPER: I'd just ask one question.

RECROSS-EXAMINATION

BY MR. SOPER:

Q. It seems to me that the Sandia -- Sandia study was quite different than the Holtec study with respect to the 10,000-year event in that my recollection is Holtec predicted a rotation, a maximum rotation, of something in excess of 10 degrees; is that correct?

DR. SOLER: The initial report predicted a -- I believe that's correct. The report that you have seen the results of in the movie, I haven't calculated explicitly the rotations, but they obviously, as you could see, would be large.

Q. But doesn't your answer say 10 degrees? What do you mean you haven't calculated it?

DR. SOLER: My answer for the design basis -- for the original Reference 11-1, I made a calculation. The Beyond Design Basis Scoping Analysis, I showed you movies. And I do not believe in that report -- and I'll just look in a second -- that I've quoted any 10-degree value in that report.

There is no 10-degree value quoted in the Beyond Design Basis Report.

Q. What about your testimony?

DR. SOLER: My testimony, I quoted the 10-degree value, I believe, and, again, I was talking while the movie was playing. I am willing to estimate that it is highly likely that in those movies the angular displacement -- the angular rotation of one or more casks was probably in that order, but I have not explicitly calculated that. I believe that at least for Cask No. 1 that is a task which I have --

Q. Well, let me read your testimony for Answer 39. Although the loaded cask exhibited larger rotations relative to the pad (approximately 10.89 degrees from the vertical) than seen in the earlier analysis using the lower earthquake levels, the results of this analysis still showed the existence of significant margins against tip-over.

DR. SOLER: At the time that testimony was written, that was referring to -- and I've lost track of the number, so I'll simply read the reference. That was referring to the results in the report that we had issued at the time. Let me get the reference page here entitled "Dynamic Response of Freestanding HI-STORM 100 Excited By 10,000-Year Return Earthquake at PFS." That is the report in which I quoted the 10-degree value which

I refer to in this testimony.

Q. Well, that's precisely what I'm asking. Now, Sandia, when they came up with their maximum rotation, they came up with what, sir? Do you recall?

DR. SOLER: It was certainly not 10 degrees. It was --

Q. It was about 1 degree.

DR. SOLER: -- 1 -- 1 something.

Q. Now, one result is 1 degree, your result is almost 11 degrees, and you claim that that distance, the order of, you know, 10 times or 11 times, is somehow consistent and accurate. Is that what you're saying?

DR. SOLER: No. What I claimed, that in 2.000-year return --

Q. I'm talking about the 10,000-year. Answer my question.

DR. SOLER: I don't believe that I've ever claimed -- I think I claimed that they confirmed that the casks will not tilt over.

Q. I see. And that's all you're --

DR. SOLER: That's all I claimed.

Q. Okay. Thank you.

DR. SINGH: Now, can I supplement the

answer, Mr. Soper?

Q. No. I ask questions that I want answers to, and other people do the same, sir.

DR. SINGH: Okay.

MR. SOPER: That's all I have. Thank you, Your Honor.

(The Panel confers off the record.)

JUDGE FARRAR: Mr. Soper, is that all you have at this time?

MR. SOPER: Yes, it is. Thank you, Your Honor.

JUDGE FARRAR: Dr. Lam has some questions.

JUDGE LAM: Dr. Soler, Dr. Singh, for the 2,000-year return earthquake, are the maximum accelerations experienced by the casks within the limits set by the certificate of compliance?

DR. SINGH: The certificate of compliance does not have limits on the -- on the cask accelerations. It has limits on the pad accelerations, on the surface of the pad. It does not -- the CoC does not speak to the acceleration in the cask itself.

JUDGE LAM: Ah-ha. Because when I was reading what the State has proffered, I saw summary results for the pad, okay? So the accelerations don't refer to the pad. Then let me rephrase the question, then.

For this particular facility, do you see the maximum accelerations experienced by the pad within the limits of the certificate of compliance.

DR. SINGH: No. It won't be within the limits of the general CoC.

JUDGE LAM: Do you mean it exceeds the limits?

DR. SINGH: Yes, sir.

JUDGE LAM: Do you know, how much exceedance are we talking about here?

DR. SINGH: The 2,000-year earthquake, the values -- the horizontal values are, I think, on the order of .71.

DR. SOLER: If -- if you -- the horizontals are in the order of .7, and the vertical, I think, is .695. And the formula in the CoC, it is quite apparent that you cannot meet that formula with those combinations of accelerations.

JUDGE LAM: So for the design basis 2,000-year return earthquake, the general license would not apply here?

DR. SOLER: That is correct.

DR. SINGH: I should add that the general license accelerations, Your Honor, are based on simple static tipping of the cask. It does not consider dynamics at all. And that's deliberately done by the NRC, I believe, to set a low threshold for the general license.

JUDGE LAM: Which is what you refer to as a screening limit. Is that --

DR. SINGH: Right, that's correct.

JUDGE LAM: Now, in the same token, for the 10,000-return interval, the general license would not apply?

DR. SOLER: No.

DR. SINGH: The 10,000-year actually does not apply by virtue of the fact that it's not even a design basis earthquake, and the CoC does not deal with beyond the design basis earthquakes.

JUDGE LAM: Yeah, indeed.

If I may direct you gentleman to your answer to Question 170 in your direct testimony --

JUDGE FARRAR: Judge Lam, let me interrupt for a second.

When you say, Dr. Singh, it's not the design basis earthquake, you mean it's not the design basis earthquake under the exemption the

Staff granted you?

DR. SOLER: That's correct.

DR. SINGH: Well, I take a much more provincial view. A design basis earthquake is defined by the facility owner in every case. Every nuclear plant as a design basis earthquake. In this case PFS defined the design basis earthquake. They interacted with the NRC to define it. We take that as the legal regulatory position.

JUDGE FARRAR: For now.

DR. SINGH: For our work, yes.

DR. SOLER: Okay. We're there.

JUDGE LAM: Okay. Gentlemen, the way I read your direct testimony, you -- both of you offer an opinion that even after tip-over, based on kinetic energy impact considerations, the structural integrity of the multipurpose canister would not be breached. Is that a correct reading?

DR. SOLER: That's -- that's correct.

DR. SINGH: That's correct, yes.

JUDGE LAM: Do you have any opinion as to the cooling capacity of the MPC even when the cask is tipped over?

DR. SINGH: Yes. If the cask were in a horizontal configuration, then the ventilation action would be modified. The bottom and top vents will no longer be in the vertical alignment. Instead, the -- the -- some of the vents act as the exit vent, and the ones at the bottom will act in the horizontal configuration as the entrance. The actual transfer of heat will be somewhat diminished from the vertical orientation.

However, the -- the NRC, based on sound science, defines temperature limits for the fuel for normal condition and for off-normal condition, off-normal conditions being those that -- that obtain, but rarely. For example, when the fuel is being loaded, it is subjected to vacuum drying conditions, and it does not have the same level of heat transfer that it does when the cask is in storage. The horizontal tipped over non-mechanistic condition is beyond off-normal. That's a -- that's a -- as I said yesterday, it's a counter-factual scenario. But even under that scenario, we have done evaluations that assure us that the temperature of the fuel cladding will remain below the short-term temperature limits imposed by the NRC, which is 1058, 1,058-degree factor.

Actually, we have done additional

evaluations where we have shown that if some of the ducts were blocked by some non-mechanistic means, still the temperature will remain below 1058.

JUDGE LAM: Dr. Soler, do you have anything to add to that?

DR. SOLER: No. I'm not really involved in the thermal evaluations, so I don't want to make any statements.

JUDGE LAM: Thank you, gentlemen.

JUDGE FARRAR: That concludes the Board's questions.

It's 12:15. Mr. Gaukler, how much redirect do you have?

MR. GAUKLER: I would guess 15, 20 minutes, something like that.

JUDGE FARRAR: And, Mr. Turk?

MR. TURK: Probably 15 to 20 minutes.

JUDGE FARRAR: I would say unless you all think differently, let's press on to make sure we get the witness out rather than take a lunch break. Does that make sense?

MR. GAUKLER: Why don't we take a short 10-minute break, rest room. Is that all right?

JUDGE FARRAR: Okay. It's -- let's shorten it and be back at 12:25.

(A recess was taken.)

JUDGE FARRAR: Before, Mr. Gaukler, starting your redirect, Judge Kline, has a follow-up question to something that was said before.

JUDGE KLINE: One of the hazards of taking a break is the Board thinks of more questions.

One question has to do with the cooling that you mentioned just before the break, and you mentioned it was a short-term cooling standard. So the question we have is what's the -- what is the time limit on that short-term standard, and what happens, if, in fact, it's exceeded?

DR. SINGH: The NRC, to my knowledge, has not specified a specific time limit.

JUDGE KLINE: Okay.

DR. SINGH: We would consider, based on the underlying principles, that if the casks were, say, horizontal for 30 days, that would be a short-term duration.

JUDGE KLINE: I see.

DR. SINGH: This is strictly my personal interpretation of the underlying principles that leads -- which led the regulators to establish a short-term limit.

JUDGE KLINE: It sounds like something we should explore with the Staff.

DR. SINGH: Or you can explore with me. I can explain the fundamental principles.

JUDGE FARRAR: Well, not so much the fundamental principles. As you say, for a short term, whatever that means, it would be under the melting temperature of the fuel. But whatever the short term is, that implies that sometime later the fuel might get -- might exceed the -- or the cladding might exceed the melting temperature. And so then the obvious question is what would you do -- you know, if you knew 30 days was all right and 35 was not, what would you do on day 32 to stop any melting from occurring?

DR. SINGH: Let me -- let me clarify first, the off-normal temperature limit is not the melting temperature. It's far from it. Off-normal is just another NRC conservatism. They specified a temperature limit -- as a matter of fact, 10 years ago the temperature limit in at least one certified cask was 1200 degrees, and they lowered it to 1058, I guess in the interest of additional conservatism.

But that is not the melting temperature

at all. There is scenario where the fuel cladding -- we can't postulate where the fuel cladding will reach melting temperature, especially at PFS, and the reason is this: Before fuel comes to PFS, it has to be cool enough so it can be transported, you know, in a cask that has no vents, no ducts, no openings, and because of that, the fuel already is -- it's so cool that it's impossible to physically make that reach the melting point by -- through any natural means.

JUDGE LAM: Dr. Singh, are you saying you have done the analysis to demonstrate if the casks tip over, sitting in a horizontal position, their will not be any fuel melting even if it sits there for --

DR. SINGH: Yes. The answer is yes. We have actually done -- I'm offering more than you asked. We have actually done evaluations where we have assumed that the cask is buried under debris for a long time, and we could not reach melting temperature --

JUDGE LAM: Buried, you mean covered with soil?

DR. SINGH: Covered with debris.

JUDGE LAM: I see.

DR. SINGH: And we could not reach melting temperature for the kind of heat loads that PFS will have.

JUDGE LAM: Even when it's sitting horizontally so that --

DR. SINGH: Horizontal, vertical, any configuration. If you cover it with debris, then there's no ventilation.

JUDGE LAM: Are you saying all the passages were blocked?

DR. SINGH: All the passages are blocked.

JUDGE LAM: All the cooling passages were blocked --

DR. SINGH: It would not reach melting temperature for PFS heat loads.

JUDGE LAM: Are you saying that in that situation, you rely on just conductive heat transfer?

DR. SINGH: Conduction, that's exactly right, and radiation.

DR. LAM: And conductive heat transfer to surface?

DR. SINGH: Some conductive heat transfer, yes.

We have done in-depth studies of the behavior of the cask under a variety of natural phenomena, and it's absolutely stable, totally. There's no potential fuel clad melting at all.

JUDGE LAM: Is the Staff aware of these types of calculations?

DR. SINGH: The staff is in our shorts all the time. I'm sure they know.

JUDGE FARRAR: Dr. Singh, when I jumped into this line of questioning, I thought it was going to lead to a discussion of what remedies you would take, whether that was cooling or righting the cask or so forth. In light of what you've just said, it's going to lead to something else. You're not getting out of town for a while.

I assume the State will want to pursue this -- your conclusions just stated either with Dr. Singh or with somebody.

MR. GAUKLER: I would say, Your Honors, the issue of fuel degradation is covered in part in the testimony on radiation dose consequences which is part of Section E. So this same topic is kind of natural there, so I don't have to stay -- think it necessarily would require him to stay today.

JUDGE FARRAR: Are you able to come back

at some future point?

DR. SINGH: I am in principle committed to providing support to this Board and to the State of Utah to come to the -- to get a good, solid understanding how safe these systems are. Yes, I'll be back if necessary.

MR. TURK: May I note also, Your Honor, that Mr. Guttman, who is now sitting beside me, was the thermal reviewer for the HI-STORM cask. He's listened to the questions and answers, and he will be part of the testimony with Dr. Luk. And he's available to respond to your questions as well.

JUDGE FARRAR: I might say to the State this is an example of when we ask questions -- we have questions we don't know what the answers will be, but any time if the answers are something that you want to take up at greater length, certainly we will give you the opportunity to do so.

MS. CHANCELLOR: Your Honor, I believe that this line of questioning -- I know it was initiated by the Board, but it does get into the area of Part E of the Unified Contention. And we would take issue with a lot of things that Dr. Singh had said, and those issues will be litigated in Part E. And for now we will not follow-up, and I believe that Dr. Singh, Dr. Soler and Dr. Redman will be a panel on Section E.

JUDGE FARRAR: Okay. I just wanted to make sure your interests were not harmed by any questions we asked.

DR. SINGH: In Washington I hope, right?

MS. CHANCELLOR: Depends how the Board viewed Dr. Singh's responses whether our interests are harmed.

JUDGE FARRAR: Mr. Gaukler, go ahead.

MR. GAUKLER: Yes.

REDIRECT EXAMINATION

BY MR. GAUKLER:

Q. Towards the end -- excuse me -- towards the end of your cross-examination, you were discussing -- Mr. Soper was discussing with you the differences between the Sandia report and your report, particularly with respect to the 10,000-year earthquake. Dr. Singh, you were about to supplement with something. Do you see any particular difference between -- in terms of the results from the Sandia report and the Holtec report, do you see any discrepancy between them, so to speak?

DR. SINGH: No. I find the two pieces of work to be quite consistent in the results they produce. The -- the Holtec evaluation, it can be a designer's assistance.

We make conservative models deliberately. We want to leave margins. We don't want to get close to the limit. Our model had built-in features of safety that exaggerate the response of the cask under the earthquake.

Sandia, which does evaluations, analyses, tries to replicate the behavior of the system as best -- as best as that they can. That's their mission at the National Laboratory. They model, for example, in this case the foundation, the soil subgrade as a continuum. We modeled it as a set of discrete springs. The -- they used a computer program, and they clearly expended a great deal of effort in trying to simulate many aspects of the model, of the actual system that we, as designers, use -- if you'll recall -- we call it -- we use a conservative approach to simulate them.

Now, that said, I should inform you that the Sandia solution also has many aspects of conservatism. The rotations that they predict, which is based on a more exacting model, are not the rotations, in my view, that will obtain in nature if that 10,000-year earthquake were to occur.

And the reason -- I'll give you a simple example. We model the overpack and the MPC and the fuel inside it as a solid monolith. In reality, in reality during an earthquake, under vibratory motion, the fuel assemblies will vibrate. They rattle inside the storage location. So does the MPC. They do not -- their motion during an earthquake is not synchronized. It's not in unison.

Now, our studies have shown -- for example, we did some work on planar motion of rattling of fuel, and we found that only 44 percent of the total mass of the fuel participates in the dynamic motion. The aggregate effect of the fuel mass is only 44 percent of its total mass because of the rattling effect. Now, consider that if the mass that is trying to overturn the cask is reduced in that manner, of course the response will be reduced.

Now, these things are not considered in our solution deliberately to have built-in conservatism. With all this -- and there are many,

many other conservatisms. Clearly, the cask is not rigid. The cask has a finite amount of energy absorption capability. After all, if you subject it to large force, it will deform. Clearly the structure can absorb energy. We don't consider that. We did not consider the flexibility in the MPC. We do not consider the flexibility in the fuel basket, the fuel and so on. So all this adds to the layers of conservatism that the model -- we get so used to it, we don't even think about it. We have -- we have kind of permanently embedded these conservatisms in our model.

So, yes, our solution is very conservative. Sandia's solution is conservative. The reality will be something less than what I believe Sandia predicts in terms of the rotation of the casks during earthquakes.

Does that answer you, Mr. Gaukler?

Q. Yes, it does. Thank you.

Yesterday in the discussion you were discussing various types of damping, Dr. Soler and Dr. Singh. You may have clarified that somewhat today, but I'd like to have you go -- try to clarify it more, the different types of damping and particularly the different locations of damping as they relate to the system that we're talking about here.

DR. SOLER: We talk about material damping, impact damping, soil damping. I'm going to break it into -- and structural damping. Material damping -- and I -- as I describe it, I will tell you what is and is not included in the PFS analysis.

Material damping is the kind of damping you would get because of the inelastic behavior of the material. A classic case would be an elastic plastic material where you have yielding during your loading, and every time you have yielding, you get some energy dissipation. Material damping is not modeled in our system because we do not model the cask as either an elastic or inelastic material. We model it as simply a rigid body.

Structural damping is the damping you ascribe to a given structure to account for things that you perhaps have not modeled in detail in the structure. For instance, in a bolted joint that forms a very small part of a large complex structure that you're analyzing, the fact that the surfaces rub against one another during a loading -- during -- under any loading condition will lead to some energy dissipation, and if you look at the structure as a whole, you can ascribe an effective damping to that to model the energy dissipation that's observed.

And the -- some of the Reg Guides give you a broad overview. For a certain kind of structure, you're allowed a certain amount of damping if you're going to model structural damping.

But the use of structural damping assumes that you have modeled your structure as an elastic material. Since we have modeled our structure as a rigid body, we do not have any structural damping included.

The only two damping mechanisms that are left are the dampers that we have interposed between the cask and the pad to model the energy dissipation which is physically observable when you have an impact and the soil damping. Now, the soil damping is made up of a -- of a number of mechanisms, but we have simply either included that damping in accordance with recognized formulas in ASCE or we have arbitrarily chosen the soil damping to be very low.

As far as the impact damping is

concerned, we have in our 2,000-year design basis used a very conservative representation of that damping as 5 percent of critical damping. Critical damping has a formal definition in the vibration literature as 2 times the square root of the mass times the stiffness. A percentage of critical damping would be to take whatever value you compute from that formula and multiply it by that percentage divided by a hundred to get the damping value for your system.

So in the 2,000-year design basis runs, we used a very conservative value of 5 percent of critical damping. For the 10,000-year scoping analysis and the first report which considered just a vibrating pad and 1 cask, we assumed that the damping between cask and pad was equivalent to 40 percent of critical damping. The State, in their solutions that they have run on their simple models, have used damping at that location as low as -- as low as .01 percent of critical damping.

So in all of the analyses that have done -- that have been done for PFS on one side or another, different values of damping have been ascribed to that interface, and the only way to describe what is correct is, again, to resort to physical principles and observable facts.

Have I --

Q. Yes.

Dr. Singh, do you want to add something?

DR. SINGH: Yes. The impact damping for the HI-STORM-to-pad interface, in my opinion, would be well in excess of 40 percent. I base this on the fact that you actually can calculate damping between two interfaces. You actually can. It's a tedious effort, but you can do that.

We did some evaluations where we -- we postulated that a metal cask drops on a thick, very thick concrete foundation, and we simulated that on a computer program that allows treatment of material not in reality, in other words, allows the physical phenomenon of impact to be simulated. Having done that, we determined the amount of bounce back from the -- from the cask. You drop the cask, you see how much the cask bounces back. You can directly, by the amount it bounces back, calculate the damping.

We did that evaluation, and we found that the damping, even for a metal cask, all metal cask, was in excess of 50 percent. So 40 percent is, in our view, a conservative value consistent with our approach to use conservative numbers in our analyses.

Q. Dr. Singh, yesterday you were being questioned by Dr. Soper in terms of the use of DYNAMO model for PFS as opposed to, say, for example, the VisualNastran with respect to analyzing the design basis earthquake, and you've mentioned that there were compelling reasons for the use of DYNAMO. Would you please elaborate on the reasons you believe exist for the use of the DYNAMO code with respect to the evaluation of the PFS 2,000-year design basis earthquake?

DR. SINGH: Well, I kind of forget the context of the question that was asked, but DYNAMO has, in our -- in our experience, shown to be impeccable with respect to its performance in dealing with problems where tilting impact and so on can take place.

We actually had modelled ANSYS, which is a general purpose program, and we have found that in the case of a simulation of an earthquake on a freestanding structure, it was given unstable, actually incorrect results. General purpose programs have the capability to do a large variety of problems, but in a specific instance, in that specific instance, we found that ANSYS was not accurate whereas DYNAMO did give a stable and correct solution.

This program having been used -- DYNAMO having been used in over a thousand discrete structures, qualifying them, is a well tested program, and that was the reason I said there were compelling reasons to use it when the question was asked.

Q. Dr. Singh and Dr. Soler, in the context of the questioning, it was discussed that DYNAMO was a small rotation program as opposed to VisualNastran and, because of large angles of rotation, you would want to go to a different program. Do you have a range in which you would say that DYNAMO would be capable of evaluating rotations?

DR. SOLER: Well, if you -- outside of any computer program, if you simply take the classical physics problem of asking yourself that you want to examine a body in a slightly rotated position subject to a side force and resisted by simply gravity, intending to pivot around its edge, you can write the equations of motion for that or the equations of equilibrium for that, invoking what I'll call a small motion or small rotation principle where you replace the sine of the angle by the angle itself and when you replace the cosine of the angle by the -- by 1 plus Theta squared over 2 -- I don't want to embarrass myself here. In any case, you can also assume that you use the sine and cosine directly and form the imposed moment versus the resisting moment to determine the position -- the final configuration.

So in that simple problem you could use as a measure of when small deflection theory breaks down by simply looking at the stability relationship which would involve, in that case, the tangent of the angle Theta. So if you take for different angles Theta from, say, 0 to wherever you want to go and compare the value of tangent Theta to the value Theta itself, you find that at about 20 degrees you -- I believe it's roughly about a 10-percent difference between tangent Theta and Theta. And if you go way back into mathematics, tangent Theta is also expressible in terms of a power series in Theta, so the question you're really asking is when is the power -- the first term of the power series able to represent the trigonometric function.

And I would say in engineering problems, depending on the particular application you're looking at, 10 percent may be even -- less than 10 percent would be a place beyond which you necessarily don't want to go without a program that can treat -- capable of handling large rotations. And if you go there with a small rotation problem, you go there with full knowledge that results that you get may not be -- I won't say incorrect, but it -- at the very least they may be inaccurate.

Q. Also, in yesterday's cross-examination the question came up with respect to using a VisualNastran, for analysis at Diablo Canyon with respect to the anchored casks to be used there as opposed to using DYNAMO for that analysis. Would you please explain the reasons why VisualNastran was used with respect to the analysis of the anchored casks at Diablo Canyon?

DR. SOLER: The analysis at the ISFSI pad was truly for an anchored cask, and we could have equally well used DYNAMO or VisualNastran at that location. However, the project as a whole had to look at the transfer cask in the spent fuel pool, coming out of the spent fuel pool, sitting in the cask wash-down area, being raised and lowered during that time, being put on a transporter, the transporter moving over the road to a cask transfer facility and eventually getting into a HI-STORM -- into an anchored HI-STORM on the pad.

The sum total of all of the analyses that we performed in our role for Diablo Canyon, the ISFSI calculation was only one part of that, and we determined at the outset that, to the extent possible, we were going to use one program to encompass all the analyses. And some of the analyses in the pool building, as we did them, definitely had the potential of having large rotations. So we -- we were using VisualNastran from, in effect, the cradle to the grave during the whole project.

Q. Also, the question came up, Dr. Singh, a question of you with respect to the use of DYNAMO or VisualNastran for the HI-STORM 100-SA, which is, I take it, the HI-STORM anchored cask -- 100-S anchored cask. Would you please describe in more detail how DYNAMO was used or VisualNastran was used in respect to that cask? And we're talking about, I think, the Holtec filings with the NRC.

DR. SINGH: I understand. We used -- for the anchored cask configuration, we used both

DYNAMO and VisualNastran. The DYNAMO solutions are reported in complete detail in a topical report that we provided to the NRC in the 1998 time frame. That report is entirely predicated on the use of DYNAMO.

The recent submittal that we made to the NRC where we were in the midst of doing evaluations for Diablo Canyon anchored situation and also attempting to get a general CoC, we stayed consistent and used VisualNastran. But on our document with the NRC, there are solutions for using both DYNAMO and VisualNastran.

Q. And were there any material differences between the solutions using DYNAMO and VisualNastran?

DR. SINGH: There are no differences in the sense that if you use the same data, you get similar answers. But in our -- in our dealing with the regulators, you know, we -- we vary the margins in the parameters, central parameters, so the actual numbers may be different, but the consequence, the solution, in the end, is the same.

Q. Yesterday, Dr. Singh, in the questioning by Mr. Turk, you were discussing the diametral gap between the multipurpose canister and the HI-STORM cask inside of that cylinder, the HI-STORM 100, and you mentioned some numbers from memory. Have you had a chance to go back and check those numbers and get precise numbers for that?

DR. SINGH: Yes, yes. It was bothering me all afternoon, but I was speaking from some fuzzy memory and I wasn't sure if it was quite right.

I borrowed the CD from the NRC. They graciously lent it to me so I could check the FSAR. The range that I'd given yesterday was between 2 to 4 inches diametral range. The range actually is wider. It's from 3/4 of an inch, minimum, to 4.75-inch maximum. The effective gap that retains the MPC in its place over most of its axial extent, the diametral gap is 3/4 of an inch. The radial gap, therefore, is 3/8 of an inch.

I apologize for having not consulted my information before -- before speaking yesterday.

Q. Dr. Singh and Dr. Soler, you were shown State Exhibit 174, which was a paper concerning the HI-STORM 100 cask. First of all, the HI-STAR 100 cask is a different cask system from the HI-STORM 100 cask system; is that correct?

DR. SOLER: That is correct.

Q. And it was referring to c over g of approximately .25 in there. In general, what's the margin of safety in terms of the HI-STORM 100 cask with respect to tip-over?

DR. SOLER: Well, the pure geometry, if you -- if you look at the two casks, they were both on the -- on the computer background yesterday. The HI-STAR transport cask's base is 83.25 inches, and its height, total height, is about 203 inches, I believe. And, again, I'm working without consulting the FSAR, but I believe that's a correct number.

So if you look at the ratio of height to diameter, that's 200 over 83 -- I won't calculate it. Compare that same number with HI-STORM where, in our analyses we were using 231, which is the height, divided by the diameter, which is 133, roughly 50-percent larger, you will find that the ratio of height to diameter of the HI-STORM is roughly -- it's less than 2. I think it's about 1.8. And the ratio for HI-STAR is 240 over 80, it's 2.something. So the HI-STAR is inherently less -- the HI-STAR is more prone to incipient tipping, if you will, incipient tipping meaning the classical solution at what coefficient of friction will you just begin to tip.

So the HI-STORM is inherently stable. They both have a well defined center of gravity-over-corner position, and if you were doing a dynamic analysis of either, depending on the scenario, meaning whether you were trying to do an actual calculation of what might happen under bounding conditions or whether you are doing a calculation to establish or suggest a regulatory position may differ, but in either case or in any case, I would adjudge a factor of safety against overturning against that cg-over-corner angle.

So in the case of the beyond the design basis analyses that I performed here in two reports, taking the 10-degree angle that I have quoted for one of the them, the cg-over-corner angle is 29 degrees plus a little bit. Therefore, I would ascribe the -- in reality, I would ascribe the safety factor against reaching cg-over-corner to be just under about 3. It would be 2.9.

As far as the safety factor against overturning, I mean if you ask a direct question as to exactly when will the cask overturn, the only way you could answer that is by numerical experiments of raising the strength of the earthquake, because this -- in a dynamic situation, the way the casks are behaving, they're not only tilting, but they're precessing. You get very complex motions with --

Q. What do you mean by precessing?

DR. SOLER: Precessing, I guess the simplest example of precessing is a child's top where you buy it out of the store, you press the handle, it spins, you let it go. And then it will eventually start to wobble and move around in a circle before it falls over. Precessing is basically the -- well, this is easy. That's precessing.

DR. SINGH: It's a helical motion.

DR. SOLER: A glass rolling around its rim, its base, that's precessing motion.

All those motions taken together, I would not preclude a cask going beyond it's cg-over-corner angle for some instance of time and then being able to right itself.

DR. SINGH: Just like a bicyclist going around a curve, you know, the cg could be outside of the footprint of the bike, but it doesn't fall over. So you have -- you can have in a cask also -- even though we set the cg-over-corner as a limit, in reality, there are additional margins.

Q. Thank you, Dr. Singh and Dr. Soler.

One last question, I believe.

JUDGE FARRAR: Mr. Gaukler?

MR. GAUKLER: Yeah.

JUDGE FARRAR: You mean there's going to be a gyroscopic action on the casks?

DR. SOLER: I would not be attributing the safety of this cask to gyroscopic action. I'm just stating that looking at those analyses, I cannot be persuaded that if I ever calculated 29.7 degrees that the cask would necessarily tip over. It may right itself.

DR. SINGH: And it's strictly a theoretical evaluation. All we are saying is that if you were to look at the stability of a cask statically, rotation by 29.3 degrees, if you leave the cask at that location and slightly move it over, it will tip-over in the static world. In the dynamic world, the structure is more tolerant. It would -- it may be leaning to a greater angle, and still it would not tip over. It's strictly a theoretical postulate. Our belief is that even a 10,000-year earthquake -- as I said earlier, Sandia's report that shows rotations in the order of 1 degree is closer to the truth than ours is, which shows about 10 degrees.

JUDGE LAM: But shouldn't the optimism for taking credit for precessing be moderated somewhat by if you -- if your center gravity over the corner, you may have some residual momentum going that way, so by the time you're there, you could tip over --

DR. SINGH: Yes, yes, you could, and the solution will predict that. The capability of the program that can do the true geometric nonlinearity means that it can -- it has the capability to predict accurately -- accurately within the constraints of the model, capture movement of the centerline. This statement was made strictly to -- to -- in the theoretical space. In reality, the rotations of the cask are going to be very small, even in the 10,000-year earthquake postulate.

JUDGE FARRAR: Mr. Gaukler, you had one more question?

MR. GAUKLER: I think I have no more questions. You took my one question, Your Honor.

JUDGE FARRAR: Mr. Turk?

MR. TURK: Yes, Your Honor. Do you want me to start now?

JUDGE FARRAR: Yes, please.

RECROSS-EXAMINATION

BY MR. TURK:

Q. Gentlemen, let me ask you, first of all, if you're familiar with the history of the VisualNastran model. Do you recall if it was ever called by some other name?

DR. SOLER: Yes. It was originally called Working Model, and it was developed -- I'm not privy to the details of the development, but there was a firm called Knowledge Revolution which was a small firm up in the San Francisco area. They were bought out by MSC Software, I suspect, about 3 or 4 years ago, and after an initial period of one year, they changed the name of the code to VisualNastran Desktop because MSC Software is one of the suppliers of the Nastran computer code.

Q. Is it correct also that Holtec utilized Working Model in their submittal of the HI-STAR transportation cask to the NRC?

DR. SOLER: Yes. Did you want me to elaborate or did you just want a yes --

DR. SINGH: It's true that -- I believe we used it in HI-STAR.

DR. SOLER: We did use it in HI-STAR in the transport submittal.

Q. Yes, and the Staff accepted --

DR. SOLER: The Staff accepted --

Q. -- the HI-STAR submittal --

DR. SOLER: Yes.

Q. -- using that code.

DR. SOLER: Yes.

Q. Is it the same code? I believe for HI-STAR it was Version 3.0/4.0. Is that the same as the current --

DR. SOLER: Yes, and it was the two-dimensional version. The current code is now denoted by its year. The code that we used on the early reports were 2001, and on -- I believe on the recent calculations here it's 2001 R2.

DR. SINGH: Mr. Turk, we are routinely receive updates on general purpose programs from the owner of the program. We buy the program, you also buy updates. When we get an updated program, such as the case with Working Model, we got -- we received later updates. We have a stringent quality assurance program. We put the new update, check it against the old proven version, and we run -- we run a variety of test problems according to our quality assurance program to ensure that the new program does not have any bugs. And there have been instances where we have not accepted a later rev of the program because we found that it had bugs. So it's -- it's a -- it's a rigorous, well orchestrated process at our company.

Programs are continuously being improved. For example, VisualNastran now has visual capabilities, as you saw yesterday. Working Model did not, not to that extent.

DR. SOLER: Not to that extent. We couldn't make a movie with Working Model, but it -- you could watch the motion in real time.

Q. You mentioned in your testimony the Diablo Canyon application. Is it correct that to this date the NRC has not yet accepted the Diablo Canyon application or has not yet approved --

DR. SINGH: To my knowledge, the application is currently being reviewed by the Staff.

Q. So we don't know yet whether the Staff has accepted the use of VisualNastran in the Diablo Canyon application?

DR. SINGH: That is correct.

DR. SOLER: Could I add a little bit to

that?

The Staff has accepted the latest version of the HI-STORM FSAR, and that includes a section on anchored casks where VisualNastran has been employed.

Q. For which cask is that?

DR. SINGH: HI-STORM.

DR. SOLER: HI-STORM 100-SA.

Q. The Staff has not yet issued the final approval -- not yet issued the certificate of compliance for that, have they?

DR. SOLER: That is correct --

DR. SINGH: We have the SER, and it's in rulemaking right now.

Q. And is it correct that current status of -- the Staff has published for comment the draft SER?

DR. SINGH: That's correct.

Q. And in the draft SER the Staff proposes to accept --

DR. SINGH: That is correct.

Q. -- the use of that model?

DR. SINGH: I don't know if there are explicit statements in the SER, but my understanding is the anchored HI-STORM methodology and solutions were found to be acceptable to the Staff.

Q. So we're waiting to see if that's ultimately the final decision in the CoC if and when it issues?

DR. SINGH: I believe it's -- June the 10th is supposed to be when the period ends.

Q. That's the comment period?

DR. SINGH: Yes.

Q. Earlier in testimony today, it may have been a mistake in my hearing the answer, but I thought I had heard you say that the NRC directed your use of a -- or I'm sorry, that the NRC set the lower limit --

DR. SOLER: No, I meant to --

Q. -- of cask tip-over.

DR. SOLER: By that I was referring to Reg Guide 161 which ascribes the appropriate damping that you could use, structural damping that you could use for, say, welded structures, bolted structures. I think in that case it's probably a maximum limit.

MR. TURK: I'm sorry, Your Honor. May we just take a minute?

Q. The NRC Staff did not direct you to use any particular static analysis, did they?

DR. SINGH: No.

DR. SOLER: That is correct.

DR. SINGH: The Staff does not direct. The Staff only asks questions.

Q. So, in effect, what happened, then, is you proposed your own or you used your own static analysis submitted to the Staff for approval, and then the Staff provided its --

DR. SINGH: That is correct.

Q. -- its comments as to whether it's acceptable or not?

DR. SINGH: Right.

Q. Yesterday there was some testimony about cold bonding, and I believe, Dr. Singh, you were talking about the cold bonding force where steel is in contact with other steel. Those are not the forces that you would expect for cold bonding between steel and concrete, are they?

DR. SINGH: I cannot visualize the -- even physically the potential of steel bonding with concrete because the material is so dissimilar and concrete has such a low compressive strength that before it will bond, it will crush. You know, if you apply pressure, increasingly larger pressure, before you will create a bond, it will crush. I just can't physically visualize a process where cold bonding between concrete and steel can be carried out, industrial.

Q. In that case, in your response to Judge Farrar's question earlier today when you were discussing the potential use of the value of 1 for bonding, would you believe, then, that that's a totally theoretical but impossible value that would be achieved in the real world for the cask-on-concrete situation?

DR. SINGH: Yes. Actually, the value of concrete-to-steel interface friction coefficients -- and they were measured a long time ago. You know, when that used to be the state-of-the-art research to do 60 years ago, people were measuring friction coefficients. And the values that are quoted in the literature do not, to my knowledge, exceed .7 under any measurement. So that's why we used .8 as an upper hypothetical limit. It's a -- the approach in friction, being the friction determinate, needs to be lavish with respect to the range of parameters that we study.

Q. Okay. And, Dr. Soler, in your written testimony Ms. Nakahara was asking you about something that appears I believe on page 82 of your testimony in Answer 144. At the top of page 82 there is a mathematical formula depicted where you take 360,000 pounds, divide it by 454 million pounds per inch. Could you take a moment to do that calculation and see if your testimony correctly represents the value?

I have a calculator which has proven invaluable to me, if you'd like to borrow it.

DR. SOLER: All right.

JUDGE FARRAR: Off the record.

(A discussion was held off the record.)

DR. SOLER: It's actually .0079, so there is two extra 0s in that.

Q. Will you try that one more time?

DR. SOLER: I'm going to go back to school.

Q. It's not the best key pad on that calculator.

MR. SOPER: Is that the calculator the Staff uses for everything?

DR. SOLER: It seemed to stop --

MR. TURK: Actually, the calculator I used yesterday very well, the State told me it was too simple a calculator.

DR. SOLER: I think -- I think your calculator runs out of gas with the 454 million.

Let me -- let me -- wait. I'll take off the three 0s. There we go. Three 0s. .00079 is the answer to that division, so I believe that there is an extra 0 in that calculation result on line --

Q. Knowing that the correct value is essentially an order of magnitude different from what's stated in the testimony, does that change in any way your answer to this question?

DR. SOLER: No, because as you'll note in the next to the last line, there is a value quoted of .009 which is even an order of magnitude larger than that, and we feel that that is acceptable also.

Q. There was some examination about PFS Exhibit 86 which is the Beyond the Design Basis Analysis, and there's a table that has the 11 different cases.

DR. SOLER: Yes.

Q. Which of the cases in that table would you believe represents the most realistic case for the 10,000-year earthquake?

DR. SOLER: Well, all of them are realistic because it's possible to load in the manner in which the casks are loaded. I do not think any of the analyses using the tune stiffness, at least for 1 cask and 8 casks, is realistic. That was deliberately chosen as bounding. The most realistic is probably 11, if you assume that the soil moduli are consistent with the 10,000-year earthquake, and, of course, No. 1, which is also based on soil moduli which are consistent with the earthquake level.

As far as friction is concerned, regardless of anything else, I would think the real case would be Case 5 and 10, which is random coefficients of friction. As a rule, though, those would not be considered suitable for submission to the regulators because they're not reproducible.

Q. In your opinion, are any of the cases depicted here actually realistic cases for a 10,000-year earthquake?

DR. SOLER: 11, 10, with the conservative assumption that the soil is tuned. Same for 9. That uses a lower bound of .2. The damping in all of these cases, I believe, is extremely conservative, so I guess I would summarize my answers is that they're all realistic, but they're all very conservative.

Q. And the most realistic one would be Case No. 11?

DR. SINGH: None of them, in my opinion, are realistic. They are all run with extremely conservative parameters. Key parameters which are used in the solution are extremely conservative. Each one of them gives an exaggerated response of the actual problem. None of them should be considered realistic in the sense that you will actually expect it if you were to subject the equipment to the earthquake.

Q. Earlier today there was some discussion of the Sandia report, and just to make sure I understand your testimony, you degree that Sandia did not use any soil springs in its analysis?

DR. SOLER: That's correct.

Q. And they did not use any dampers between the cask and the pad?

DR. SOLER: I --

DR. SINGH: I believe they have.

DR. SOLER: Well, I believe -- I'm not sure of that, but the normal way of simulating contact would certainly require you to ascribe a stiffness there. But I do not know whether they also ascribed a damper there.

DR. SINGH: If they did not put a damper, then they made their model very conservative.

Q. It's also correct that Sandia did not confirm your methodology, but, instead, they confirmed that the results --

DR. SINGH: I don't know what their task was. You know, we don't know what their mission was.

Q. But you were asked whether you believe the Sandia report confirms your analysis. And I'd like to clarify that the Sandia report confirmed that your results are within the range that they would consider appropriate, but they didn't go out and confirm your method of analysis, did they?

DR. SINGH: I don't believe they have reviewed our analysis and specifically concurred with it in the actual analysis model. If I stated that they -- they have validated our analysis, that was a misstatement. They -- all I meant to state is that their solution -- considering the level of additional detail they used in their model, it is not unreasonable to expect that their solution would be closer to reality than ours, being that we used many, many conservative elements in our --

JUDGE FARRAR: And when Mr. Tuck suggested that confirmed your result, that would be limited to confirming that you said the cask wouldn't tip over and they said the cask wouldn't tip over?

DR. SINGH: That's correct.

DR. SOLER: Correct.

JUDGE FARRAR: But not going beyond -- not going beyond that?

DR. SINGH: That is correct.

Q. (By Mr. Turk) Do you know also whether Sandia considered a case in which all 8 casks were placed upon the pad?

DR. SINGH: No. I don't remember.

JUDGE FARRAR: We're getting to the point where we need to make a choice.

MR. TURK: I'll be done in a moment.

JUDGE FARRAR: You can ask -- if you can ask fewer questions, the witnesses can give shorter answers, we could eat or -- because you can't leave until the state gets another go-round.

MR. TURK: Your honor, at this point I don't think I have any other questions.

JUDGE FARRAR: All right. Ms. Nakahara?

MR. SOPER: Actually, these are probably questions that fall in --

JUDGE FARRAR: Go ahead, Mr. Soper.

MR. SOPER: Thank you, Your Honor.

FURTHER RECROSS-EXAMINATION

BY MR. SOPER:

Q. Dr. Singh, you're not changing your testimony that DYNAMO is not capable of analyzing large rotations, are you?

DR. SINGH: No. I did not -- I do not claim to state that DYNAMO would produce accurate results for very large rotations -- situations in the cask where large rotations can occur.

Q. And of the various NRC proceedings where you've used DYNAMO, you certainly haven't asked the NRC to rely on a DYNAMO analysis for anything involving large rotations, have you?

DR. SINGH: To our knowledge, we have not had DYNAMO used, to my recollection, where large rotations -- large rotations occurred.

Q. Well, given your acknowledgment that it's incapable of that, I guess that makes sense.

DR. SINGH: Well, incapable would be

perhaps a poor choice of terms. A program -- Dr. Soler explained earlier today and I explained yesterday, a program that does not have the geometric nonlinearity modeling capability, it will become increasingly more inaccurate as the solution becomes one of large rotation. It does not mean -- you will -- you will get a solution. It will have a greater inaccuracy to it than if you were doing a problem that involved only small rotations.

Q. Well, Dr. Soler put it this way: It's not capable without modification of modeling the potential for a cask to execute a large rotation. Do you agree with that?

DR. SINGH: I agree with it in the tech -- as spoken by an engineer. In the legal statement, you can sparse -- you know, parse words and come up with different conclusions. I think he means to say that the program will -- the solution will be part -- will become less accurate -- as the problem itself begins to exhibit -- the problem begins to exhibit large rotations, it becomes increasingly less accurate. Inapplicable is a strong term.

Q. All right. Well, incapable was the word that Dr. Soler used.

It's that very reason, sir, is it not, that you did not use DYNAMO in the 10,000-year analysis at PFC -- PFS?

DR. SINGH: That, I believe, is correct.

DR. SOLER: That is a correct statement.

Q. And with respect to the difference in -- going to the Sandia analysis, now, with respect to the difference in the 10,000-year results indicating that -- the Holtec results indicating almost 11 percent cask rotation --

DR. SOLER: 11 degrees.

DR. SINGH: 11 degrees.

Q. -- 11 degrees, excuse me, compared to the Sandia results indicating more like 1 degree, Dr. Singh, I think you told us that that was not surprising or remarkable in your view.

DR. SINGH: That's correct.

Q. And that would be because the various engineering judgments involved in running the two analysis are to be expected?

DR. SINGH: The -- I would call it engineering judgments. I would say the details of the model -- we tried to capture the response of a complex structure. In doing so, two groups of analysts would use -- they used clearly different programs, as you see in this case.

They also go to a different level of articulating the problem in the model. The less the level of articulation, in our case being -- as I said before, we are designers. We try to stay -- keep things conservative. We will -- our model will typically, typically predict a greater rotation, much greater than the nature will obtain.

Now, another analyst -- another group of analysts who is trying to study the behavior with a greater focused effort and larger model can get closer to the truth. The actual rotation may be -- in nature may be half a degree. And Sandia has gotten, through a more elaborate model -- and I'm only giving my opinion -- closer to the truth than our model which is loaded with conservatisms in many, many respects would.

But, again, our object is not to predict tip-over. Our object is to conservatively prognosticate whether tip-over will occur.

Q. I see. So these large differences are explainable due to the way the problem was modeled by the various analysts?

DR. SINGH: That is correct.

Q. I see. And not surprising or remarkable in your view, I guess you'd say?

DR. SINGH: It is not. Considering the caliber of work Sandia has done and the caliber of work done by Holtec, I believe that we have both modeled the problems with reasonable accuracy.

Q. Okay.

DR. SINGH: The differences in solutions are only because of the differences in the level of articulation in the -- of the physical problem into the model.

MR. SOPER: Thank you. That's all I have.

JUDGE FARRAR: Okay. Then that concludes it.

I just have one curious question. What does STAR and STORM stand for, if anything?

DR. SINGH: They do stand -- STAR is -- H-I, you know, is Holtec International. It's also a nice word in English. STAR stands for storage transport and repository.

This cask can be used for storage, HI-STAR, it can be used for transport and it can be taken to the repository.

JUDGE FARRAR: No commercials. Just answer the question.

DR. SINGH: STORM --

MR. GAUKLER: Got you on that one.

DR. SINGH: STORM, incidentally -- just a definition. STORM is storage module, STOR, M, storage module. It's strictly for storage.

But hopefully you will publicize it for the world.

JUDGE FARRAR: We thank you for your testimony. You will be back later in the proceeding, another panel with another gentleman, so we'll see you then.

It's now 1:30. How long will the State's cross-examination of Mr. Trudeau take?

MS. CHANCELLOR: Your Honor, I believe that we'll get through Mr. Trudeau and Mr. Ebbeson today and -- what day are we at?

JUDGE FARRAR: This is Wednesday.

MS. CHANCELLOR: Wednesday. We'll be through -- Mr. Trudeau will take quite a while. Mr. Ebbeson and the following witness won't take as long, if you're worried about where we are in the schedule. But Mr. Trudeau will take a long time.

JUDGE FARRAR: Then in terms of the court reporters, who we've now knocked off their schedule by having such a late lunch, we don't need to go late tonight for any particular reason?

MS. CHANCELLOR: Certainly not, Your Honor.

JUDGE FARRAR: Okay. Then let's take an hour lunch break, come back at 2:30. See you then.

(Lunch break was taken.)

JUDGE FARRAR: We're back on the record for the afternoon session. We have some housekeeping matters. Apparently yesterday I didn't say the magic words about State Exhibit 173. I thought I had admitted it but had not. We got to the point of no objections. So that will be admitted, since there were in fact no objections.

Mr. Travieso-Diaz, you said you had a -- oh, no, let's hold that one.

Ms. Nakahara, you've now submitted State Exhibit 174 for identification which has the proper pages replacing the previous 174 for identification?

MS. NAKAHARA: Yes, that's right.

JUDGE FARRAR: Did you want to move to admit that at this point?

MS. NAKAHARA: Yes, please.

JUDGE FARRAR: Any objections?

MR. GAUKLER: No objection, your Honor.

JUDGE FARRAR: Mr. Turk?

MR. TURK: No, your Honor.

JUDGE FARRAR: Then that will be admitted.

(INTERVENOR EXHIBIT-174 ADMITTED.)

JUDGE FARRAR: PFS 86 I think has never been moved.

MR. GAUKLER: We moved to admit it last night. Mr. Soper still had some questions about it. I don't know where the State stands on that, but we're going to get more information to the State on that this afternoon.

JUDGE FARRAR: That's fine. We'll just leave that under advisement. Then we're ready to start with Mr. Trudeau.

MR. TRAVIESO-DIAZ: Good afternoon, gentlemen. I want to start by saying that I'm here to give Mr. Gaukler so much needed relief. Before we start, there is a small housekeeping matter that we'll have to clarify on the record. Last night in reviewing the exhibits to Mr. Trudeau's testimony that we're going to hear in a moment, I realized to my dismay that Exhibit VV --

JUDGE FARRAR: V as in Victor?

MR. TRAVIESO-DIAZ: Two Victors. --

which is the copy of a PFS calculation for the canister transfer building that a copy that was distributed in the books that you have was missing a number of pages. The copies that were mailed out with initial filing was also deficient. So what I have done is I have distributed to the parties and to the Board a substitute exhibit that has the entirety of that calculation. I have consulted with counsel for the State and the Staff, and they have no objections. So if you wouldn't mind replacing Exhibit VV in your books with the new exhibit, that would be good.

JUDGE FARRAR: Have you pointed out to them which were the missing pages?

MR. TRAVIESO-DIAZ: It's obvious, because it stops at page 34, and the calculation is 65 pages long. And I think the State was aware of the problem.

JUDGE FARRAR: Okay.

MR. TRAVIESO-DIAZ: I'd also like to state for the record that we have distributed in advance to the court reporter and the parties and the Board the prefiled testimony of Mr. Trudeau, so I think everybody should have copies of that.

JUDGE FARRAR: Is that any different

from what you sent in a month ago?

MR. TRAVIESO-DIAZ: There are some corrections that Mr. Trudeau will refer to, but they have already been inserted in the copy that you have now.

I think the witness is available to be sworn.

JUDGE FARRAR: Yes. Stand and raise your right hand, sir.

PAUL J. TRUDEAU,

called as a witness, having first been duly sworn, was examined and testified as follows:

DIRECT EXAMINATION

BY MR. TRAVIESO-DIAZ:

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

A. I'm Paul Trudeau.

Q. Do you have before you a copy of a document entitled Testimony of Paul J. Trudeau on Section D of Unified Contention Utah L/QQ dated April 1, 2002?

A. Yes, I do.

Q. Are there any corrections you wish to make to that document?

A. Yes.

JUDGE FARRAR: Let me interrupt you there. I should have asked this on previous occasions. If the corrections are already entered, why do we need to go through them?

MR. TRAVIESO-DIAZ: Well, the only reason we need to go through them is in the event that any party has objections to corrections at this time. I'll be happy to dispense with this question to save time.

JUDGE FARRAR: How many are there?

THE WITNESS: Three.

JUDGE FARRAR: Then go ahead and do them.

MS. CHANCELLOR: I'll state for the record the State has no objection.

MR. TRAVIESO-DIAZ: I will then in that case dispense with reading of the corrections. They are obvious. They are marked in the document.

JUDGE FARRAR: And there's nothing scientifically overwhelmingly significant?

MR. TRAVIESO-DIAZ: I don't believe so, but if there is anything I'm sure we'll hear somehow.

Q. (By Mr. Travieso-Diaz) Did you prepare this testimony and the corrections thereto or under your supervision, or was it done under your supervision?

A. Yes.

Q. Were the corrections that you made in your testimony true and correct, to the best of your knowledge and belief?

A. Yes.

MR. TRAVIESO-DIAZ: Your Honor, I move that the testimony of Paul J. Trudeau be admitted into evidence as if read as his direct testimony in this proceeding.

JUDGE FARRAR: Any objection, Ms. Chancellor?

MS. CHANCELLOR: No objection, your Honor.

JUDGE FARRAR: Mr. Turk?

MR. TURK: No. I point out, Mr. O'Neill will be handling the examination of this witness.

JUDGE FARRAR: Okay.

MR. O'NEILL: No objections.

JUDGE FARRAR: Then the testimony will be bound in the record at this point as if read.

(PREFILED TESTIMONY OF PAUL J. TRUDEAU 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 PAUL J. TRUDEAU ON SECTION D OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESS BACKGROUND

Q1. Please state your full name.

A1. Paul J. Trudeau.

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

A2. I am a Senior Lead Geotechnical Engineer at Stone & Webster, Inc., a Shaw Group Company ("S&W") in Stoughton, Massachusetts.

Q3. Please summarize your educational and professional qualifications.

A3. My professional and educational experience is described in the *curriculum vitae* attached hereto. As indicated there, I have twenty-nine years of experience in geotechnical engineering. My experience includes the performance of subsurface soil investigations; the performance and supervision of the analysis of foundations in support of the design of structures; the performance of laboratory tests of soils including index property tests, consolidation tests, static and dynamic triaxial tests, and other tests; the performance of analyses of the performance of soils and structures under static and dynamic conditions; the development of geotechnical design criteria for other engineering disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical; and the preparation of the geotechnical sections of Preliminary and Final Safety Analyses Reports and Environmental Reports.

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. My particular areas of concentration on the PFSF project are the analysis of soils – settlement, bearing capacity, and stability of foundations – as well as the conduct of soils investigations, laboratory testing of soils to measure static and dynamic properties, and the performance of computer-aided analyses of the behavior of soils and structures under static and dynamic loading conditions.

Q5. What is the purpose of your testimony?

A5. The purpose of my testimony is to respond to allegations raised by the State of Utah in Section D of Unified Contention Utah L/QQ with respect to the seismic analysis of the storage pads, casks, and their foundation soils and the seismic analysis of the Canister Transfer Building and its foundation. I am also filing separate testimony on the allegations raised by the State in Section C of Unified Contention Utah L/QQ. That testimony addresses: (1) the characterization of subsurface soils at the PFSF site through subsurface investigations, sampling and analyses; (2) the stress/strain behavior of the soils under design basis earthquake conditions; and (3) the use of soil cement and cement-treated soil to enhance the seismic behavior of the soils beneath and adjacent to the foundations of the safety-related structures at the PFSF.

II. SEISMIC STABILITY ANALYSES PERFORMED BY S&W FOR THE PFSF

Q6. What are the main stability analyses that you have conducted regarding the performance of safety-related structures at the PFSF during seismic events?

A6. Part of my duties as lead geotechnical engineer is to perform, or direct the performance of, analyses of the response of the PFSF structures to the forces imparted by postulated seismic events. In particular, I was responsible for the preparation of Stone & Webster Calculation Nos. 05996.02-G(B)-04, Rev. 9, *Stability Analyses of Cask Storage Pads* (July 26, 2001) ("Cask Storage Pad Stability Calc. Rev. 9"), and 05996.02-G(B)-13, Rev. 6, *Stability Analyses of Canister Transfer Building* (July 26, 2001) ("CTB Stability Calc. Rev. 6"). Copies of relevant excerpts from these two calculations are included as PFS Exhibits UU and VV.

Q7. Would you please describe how seismic stability analyses such as those are conducted?

A7. In the seismic stability analyses, we seek to evaluate three potential failure modes for the structures: sliding stability, overturning stability, and bearing capacity stability. Sliding failure occurs if the structure moves horizontally, parallel to the ground. Overturning failure occurs if the structure rotates as a rigid body about a horizontal axis. Bearing capacity failure takes place if the soils beneath the structure become overloaded in the vertical direction, leading to excessive settlement or rotation of the structure's foundation.

Q8. You use the term failure. Is the intent of the analyses to determine whether the structure in question will actually undergo sliding, overturning or bearing capacity failure?

A8. No. The intent of the analyses is to establish what margin or "factor of safety" ("FS") is provided by the design of the structure's foundations against each of the failure modes. It is typical in the industry to use FS = 1.1 as the desired safety factor against each of the three failure modes that I mentioned for load combinations that include seismic loads from the design basis earthquake. For example, Section 3.8.5 of NUREG-0800, the Standard Review Plan ("SRP") for Nuclear Power Plants, indicates that the factors of safety against overturning and sliding are acceptable if they exceed 1.1 for load combinations that include seismic loads due to the design basis earthquake.

Q9. If, for example, a factor of safety of 1.1 against sliding is not demonstrated, does that mean that the structure will actually slide in a seismic event?

A9. No. It is only when the results of the analysis predict a factor of safety of less than 1.0 that the failure mode in question might occur. Even then, our analyses include additional conservatism in various parameters, such that even if the calculated factor of safety was less than 1, the structures likely would not slide during the seismic event. In addition, because of the cyclic nature of the seismic loading, each of the peak accelerations we use to estimate the dynamic loads from the earthquake exists only for one, very brief moment in time – typically less than 0.005 seconds – and then the earthquake accelerations reverse direction. Therefore, even if the forces due to the peak acceleration of the earthquake exceeded the resisting forces, a fraction of a second later the accelerations would decrease, and the corresponding inertial forces would decrease as well, such that the structure would not experience significant horizontal displacement. In addition, even for an earthquake as large as the design basis earthquake for the PFSF, there will be only one point in time where the acceleration will equal the maximum value – at every other point in time, the accelerations will be much less than the peak value – yet the analyses assume that the forces due to these peak accelerations act continuously for purposes of computing the factor of safety.

Q10. Do you analyze, for each type of failure mode, various combinations of earthquake loadings?

A10. Yes. In addition to a reference "static" case ("Case I" in PFS Exhibits UU and VV), in which only the weight of the structure and its effect on the soils beneath the foundation are determined, we run, for each seismic failure mode, three families of cases: one (labeled "Case II") for static loads plus dynamic horizontal forces due the earthquake; another (labeled "Case IIIA," "Case IIIB," and "Case IIIC") for static plus various combinations of horizontal and vertical uplift forces due to the earthquake; and another family (labeled "Case IVA," "Case IVB," and "Case IVC") for static plus various combinations of horizontal and vertical compression forces due to the earthquake.

Q11. Do you also perform variations of each case in which some of the assumptions or parameters are varied?

A11. Yes. In addition to a "base case" that reflects the design intent with respect to the soils and foundations, we also perform hypothetical, "what if" analyses, in which other behavioral modes are explored.

Q12. Does performance of those hypothetical "what if" analyses mean that they are regarded as constituting credible scenarios for the behavior of soils and structures in an earthquake?

A12. No. The hypothetical analyses may be performed for a variety of reasons, such as, for example, determining what additional margins may be present in the design for which credit is not taken. However, performance of a hypothetical analysis does not necessarily mean that it is regarded as credible.

Q13. What was the "base case" you analyzed with respect to the sliding stability of the cask storage pads?

A13. That case is described and analyzed on pages 15 through 28 of Cask Storage Pad Stability Calc. 6(B)-04, Rev. 9 (PFS Exh. UU). It is based on engaging the shear strength of the soils beneath the pads to provide resistance against sliding forces. To ensure that the full shear strength of the soils is available to provide resistance against sliding, an "engineered mechanism" will be provided through the replacement of the top layer (1 to 2 feet) of soil below the cask storage pads with a cement-treated soil mixture having a minimum compressive strength of 40 psi, which provides a shear strength that is nearly twice as strong as the underlying clayey soils. The details of the design, testing, and construction of this cement-treated soil layer are described in my testimony on Section C of Unified Contention Utah L/QQ.

Q14. What conservative assumptions are made in the base case?

A14. In addition to replacing the soils within one to two feet beneath the pads with cement-treated soil that provides nearly twice the shear resistance as the *in situ* clayey soils beneath the pads, the design intent is also to replace the top 3 ft. of soil below grade in the areas around the cask storage pads with a 2 ft.-4 in. thick layer of soil cement with a minimum compressive strength of 250 psi, topped with 8 in. of compacted aggregate. The purpose of this soil cement placed adjacent to the pads is to provide a firm foundation for supporting the cask transporter that will move storage casks onto the pads. This soil cement installation will provide significant, additional, resistance against sliding of the pads in an earthquake; however, the base case conservatively does not take credit for the strength of the soil cement installed around the pads to resist these sliding forces. Thus, the base case analysis conservatively ignores the cohesive strength of the soil cement in calculating the dynamic active earth pressures that must be resisted to preclude sliding. In addition, it ignores the passive resistance provided by the soil cement adjacent to the pad, and it ignores the shearing resistance available between the sides of the pad parallel to the direction of sliding and the soil cement adjacent to the pads. The analysis also conservatively uses shear strengths of the clayey soils based on static strengths measured in direct shear tests, despite the well-known phenomenon that such clayey soils exhibit increases in shear strength of as much as 100% when subjected to rapid loadings, such as those imparted by the design basis earthquake.

Q15. Are similarly conservative assumptions also made in the base cases for the other potential failure mechanisms?

A15. Yes. Similarly conservative assumptions (such as the use of static shear strength for the soils) are also made in the bearing capacity and overturning failure cases.

Q16. Have you sought to estimate how much the factors of safety would increase in the various stability calculations if, for example, more realistic values of the shear strength of the soils were used?

A16. Yes. I performed several simple calculations to estimate how much the factors of safety against failure would increase if the shear strength of the clayey soils was increased 50% from the strengths obtained in the static strength tests to account for the well known phenomenon that the dynamic strength of clayey soils under rapid rates of loading comparable to the cycling applicable for earthquakes is 50% to 100% greater than the strength measured in static shear tests. The results are as follows:

For the pads (bearing capacity failure):

As shown in SAR Table 2.6-7 (also p. 107 of Calc. G(B)-04-9, PFS Exh. UU), of the cases that combine the earthquake components in accordance with the 40-40-100 rule recommended by ASCE 4-86 (p. 12 of G(B)-04-9, PFS Exh. UU), Load Case IVB had the lowest FS against a bearing capacity failure based on inertial forces (p. 69 of Calc. G(B)-04-9, PFS Exh. UU: FS = 2.1 using the static shear strength, c = 2,200 psf). Increasing the soil shear strength by 50% to 3,300 psf to account for the dynamic strength of this clayey soil, increases this FS to 3.63. Conversely, the earthquake accelerations would have to be increased by a factor of 1.74 (i.e., to a horizontal acceleration of 1.24g and a vertical acceleration of 1.21g) to reduce the FS to 1.1, and by a factor of 1.79 (i.e., a horizontal acceleration of 1.27g and a vertical acceleration of 1.24g) to reduce the FS to 1.0.

For the pads (sliding failure): (we considered for demonstrating how the safety margin)

increased is

For the sliding stability of the pads, the critical case will be for 10 pads sliding in the north-south direction. Pages 32 and 33 of Calc. G(B)-04-9 illustrate that using the static shear strength of the clay soils, the factor of safety against sliding of an entire column of pads in the north-south direction is 1.51. If we increase the clay soil strength by 50% to account for the normal increase of strength for clayey soils to dynamic loadings such as these, the factor of safety for this case increases to 2.2. Conversely, the pad + soil cement + cement-treated soil inertial forces and the maximum cask dynamic forces from the 2,000-yr return period earthquake would have to be more than doubled (i.e., the horizontal earthquake acceleration would have to be increased to 1.44g) for this case to obtain a factor of safety against sliding equal to 1.1.

For the CTB (bearing capacity failure):

As shown in SAR Table 2.6-10 (also p. 48 of Calc. G(B)-13-6, PFS Exh. VV), of the cases that combine the earthquake components in accordance with the 40-40-100 rule recommended by ASCE 4-86, Load Case IVB had the lowest FS against a bearing capacity failure based on inertial forces (p. 41 of Calc. G(B)-13-6, PFS Exh. VV: FS = 6.25, with a shear strength c = 3,180 psf.) (The soil shear strength c = 3,180 psf was adjusted from the c = 2,200 psf for these soils based on the CPT results, as described on p. 9 of the calculation.) Increasing the soil shear strength by 50% (to c = 4,770 psf) to account for the dynamic strength of these clayey soils increases the FS to 10.1. Conversely, the earthquake accelerations would have to be increased by a factor of 4.34 to reduce the FS to 1.1, and by a factor of 4.39 to reduce the FS to 1.0.

For the CTB (sliding failure):

As shown in p. 23 of G(B)-13-6, PFS Exh. VV, c = 1.36 ksf is the applicable static residual shear strength of the soil for the CTB sliding case that used the full passive resistance of the soil cement around the building. The factor of safety against sliding for that shear strength value is 1.26. Increasing the shear strength of the soil by 50% (c = 2.04 ksf) to account for the dynamic strength of the clayey soils, increases the FS against sliding to 1.61. Conversely, the earthquake accelerations would have to be increased by a factor of 1.46 to reduce the FS to 1.1, and by a factor of 1.61 to reduce the FS to 1.0 for c = 2.04 ksf.

Q17. Are there other conservatisms incorporated into the design practices and the codes and standards used in performing this and the other stability analyses?

A17. There are several major elements of conservatism in nuclear industry design practices and applicable codes and standards that are reflected in the stability analyses conducted by PFS. These conservatisms include those in the utilization of "lower bound" (as opposed to best estimate or mean) values of the soil properties, in analysis assumptions, and the definition of "failure". Such conservatisms form part of the intentional and recognized safety margin inherent in the NRC seismic evaluation process discussed in the testimony of Dr. Allin Cornell being filed simultaneously with this testimony. These conservatisms imply that the foundations will have much greater factors of safety against failure than the analyses predict, and would not actually fail until the earthquake ground motions become far larger than the design basis motions.

Q18. What were the results of the base case analyses?

A18. The analyses show that the minimum factor of safety against sliding of the storage pads in the event of a design basis earthquake is 1.27 (versus a target of 1.1), ignoring, as indicated above, the passive resistance available due to the soil cement adjacent to the pad. This value is based on the dynamic loads acting in the east-west direction. Those acting in the north-south direction are somewhat lower, resulting in a factor of safety against sliding of a single pad in the north-south direction of 1.36. This means that the storage pads will not slide in the event of a design basis earthquake. It should be noted that the calculated factor of safety against sliding between the base of the concrete pad and the underlying cement treated soil layer is 1.98, meaning that the limiting factor in the resistance to sliding is the bond between the cement-treated soil and the native soil underneath, not the bond between the cement-treated soil and the concrete pad above it.

Q19. What other sliding cases did you analyze for the storage pads?

A19. We also considered a case in which we take credit for the passive resistance provided by the 2 ft.-4 in. layer of soil cement to be placed around the pads, in order to demonstrate the beneficial effect of placing this soil cement adjacent to the pads. Our calculations for that case, which include only the forces acting on the pad, not those on the underlying cement-treated soil, are presented on pages 29 and 30 of Cask Storage Pad Stability Calc. Rev. 9 (PFS Exh. UU), show that the minimum factor of safety against sliding in the north-south direction without including the passive resistance of the soil cement is 1.52, and that this factor of safety increases to 2.35 when the passive resistance due to the soil cement adjacent to the pad is included. It also demonstrates that the factor of safety against sliding in the east-west direction is increased to 3.3 when the passive resistance of the soil cement is included; thus, the critical direction for sliding of the pads is the north-south direction.

The sliding stability of an entire column of 10 pads in the north-south direction also was considered. In this case, the resistance to sliding of the entire column (running N-S) of pads exceeds that of each individual pad because there is more area available to engage more shearing resistance from the underlying soils than just the area directly beneath the individual pads. The extra area is provided by the 5-ft long x 30-ft wide plug of soil cement that exists between each of the pads in the north-south direction. This analysis assumes that the soil cement east and west of the long column of pads provides no resistance to sliding, conservatively assuming that the soil cement somehow shears along a vertical plane at the eastern and western sides of the column of 10 pads running north-south. The resulting factor of safety increases from 1.36 for an individual pad in the north-south direction to 1.50 for an entire column of 10 pads.

We also considered a hypothetical sliding stability case, presented on pages 36 to 45 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, in which the cohesive portion of the strength of the clayey soils along the interface with the cement-treated soils underneath the pads is completely ignored. In this hypothetical case, resistance to sliding is provided only by the frictional portion of the shear strength of the clayey soils beneath the cement-treated soil layer underneath the pads, and it is based on an obviously conservative value of the friction angle for the underlying soils. Not surprisingly, the pads are shown to slide in an earthquake under these assumptions, whether a single pad or a row of pads is considered.

This analysis also includes an estimation of the horizontal displacements that will be experienced by a row of 20 pads under the assumptions described above. The estimation is based on a method described in the technical literature for assessing the displacement of dams and embankments during earthquakes. This analysis yields horizontal displacements of the pads on the order of 2 to 6 inches. Again, these displacements apply only to a hypothetical case based on extremely conservative assumptions.

Another hypothetical analysis (pages 46-51 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU) was conducted in which it was assumed that the storage pads rest directly on cohesionless soils, instead of on cement-treated soil and the clays that exist at the PFSF site. For that case, based on a conservative, lower-bound friction angle of 30 degrees for the cohesionless soils that were postulated to exist directly at the base of the pads, horizontal displacements of the pads on the order of 1.9 to 2.2 inches are predicted.

Q20. What weight should be given to the various hypothetical cases you just described?

A20. These cases are important in that they illustrate various conditions that bound the characteristics of the PFSF site soils and their performance in a design basis earthquake. However, the case that represents the design basis of the pads, which in itself incorporates a number of conservative assumptions, demonstrates that the design of the foundations of the cask storage pads provides a more than adequate factor of safety against sliding of the pads and the casks they support in an earthquake.

Q21. What analyses did you perform of the bearing capacity of the cask storage pads?

A21. The bearing capacity analyses, which are presented on pages 52-98 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, consider both static load cases and two different sets of dynamic loads. One set of dynamic loads was that resulting from the inertial forces applicable to the peak ground accelerations from the design basis ground motion. The other set of dynamic loads was based on the maximum dynamic cask driving forces obtained by the designer of the pads for cases in which the pad supports 2, 4, and 8 casks.

Q22. What results did you obtain?

A22. For the case of dynamic loads based on inertial forces from the design basis ground motion, the lowest factor of safety against bearing capacity failure was

1.17 (Case II, p. 59 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU), and was obtained under the very conservative assumption that 100% of the earthquake loads act in both horizontal directions at the same time. More realistic cases, in which the loads were distributed among the three dimensions in accordance with procedures set forth in industry standards, yielded factors of safety against bearing capacity failure exceeded 2 (Case IVB, p. 69 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU).

In the second set of analyses, the dynamic loads were based on those developed by the pad designer for varying numbers of casks loaded onto the pads. Those analyses were based on the conservative assumption that the maximum dynamic forces will all occur at the same time at each node in the model used to represent the cask storage pads, which, therefore, represents an upper bound of the dynamic forces that can be applied to the pads. A minimum factor of safety against bearing capacity failure of 1.6 (Case IVB, p. 97 of Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU) was obtained, applying to the case in which 8 casks are loaded onto the pad.

Q23. What analyses did you perform of the overturning stability of the cask storage pads?

A23. Overturning analyses were based on the dynamic loadings from the design basis ground motion. The analyses showed that the factor of safety of the storage pads against overturning is 5.6, well in excess of recommended margins.

Q24. Would you please summarize the results of the stability analyses of the storage pads under design basis earthquake loadings?

A24. The analyses that we performed of the sliding stability, bearing capacity, and overturning stability of the foundations of the storage pads show that significant margins are available for those foundations in the event of a design basis earthquake. These factors of safety, which incorporate a number of conservative assumptions, assure that the pads and the storage casks will remain stable under the loads imparted by the design basis earthquake. Moreover, the results of the base cases plus the conservatisms built into the stability analyses (as demonstrated just by increasing the shear strength of the soils to more realistic values) make it safe to predict that the storage pads will not experience failure under the loadings from an earthquake far more severe than the design basis earthquake.

III. RESPONSE TO STATE CLAIMS IN SECTION D RELATING TO SEISMIC STABILITY ANALYSES OF STORAGE PADS AND CASKS

Q25. In Section D of Unified Contention Utah L/QQ, the State alleges several deficiencies in the PFS seismic stability analyses for the storage cask pads and the CTB and its foundation. Are you familiar with those allegations?

A25. Yes.

Q26. What is your general response to the State's allegations?

A26. The claims raised by the State are either incorrect or seek to find fault with some of the hypothetical cases that are included in the seismic stability analyses but which do not represent the design basis case; therefore the claims are irrelevant. They are also inconsequential in that the deficiencies alleged to exist, even if present, would not materially affect the validity of the analyses.

Q27. In Subsection D.1.b(i) of Unified Contention Utah L/QQ, the State asserts that the Applicant has not demonstrated adequate factors of safety against overturning and sliding stability of the storage pads and their foundation system for the design basis earthquake because the Applicant's calculations incorrectly assume that the pads will behave rigidly during the design basis earthquake. The assumption of rigidity is alleged to lead to significant underestimation of the dynamic loading atop the pads, especially in the vertical direction. Is this claim correct?

A27. No. As discussed in the testimony of Dr. Wen-Shou Tseng filed simultaneously herewith, the storage cask pad deflections under design basis earthquake loads are very small and the pads can be considered as essentially rigid for analytical purposes (although Dr. Tseng's organization, International Civil Engineering Consultants, Inc. or ICEC, conservatively treated the pads as flexible for purposes of their structural design). Because the pads are essentially rigid, the premise to the State's assertion that our stability analysis are faulty is incorrect. In addition, it can be demonstrated that the dynamic loads have not been underestimated in these analyses.

Q28. State witnesses have testified that the estimate you used of the seismic loadings on the pads in the horizontal and vertical direction use the peak ground acceleration of the design basis motion, which underestimates the accelerations to which the pads and storage casks will be subjected. Is there a significant difference between the peak ground acceleration and the accelerations to which the pads will be subjected?

A28. No. The difference, if any, is not significant, because the appropriate response spectrum curve to be used for determining these acceleration values should be based on the damping applicable for the pad + casks + soil system. This damping should include both radiation damping and material damping; however, the bulk of the energy is dissipated due to radiation damping in this case. The radiation damping is calculated based on a relatively simple formulation, and for the best-estimate soil properties, it can be shown to be approximately 50% for vertical vibration of the pad + casks + soil system. This number varies only slightly for the lower-bound (52%) and upper-bound (48%) soil properties. For such high degrees of damping, the amplification that would occur for the pad + casks + soil system would be much lower than would apply based on the response spectrum plot for 5% damping referred to by the State [Trudeau/Chang Deposition 11/15/00 at 172:22] as a demonstration that a huge amplified response is applicable for the pad foundations. The fundamental frequency for this case is approximately 6.21 Hz, corresponding to a fundamental period of 0.16 sec. The response spectrum for the vertical earthquake time history for 50% damping indicates that the maximum acceleration should be 0.757g, which is a slight amplification over the 0.695g used to calculate the inertial forces applicable for the pad + soil cement in these analyses. This is not the smoothed design response spectrum, however. As shown in Table 1 in Calc. 05996.02-G(PO18)-3-1 for a period of 0.16 sec, the response spectrum of the PFS vertical time history overestimates the design response spectrum by approximately 13%. If this adjustment is taken into consideration, the applicable vertical acceleration for 50% damping would be 0.757g ÷ 1.13, or 0.67g. This value is less than the value of 0.695g that was used to calculate the inertial forces applicable for the pad + soil cement in these analyses.

At any rate, the response spectrum technique is a very conservative way of arriving at the dynamic loads applicable for the pads and underlying cement-treated soils in this case. An independent verification of the dynamic loads used in the sliding stability analyses presented in the Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, can be obtained from a review of the time histories of the forces used by Holtec in its soil-structure interaction analysis of the cask storage pad. Time histories of the forces at the base of the pad generated by Holtec (without the soil mass attached to the pad) show that the peak horizontal force at the base of the pads during the entire earthquake record was 3,310 kips, acting in the east-west direction at 4.675 seconds into the time history. (Holtec's time history of forces from their SSI analysis of the casks + pad + virtual soil mass underlying the pad show that these peak forces are less when the virtual soil mass attached to the pad is included.) The peak horizontal force acting in the north-south direction was less than this, equaling only 2,540 kips at 5.445 seconds into the time history. Therefore, the critical direction for sliding is in the east-west direction. The factor of safety against sliding of the pad for this worst-case loading from Holtec's SSI analysis is calculated as follows for the 30 ft x 67 ft pad:

$$FS_{\text{Sliding}} = \frac{\Sigma\,\text{Resisting Forces}}{\Sigma\text{DrivingForces}}$$

$$FS_{\text{Sliding E-W w/oPassive}} = \frac{2.1\,\text{ksf} \times 30\,\text{ft} \times 67\,\text{ft}}{3{,}310\,\text{k} + 65.3\,\text{k}} = 1.25$$

(In this equation, 2.1 ksf is the shear strength of the soil, 3,310 kips is the earthquake's peak horizontal sliding force, and 65.3 kips is the force due to dynamic active earth pressure acting on the pad in the same direction as the earthquake's acceleration.)

The minimum factor of safety against sliding of the pad at any point in time resulting from the time history of forces from Holtec's SSI analysis, 1.25, is nearly the same as the minimum factor of safety against sliding of 1.27 calculated on p. 23 of the Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, for the design basis case. Therefore, the use of the peak horizontal ground accelerations in determining the sliding forces of the pad and underlying cement-treated soil does not significantly underestimate the dynamic loads acting on the storage cask pad foundation, if it underestimates them at all.

Further, the minimum factor of safety against sliding applies to only a single point in time in the entire time history. At every other point in the time history, the factors of safety against sliding exceed this value. Plotting the factor of safety against sliding vs time based on the time history of forces from Holtec's SSI analysis without the virtual soil mass included, demonstrates that the average factor of safety against sliding is approximately 10 throughout the duration of the earthquake, greatly exceeding this minimum value, as shown in PFS Exh. WW.

Q29. By how much would you expect the seismic loadings would change if you used the natural frequency of the pads in the analyses?

A29. The time history of forces, described in my previous answer, which were developed by Holtec in their SSI analysis of the pad + casks, provides a more rigorous and correct determination of these dynamic forces than you would obtain from the use of the response spectrum at the appropriate damping value. The calculation of the factor of safety against sliding based on this time history of forces at the base of the pad + casks demonstrates that there is only a very slight reduction in the minimum factor of safety against sliding when these loads are used, from 1.27 to 1.25, compared to the use of inertial forces of the pad and cement-treated soil based on the peak horizontal ground accelerations.

Q30. Subsection D.1.c(i) of Unified Contention L/QQ asserts that the Applicant has failed to provide a realistic evaluation of the foundation pad motion with cement-treated soil under and around the pads in relation to motion of the casks sliding on the pads in that Applicant's evaluation ignores the effect of soil-cement around the pads and the unsymmetrical loading that the soil-cement would impart on the pads once the pads undergo sliding motion. State witnesses have asserted that one of the consequences of this deficiency is that the Newmark sliding block analysis for the storage casks did not consider the potential for unsymmetrical sliding and underestimated the displacement of the storage pads. How do you respond?

A30. In considering this hypothetical scenario, it is important to understand that the pads have a greater resistance to sliding along their base than does the soil cement. As indicated on p. 39 of the Cask Storage Pad Stability Calc. Rev. 9, PFS Exh. UU, "the soil cement cannot even resist sliding of itself during the earthquake **if only the frictional portion of the strength is assumed to be available** along its base." Thus, if the pads slide, so will the soil cement – they will move in concert – and the pads will not be impacting the soil cement. In this situation, it is proper to ignore the presence of the soil cement in estimating displacements of the pads. It is unreasonable for the State's witness to assume that the soil cement will have more resistance to sliding than the pads.

The Newmark sliding block analysis is included in the pad stability calculation for the hypothetical case where it is assumed that the shear strength available to resist sliding at the interface between the cement-treated soil and the in situ clayey soils is based only on the frictional portion of the clay strength, completely ignoring the cohesive strength of the clay. For this obviously conservative scenario, the factor of safety against sliding was less than 1, indicating that the pads might be expected to slide due to the earthquake. An estimation of the amount of sliding that might occur was made based on the method proposed by Newmark[1] for estimating displacements of dams and embankments during earthquakes.

Newmark defines "N·W" as the steady force applied at the center of gravity of the sliding mass in the direction which the force can have its lowest value to just overcome the stabilizing forces and keep the mass moving. If the surface is horizontal, then it is just as easy for the block to slide to the left as it is to the right. In this case there is symmetrical resistance to sliding, and this is the case

[1] Newmark, N. M., 1965, "Effects of Earthquakes on Dams and Embankments," *Fifth Rankine Lecture*, Geotechnique, Institution of Civil Engineers, London, 15(2), pp139-60.

that applies for the pads at the PFSF, because the site is essentially horizontal. For a block resting on an inclined plane, such as applies to a model of the slope of an embankment or a dam, the situation is different. It is much easier for the block on the slope to move downhill than it is to move uphill, because gravity helps in moving the block downhill. The force required to move the block uphill must overcome both the resistance to sliding of at the base of the block on the slope and gravity. In this case, the resistance to sliding is considered to be unsymmetrical, because it is more difficult to move the block back up the hill than to move it down the hill.

The soil cement at one side of the cask storage pad provides the same resistance to sliding as at the other; therefore, this clearly is a case of symmetrical sliding as defined by Newmark.

It is also worth remembering that this is not the design basis case for the pads. PFS's design basis for the pads relies on the shear strength available at the interfaces between the cask storage pad and the underlying cement-treated soil and between the cement-treated soil and the underlying clayey soils, and on the commitment to demonstrate by testing that this shear strength can be achieved and that it is achieved by construction. The design basis of the pads provides a conservatively calculated factor of safety against sliding that exceeds 1.1; therefore, the pads do not slide. Since the pads do not slide, the question is moot.

Q31. In paragraph D.1.g of Unified Contention L/QQ, the State asserts that PFS has failed to analyze for the potential of pad-to-pad interaction in its sliding analyses for pads spaced approximately five feet apart in the longitudinal direction. What is your understanding of the bases for the State's claim?

A31. My understanding is that the State is claiming that the stability analysis of the storage pads failed to consider potential of pad-to-pad interaction, but assumed all pads in a quadrant move together as an integrated foundation. The State believes this is an erroneous assumption.

Q32. Is it?

A32. No. PFS's design basis for the pads provides a factor of safety against sliding that exceeds 1.1; therefore, the pads do not slide, but rather, they will move with the underlying soil during the earthquake. The only possible interaction between the pads is dependent on the shear deformation of the pad above its base and the soil cement plug between the pads. However, the concrete pads and the soil cement plug between the pads are both very rigid with respect to the seismic shear loading. For example, the SHAKE analyses included in Calc. 05996.02-G(PO18)-2, Rev. 1 include soil cement at the top of the profile. The results for the lower-bound, fault-parallel case indicate that the effective shear strains in the clayey soil layer underlying the soil cement averaged 0.13%. This case produced the highest shear strains in this clay layer of all of the various soil property and earthquake component cases analyzed. However, even for this case, the effective shear strains in the soil cement were only 0.0034%, which is insignificant when considering movements required to effect pad-to-pad interactions. Therefore, shear distortions within the soil cement and concrete pads due to the upward propagation of seismic waves should be very small. It is, therefore, anticipated that the pad and soil cement plug between the pads will deflect in phase with the underlying soils, meaning that the interaction between the pads will be insignificant.

IV. RESPONSE TO STATE CLAIMS IN SECTION D RELATING TO SEISMIC STABILITY ANALYSES OF CANISTER TRANSFER BUILDING AND CASKS

Q33. In paragraph D.2.c of Unified Contention L/QQ, the State asserts that the Applicant's calculations are deficient because they ignore the out-of-phase motion of the CTB and the cement-treated soil cap, which potentially can lead to the development of cracking and separation of the cap around the building perimeter. How do you respond to the State's claim?

A33. The State claims that various mechanisms can lead to the formation of cracks in the soil cement that surrounds the building: shrinking and curing of the soil cement during the placement process, differential settlement between the building foundation and the surrounding soil cement, bending stresses in an earthquake, motion between the building foundation and the surrounding soil cement. I disagree that earthquake bending stresses will lead to the formation of new cracks, or that differential settlement between the building foundation and the soil cement layer will lead to crack formation. At any rate, as I discussed before, these are all thin, vertical, random cracks that do not affect the ability of the soil cement to provide the passive resistance to sliding relied upon in the design.

Q34. Why will not new cracks be formed due to earthquake bending stresses?

A34. The effect of bending stresses on the soil cement surrounding the CTB mat will be to alternately open and close the tops and bottoms of any shrinkage cracks that may have occurred in the soil cement in the area, not to form new cracks.

Q35. And why will there be no new cracks due to differential settlement?

A35. Because, as the CTB foundation mat is loaded, the soils within the profile adjacent to the mat also will experience increases in stresses, as the loading gets distributed over a wider area deeper in the soil profile. This stress distribution results in settlement of the soil cement areas adjacent to the mat which will approximate those at the edge of the mat, so that there will not be an abrupt differential settlement noted at the joint between the edge of the mat and the soil cement. These settlements will gradually decrease with increasing distance from the edge of the mat. The resulting settlement profile will be dish-shaped, concave downward, extending some distance away from the edge of the mat, so no cracks will form due to differential settlement. The concave downward shape of the settlement profile will result in closing of the lower portion of the nearly vertical shrinkage cracks. This lower portion of the soil-cement profile provides a greater percentage of the resistance due to increased passive pressure at depth; therefore, this settlement is beneficial in improving the ability of the soil cement to provide passive resistance.

Q36. Why would there be no effect on the passive resistance of soil cement around the CTB if new cracks are formed or existing cracks reopen?

A36. Because the passive resistance of soil cement is not diminished by the presence of a crack. The effect of cracks opening as seismic waves pass through the soil-cement layer is, at most, to cause the building to displace a small distance to close each crack, and then the full passive resistance of the soil cement to sliding is restored.

Q37. Does that mean that the CTB might actually slide some distance?

A37. Theoretically, the CTB might move a small distance – measured in fractions of an inch to inches – in order that the cracks in the soil cement be closed and full passive resistance be restored. Were that to happen, however, there would be no safety-related consequences, because there are no connections between the CTB and any other safety-related systems, structures, or components that would be adversely impacted by such horizontal movement.

Q38. Have concerns been expressed by the State regarding potential failure mechanisms for the CTB other than sliding?

A38. Yes. State witnesses have raised concerns about potential overturning of the CTB in a seismic event. However, my understanding is that the concerns refer to some of the assumptions made in the overturning calculations that are part of the stability analysis of the CTB, not with the calculation results, which show that there is a significant factor of safety in the CTB design against overturning (FS_{OT} = 1.95, p. 15 of CTB Stability Calc. Rev. 6, PFS Exh. VV). While I disagree with the concerns, I agree with the conclusions expressed by the State's witnesses that overturning of the CTB during a design basis earthquake is not a realistic concern.

Q39. Is bearing capacity failure of the CTB a concern?

A39. No. To my knowledge, neither the State nor any of its witnesses has raised bearing capacity as a failure mechanism of concern for the CTB. This is not surprising, since our calculations show that for all cases analyzed, the factor of safety against bearing capacity failure of the CTB is 5.5 (Load Case II, SAR Table 2.6-10 and p. 48 of CTB Stability Calc. Rev. 6, PFS Exh. VV) or greater. Thus, bearing capacity failure of the CTB is not a credible scenario.

Q40. Would you please summarize the results of the stability analyses of the CTB under design basis earthquake loadings?

A40. The analyses that we performed of the sliding stability, bearing capacity, and overturning stability of the CTB show that adequate factors of safety are available for those foundations in the event of a design basis earthquake. These factors of safety, which incorporate a number of conservative assumptions, assure that the CTB will not be subject to failure under the loads imparted by the design basis earthquake. Moreover, the results of the base cases plus the demonstrated conservatisms built into the stability analyses (as demonstrated just by increasing the shear strength of the soils to more realistic values) make it safe to predict that the CTB will not experience failure under the loadings from an earthquake significantly more severe than the design basis earthquake.

Q41. Does that conclude your testimony?

A41. Yes, it does.