PDR

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

APR 25 1985

Docket Nos.: 50-445
and 50-446

MEMORANDUM FOR: Chairman Palladino
Commissioner Roberts
Commissioner Asselstine
Commissioner Bernthal
Commissioner Zech

FROM: Vincent S. Noonan, Director
for Comanche Peak Project
Division of Licensing
Office of Nuclear Reactor Regulation

SUBJECT: BOARD NOTIFICATION - SUMMARY OF MEETING BETWEEN CASE, TEXAS UTILITIES AND THE NRC STAFF RELATING TO THE CONCERNS OF MESSRS. WALSH AND DOYLE REGARDING THE COMANCHE PEAK PLANT (BOARD NOTIFICATION NO. 85-039)

This Notification is being provided to the Commission in accordance with the revised Commission's notification policy of July 6, 1984, to inform the Commission on all issues on the cases before the Commission.

On Saturday, March 23, 1985, a meeting was held between CASE (represented by Mrs. Juanita Ellis, Mr. Mark Walsh and Mr. Jack Doyle), the Texas Utilities and the NRC staff. The meeting was held at the Ramada Inn, Arlington, Texas. The purpose of the meeting was to conduct a feedback discussion with Mr. Walsh and Mr. Doyle regarding their concerns about the Comanche Peak Plant, and to also request comments and clarification from them and to allow the applicants to comment and ask questions. The meeting was noticed and transcribed. A copy of the Summary of Meeting with enclosed transcript is provided for your information.

The parties to the proceeding are being notified by copy of this memorandum.

Vincent S. Noonan, Director
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Division of Licensing
Office of Nuclear Reactor Regulation

Enclosure: As stated

cc: See next page

8505010146

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COMANCHE PEAK

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MEETING SUMMARY DISTRIBUTION

APR 2 5 1985

Docket File
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NSIC
PRC System
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UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

APR 25 1985

Docket Nos.: 50-445
and 50-446

APPLICANT: Texas Utilities Generating Company (TUGCO)

FACILITY: Comanche Peak Steam Electric Station, Units 1 and 2

SUBJECT: SUMMARY OF MEETING BETWEEN CASE, THE COMANCHE PEAK RESPONSE TEAM AND THE NRC STAFF RELATING TO THE CONCERNS OF MESSRS. WALSH AND DOYLE

On Saturday, March 23, 1985, a meeting was held between CASE (represented by Mrs. Juanita Ellis, Mr. Mark Walsh and Mr. Jack Doyle), the Texas Utilities and the NRC staff. The meeting was held at the Ramada Inn, 700 East Lamar, Highway 157, Arlington, Texas. The purpose of the meeting was to conduct a feedback discussion with Mr. Walsh and Mr. Doyle regarding their concerns about the Comanche Peak Plant, to request comments and any clarification from them and to allow the applicants to comment and ask questions. The meeting was structured to have each NRC team member identify the key issues which he has been reviewing and discuss the status of the NRC effort. The meeting closed with a discussion directed at clarification of the issues and concerns of Mr. Walsh and Mr. Doyle.

A copy of the meeting notice is enclosed (Enclosure 1). The meeting was transcribed and the transcript is enclosed (Enclosure 2). A meeting attendance list is also enclosed (Enclosure 3).

S. B. Burwell, Project Manager
Licensing Branch No. 1
Division of Licensing

Enclosures: As stated

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

ENCLOSURE 1

MAR 19 1985

Docket Nos.: 50-445
and 50-446

MEMORANDUM FOR: B. J. Youngblood, Chief
Licensing Branch No. 1
Division of Licensing

FROM: S. B. Burwell, Project Manager
Licensing Branch No. 1
Division of Licensing

SUBJECT: FORTHCOMING MEETING BETWEEN CASE, THE COMANCHE PEAK RESPONSE TEAM AND THE NRC STAFF RELATING TO THE CONCERNS OF MESSRS. WALSH AND DOYLE

DATE & TIME: Saturday, March 23, 1985
12:00 Noon - 8:00 PM

LOCATION: Ramada Inn, Texan Room
700 Lamar Blvd.
Arlington, Texas 76012

PURPOSE: To discuss the concerns of Messrs. Walsh and Doyle as they relate to ongoing evaluations by the Comanche Peak Response Team (CPRT) and the NRC staff.

PARTICIPANTS:

NRC
L. Shao
R. Bosnak
D. Terao
J. Fair
P. Chen
C. Hofmeyer
C. Poslusny

CASE
J. Ellis
M. Walsh
J. Doyle

CPRT
H. Levin, et. al.

S. B. Burwell

S. B. Burwell, Project Manager
Licensing Branch No. 1
Division of Licensing

cc: See next page

NOTE: THIS MEETING WILL BE TRANSCRIBED

Contact: S. Burwell, 492-7038

8503250414

1p.

ENCLOSURE 2

TRANSCRIPT OF PROCEEDINGS

BEFORE THE

UNITED STATES NUCLEAR REGULATORY COMMISSION

WASHINGTON, D.C.

IN THE MATTER OF MEETING TO	§
CONDUCT FEEDBACK DISCUSSION	§
WITH MESSRS. WALSH AND DOYLE	§
RE CONCERNS ABOUT THE	§
COMANCHE PEAK PLANT	§

DISCUSSION MEETING

BE IT REMEMBERED that at 12:00 noon on Saturday, the 23rd day of March 1985, the above-entitled matter was had at the Ramada Inn, 700 East Lamar, Highway 157, Arlington, Texas 76010, before CHET POSLUSNY, Chairman; and the following proceedings were reported by Aloma J. Kennedy, a Certified Shorthand Reporter of:

KENNEDY REPORTING SERVICE, INC.
7800 Shoal Creek Blvd., Suite 346-W
Austin, Texas 78757
(512) 458-3297

(817) 469-8930
Dallas - Fort Worth

THE RECORD NEVER FORGETS

8505060166 238 pp.

TABLE OF CONTENTS

	PAGE NO.
PROCEEDINGS - SATURDAY, MARCH 23, 1985	2
PRESENTATION BY DON LANDERS	3
PRESENTATION BY DAVE TERAO	22
PRESENTATION BY JOHN FAIR	70
PRESENTATION BY DR. PAUL CHEN	112
CLARIFICATION OF ISSUES (Rom Lipinski and Charles Hofmayer)	191
MEETING CONCLUDED	232

P R O C E E D I N G S

SATURDAY, MARCH 23, 1985

MR. POSLUSNY: Good afternoon Ms. Ellis, Mr. Beck. I want to welcome everybody. The purpose of this meeting is to conduct a feedback discussion with Messrs. Walsh and Mr. Doyle regarding their concerns about the Comanche Peak Plant, to also request comments and any clarification from them and to allow the Applicant to also comment and ask questions.

As far as structure, each NRC team member here today will identify the key issues for each area, provide the status discussion of our effort and, lastly, request comments and clarification comments from Messrs. Walsh and Doyle and the Applicant.

We would ask that Mr. Walsh and Mr. Doyle do not address new issues at this meeting. We feel that if you have new items, we would like for you to take them to Mr. Noonan through the proper channels. We have a lot to cover today.

As you know, the meeting is being transcribed, and we ask that each speaker identify himself when you first start speaking. And copies of the transcripts will be provided to all parties.

What we would like to do is cover the summary

disposition items this morning -- or this afternoon -- first, then go into cable tray concerns. And we have a few structural allegations.

So we would like to start off with Mr. Landers, please.

PRESENTATION BY DON LANDERS

MR. LANDERS: Good morning or good afternoon. I'm here to talk really about the draft report that I submitted to the Staff on February 21st, and I'm sure that by now everyone has had a chance to review that so I will not spend a lot of time discussing specific items other than to indicate that in reviewing the design process that was in place over a period of years at Comanche Peak, that I did arrive at some concerns. And based on those concerns, I made some recommendations to the Staff, that further work had to be done before I could make a judgment on the adequacy of the design that is currently in place.

What I would prefer to do is to respond to any questions or go into detail on issues that I have addressed in the report, rather than go over them again in detail since I have already done that in the previous meeting. If that's acceptable to everyone, I would

prefer to do that.

MS. ELLIS: I think that -- Juanita Ellis.

I think that probably Mr. Doyle and Mr. Walsh are much more interested at this point in hearing the Staff's assessment of the particular technical matters rather than so much, you know, the design QA aspects of it. I think that's the primary thing that we would like to have you address if possible, as much as possible.

The others on design QA issues -- as I mentioned before, there are certain other matters that we are concerned about which we do want to get some more information from the Staff regarding.

But I think that if I could, I would like to mention a couple of things here. One thing, the summary disposition motions, I realize that this is the format that you want to follow here, but I want to mention a couple of things about that.

One is that first of all, many of the summary dispositions, if you have not noticed, were signed by Mark Walsh, and I want to be sure it's clear in your minds that does not mean that Jack Doyle has no input to them. And, in fact, many of the things he testified about are included in those summary dispositions. So, in other words, this was not just Mark talking or

anything like this; this goes far beyond that.

Also, I would like also to mention that as far as we are concerned at this point in time, our current thinking and our current feeling is that basically all bets are off as far as the plan that the Applicants presented, proving whether or not the design of Comanche Peak was adequate. We think that they are more in trouble now than when they were when they started out. And they have not done what they said they were going to do as far as Cygna, as far as addressing the Walsh/Doyle concerns. Their expert is somewhere in Wyoming somewhere on horseback, I suppose. We haven't heard anything from him yet.

The motions for summary disposition leave out many items which need to be addressed. And I think at this point in time we are very concerned about the issues and about the Staff's approach to these issues because we think at this point, based on what the Staff has already seen and what we have seen, the Staff should not be narrowing your perspective to just these motions for summary disposition.

Ken mentioned something about that, I think, in the last meeting, and I think that that's something that we want to be very clear about. As far as we are concerned, all the Walsh/Doyle concerns are still valid

concerns. They've not been adequately addressed, and we think they all need to be. So with that caveat -- I know that you do want to continue with the motion for summary disposition on that basis, but I want to make clear that that was our position.

MR. POSLUSNY: Chet Poslusny.

Is there nothing that you want to hear further from Don?

MR. WALSH: This is Mark Walsh speaking.

I wasn't aware what the agenda was going to be, so I'm not prepared.

MR. POSLUSNY: Your items here and able to answer questions.

MS. ELLIS: Both Mr. Walsh and Mr. Doyle have read the transcript of the last two meetings, the February 26th and 27th meeting, so you don't need to, you know, repeat the things that were said there. If there is anything -- what we would like to do is to find out the Staff's position on these matters as much as possible.

Go ahead.

MR. DOYLE: This is Jack Doyle.

I think we would also still like to know what the Staff position is and what he had to say in his report. So I think his question is valid, is what I'm

trying to say.

MR. BOSNAK: This is Bob Bosnak.

If you read the draft report that Don Landers prepared, do you have any questions, because he went into more than just QA. He covered a lot of technical things. So we wondered if you had any questions that you might want to ask Don Landers on that report, particularly on the technical areas.

MR. DOYLE: Not really because most of what he had to say, obviously I concurred with. What I would like to know is what is the Applicants' position on his report and what the Staff position is on his report.

MR. BECK: John Beck.

Can I interject here if I may? And I'm not making an observation on what the Staff's intent in this meeting or get-together was today. Let me make clear what our intent is and what we would like to assure happens to the best we can in the course of the afternoon.

As we've indicated in our meetings with Staff earlier, we're in the process of preparing a comprehensive response plan to a number of PRT issues. As a composite piece of that plan, we have also included what we refer to as "design adequacy." Within that

design adequacy umbrella will fall a number of issues, not the least of which are Walsh/Doyle concerns, if I could use that in quotes, that are before the ASLB at this point in time.

Our desire today is to be sure that within the constraints of those Walsh/Doyle concerns that are on the table that there are not any issues that Mr. Walsh and Mr. Doyle feel have not been adequately covered or amplified or clarified in the record because that record is certainly available to the CPRT.

So we want to have that interaction that will assure us that we're not missing anything if, in viewing that record, we haven't got the whole story. So it's an opportunity for CPRT, the Comanche Peak response effort being led by Mr. Levin, certainly as a whole on design adequacy, that he has this opportunity to interact.

I would like it to be as free and as open and as comprehensive as Mr. Walsh and Mr. Doyle would make it, and that's the prime reason we're here.

MR. DOYLE: Mr. Doyle again.

That's my feelings exactly, is that it should not be limited to a narrow scope which encompassed only the summary dispositions because, in the first place, the summary dispositions only addressed a few areas. And many of the areas that were discussed as long as

almost -- going on three years ago now, were never touched upon in the summary dispositions.

In addition to that, there are other areas that came out as a result of Cygna which for some reason also fell by the wayside. We have one motion for summary disposition that's been answerable within 20 days that's now going on six months with no answers. And I think what we want to do is get all the points up now; otherwise, we'll just be going over the same network all over again at some future date.

MS. ELLIS: This is Juanita Ellis again.

One of the things, John, that I think needs to be clarified perhaps that would be helpful to us to know is how much does your new team know because at this point in time we're not really sure, having reviewed all the records, you know. If so, then I think maybe there is a basis for talking. But if the record hasn't been thoroughly reviewed as of yet, I think we need to know where you're coming from at this point in time and what the status is of that. Could you maybe clarify that for us?

MR. BECK: I won't put words in Mr. Levin's mouth. I'll let him speak for himself in that regard.

MR. LEVIN: Maybe the first thing,

Juanita, would be to discuss how we're going about our development of initiatives to just the entire design adequacy question.

I think what we want to be sure of is that we've, in terms of breadth, identified the full range of issues that may exist on the part of CASE and Jack Doyle and Mark Walsh, as well as the Staff, as well as Cygna. And we have been in the process of trying to define those boundaries.

We're not, as part of our program, going to specifically go after, even though it will include this, but it will not be limited to specifically going after issues that are brought forward by any of those parties. The program is intended to be able to provide an umbrella that would include those as well as anything else.

So if we achieve our objective in these series of meetings -- and this is the third in a series. We've met with Cygna; we've met with the Staff back in February and here today -- our objective would be to come up with a program that is broad enough, comprehensive enough that even if we didn't know about the specific issues that have been raised by any of the parties, we would be able to detect those as well as others.

So I'm not sure that in terms of this meeting, for example, we would have to get involved in every -- you might say microscopic issue, if I can just term it that way, but we want to be sure that we've got the general areas identified, the key problems identified so that we can go forward and know that our program has the attributes, that we'll be able to identify issues that are similar to ones that have been brought forward by the three parties I mentioned, and resolve them.

So, yes, I understand there are these 16 areas. We want to be sure that we have a comprehensive understanding of what they are, anything that's related to them, and any clarification you would like to provide. And we'll come forward and undoubtedly have future meetings where we'll discuss a program that will deal with those.

And I want to assure that you it will not be limited to the explicit express concern that may be on the table right now. It wouldn't be a very good program if we weren't able to address why we don't believe there are similar concerns that have not been detected today.

MS. ELLIS: I think you've hit on one of the things, one the problems as we see them right now. You mentioned these are 16 areas. The Walsh/Doyle concerns go far beyond that, and that's one of our

concerns is how much of the record you have reviewed at this point in time, how much will you have reviewed at the time you make your proposal April the 1st or thereabouts, how much have you read of all of this?

And I would like to point out, too, you mentioned that this is a series of meetings. This is really the first opportunity that Jack Doyle and Mark Walsh have had to sit down with the NRC Staff people on a basis like this, on a one-to-one more or less basis, and discuss these issues. I think it's long overdue, and I'm awfully glad that we're getting this opportunity now.

But I think that that's one of the primary reasons that we wanted to have Jack down here is so that he could discuss some of the technical issues with the Staff and find out what the Staff's thinking is.

MR. LEVIN: Juanita, in the regard, I believe there are issues beyond the scope of the 16 summary dispositions that, for example, Cygna has raised. And we've had an opportunity to sit down with those people and review those. I believe there are some that are extensions of the 16, for example, that we extracted out of our February meeting with the Staff.

MS. ELLIS: Excuse me before you go on with that.

MR. LEVIN: Sure.

MS. ELLIS: The Walsh/Doyle allegations are closer like 30 -- just for the record.

MR. LEVIN: Okay. But I wanted to assure you that the effort is, in fact, much broader than those. Our intent -- and I think John expressed it very well -- is to get a full understanding of that from this meeting if we could, make sure that those are fully clarified in our minds in terms of not necessarily in detail -- okay? -- but to be sure that we have input that our program will have the key components in it to address the full range of issues that may be related to design adequacy question of Comanche Peak.

MR. DOYLE: My feeling is that if you don't have a complete layout of all of the problems, all of the shortcomings, particularly in the engineering, and you go through and take another bite at the apple, then we'll be right back here again for the ones that we still have in our head.

MR. LEVIN: I agree. I agree with your intent, and I want to make it absolutely clear that our intent is not to go after the specific issues that have been expressed. We want to develop a program that has the capability to detect anything at all that may be related in terms of the generic implications of the

concerns that you've expressed.

Now, what we will be able to do in our program development is deal with the key areas that have been identified to date, and we have provisions in the program. We discussed, in the February 26th meeting, a means of dealing with issues that will come up as our investigation is ongoing to be sure that components of the program that need to be added, as the review goes on, will in fact be added; in other words, that an issue coming into the process gets properly categorized and, in fact, there is a program developed to deal with it.

I think we would all be somewhat naive to believe that we could be 100 percent complete at any point in time, but we need to be flexible enough that our program can deal with it as time goes on. We're taking a crack at the record, and I can't quote verbatim of the specific sources of information that represent our data base, but it's quite long. And we're in the process of assimilating that, categorizing it, cross-referencing it so that we do have a grasp of the issues.

We believe that there are certain sources of information that get us 90 percent there very quickly -- okay? -- and other sources that either are redundant to or, you know, represent the last 10 percent, so to

speak, that we will have to eventually capture, and we'll present a methodology for capturing that. But what we're concentrating on right now is getting the biggest bang for the buck, so to speak, to get all the major things categorized first. And you'll see shortly a program and initiatives that will address those, and there will be a methodology defined on how we'll deal with those parts of the record in terms of volume that need to be addressed but possibly are not -- it would take a longer period of time to get up front in a program plan immediately, but there will be a process defined on how we'll deal with it.

MR. DOYLE: My feeling has always been -- and I've said it in testimony and I've said it in affidavits with caveats -- that I believe the plant can be saved. However, I don't believe you can address a problem until you first understand what the problem is, and that's why I was willing to come down here, is to get all the factors that I know on the record.

MR. LEVIN: I share that objective.

MR. BECK: John Beck again.

To that extent, Jack, anything that you feel is not on the record, that's exactly what we want to hear today because, you know, the record will speak for itself. And our examination of it and the process of

developing CPRT is going to incorporate what we see.

MR. DOYLE: Well, we currently have --

MR. BECK: If there is something missing, that's what we want.

MR. DOYLE: We currently have, if I recall, something on the order of -- what? -- 15,000 pages of testimony, several thousand documents plus tons of summary dispositions, affidavits, answers to summary dispositions, fourth round answers to summary dispositions. And for somebody to have to pore through all of that to have to pull out the elements that are of concern is overwhelming, and I think we could better serve ourselves if at this particular point in time we get all the issues on the table in one concise small record.

MR. POSLUSNY: Could we start with the Staff's discussion of this.

MS. ELLIS: One more thing I would like to mention again, if I may, before we go on.

I think one of the things that we're concerned about, Howard, is what about things like trends that have already been identified where you already know there is a problem? What about correcting those problems? We're concerned from several aspects. One is I guess the efficiency of operation, also the cost to

the ratepayers, which we're also concerned with. Why go ahead, if you know there is a problem, are you going ahead and building the same type of supports, say, in other areas of the plant?

This is the kind of thing I think that we're concerned about as well as looking at the specific items. MR. LEVIN: I agree with you. I think there is a key difference in the way we are approaching this design adequacy effort as opposed to the way an effort may have been started a year and a half ago by Cygna, for example, when they came in and -- essentially relative to the design question and design verification of Comanche Peak -- where they had to start with, let's say, a broad filter and identify areas that required further resolution.

I think to some degree we are at an advantage of being able to rely upon the work of a lot of other people, including the Walsh/Doyle efforts in the past. And so to that extent we're starting there. I think there is a degree of verification that goes along with that in terms of trying to appropriately define what the issues are that we want to attack.

But we're not, for example, starting off with a fresh IDVP. We're, in fact, starting off with the issues defined. We'll qualify them in terms of

describing what problem we really want to attack. Now, some of them are going to be very direct. We've already -- I think we've indicated in past meetings that we're not going to start, you know, a similar process as may have existed in the past again. We're going to take direct solutions. There are certain pieces of hardware that we believe the most direct path of resolution is to modify them directly. We're not starting from where maybe you were two years ago. Okay? We're going to try to take advantage of that, learn from that, and go and take that corrective action if it's indicated.

So I agree with you.

MR. WALSH: This is Mark Walsh.

In regards to your program, a trend that I have seen coming out of these hearings and the motions for summary disposition is that the Applicant has not had an effective quality assurance audit program either from Gibbs and Hill or Grinnell or NPSI or we wouldn't have these problems right now. So when you go looking at specific problems, there is the problem. There is not an effective audit, and it's more than just one organization. It's the whole plant as a group.

Gibbs and Hill structural group had problems with their cable tray supports, the upper lateral restraint. The audits that were supposedly occurring

were not effective. And just to say, "Well, cable trays are a problem," it's the whole organization that's a problem because they failed in their technical audits.

And, you know, essentially what I'm getting at is the solution to that problem is go back and audit, do a technical audit on all the calculations or whatever, not just cable trays but the whole smear because they have failed or we wouldn't have these problems right now.

MR. LEVIN: Mark, I think that certainly in terms of approach and the way we would like to deal with that, we are developing a methodology that, for example, let's say we have an issue in the cable tray area and we know a few things about that. We know who did the work; we may be able to learn something about their programs. Certainly our initial focus would be to verify the quality of that end product design. Okay? We need to deal with that.

But, you know, as we ask ourselves questions as to why that occurred, the first place we'll start, to try to define how broad or narrow it may be, is to test the work of that group. The first question we ask is: What else have they done? Okay? And if they have contributed to other design products on site, then we will, in fact, go look at those products.

But I want to make one thing clear. We still again -- getting back to Juanita's point -- want to take advantage of existing work. Cygna has audited calculations that may be of value or give us some input, insight into that question as well. It won't be starting from scratch. I think that Cygna probably has looked at or other organizations likes TRT -- for example, you mentioned other calcs in the civil area -- where we'll learn something that will have, you know, some impact on that question.

You know, I look forward to the opportunity to show you how we're going to do that. I'm trying to pull out some paper here of the kinds of attributes that might be considered in such a question in terms of our trying to either isolate it or, if it is broader, to identify that fact.

Let me list some attributes that would tend to qualify this, and this is going into the development of a logic.

The first point I mentioned was what was a common engineering discipline, what the related engineering disciplines might be, responsible manager or supervisor, responsible organization, responsible work centers, interface with other groups, whether it was done under the same program or related program, same

procedures, related procedures, same QA/QC organization, the same process. Questions like that will be asked to try to isolate -- either isolate or expand it, as the case may be.

And the list is longer, and we hope to be able to define a logic that -- this is input into that, and it's ongoing right now as to how we'll accomplish the intent.

MS. ELLIS: We'll look forward to getting more on that.

One more comment, and then we would like to hear from the Staff.

I just want to point out that Jack and Mark will do the best they can to respond off the top of their head here today, but I want to emphasize again, as far as we're concerned, all the Walsh/Doyle concerns are still open questions.

MR. POSLUSNY: We'll get started. I just wanted to answer one question. We have not developed our position on Mr. Landers' report yet.

So should we begin. Dave.

PRESENTATION BY DAVE TERAO

MR. TERAO: Dave Terao. Okay.

I think the first item of the motion for summary disposition I would like to talk about is the issue of stability.

Let me ask one question before we get into that. As Chet mentioned, that Don Landers' report is still in draft form. It's not formally reviewed and accepted. But at this point one of the purposes of this meeting, we thought we would like to get your comments on it so that we could factor it into the final Staff position. So if you did have any comments, I would like to hear that today.

MR. DOYLE: Well, the only point in there with which I couldn't 100 percent agree with is he kind of treated seismic lightly. While I do agree that the transients are more critical and can cause more grief, seismic is still a problem for the unstable supports.

MR. LANDERS: May I?

I think, in fact, Jack, that I exclude stability from that argument, that when I suggest that I'm not overly-concerned about seismic, I exclude the stability situation because I'm concerned about the stability situation in a non-seismic environment.

MR. DOYLE: Yes.

MR. LANDERS: So when I say my concerns with respect to seismic, as pointed out on Page 8, are related to a number of other issues which have nothing to do, in fact, with the stability issue, but it is really related to the loadings that are imposed on the plant versus the loadings for which the plant was --

MR. DOYLE: A lot of times I'm reading this stuff, I kind of read it through a fog. I'm working 12 hours six days, and then I have to in my spare time go through all this material.

MR. LANDERS: I understand. That is an appropriate point. I also separate stability from my lack of seismic concerns --

MR. DOYLE: Yes.

MR. TERAO: Okay. This is David Terao again. And with that, I think it leads right into stability.

What I will basically be doing is getting into some of the details of the Staff review of the stability issue. At the February 26th and 27th meeting with the Applicant, I really did not get a chance to get into the details. So actually this meeting is beneficial for both the Applicant and for CASE today.

I think, Howard, you wanted me to get into the

details at that meeting, but apparently it wasn't structured for me to do so.

Let me just basically try to summarize what the issue on instability is. This may be a little rough, and I apologize. But the issue of instability of pipe supports first came up back in about 1982 when CASE witness Jack Doyle submitted several preliminary design drawings of the Comanche Peak pipe supports which he alleged were unstable. And in particular Mr. Doyle alleged that the supports utilizing struts or snubbers in combination with box frames or U-bolts could rotate around the supported piping due to the presence of gaps. That was the initial issue of stability back in the September 1982 hearings.

Mr. Doyle also expressed concern about similar supports without gaps between box frames or U-bolts as also being unstable because these gaps could be formed by yielding and cause permanent deformation of the pipe, box frame, or U-bolt and because friction forces were insufficient to preclude rotation. That's rotation of the frame around the pipe.

So according to the Applicants -- Mr. Doyle also raised the concern with double-strutted single plane frame pipe supports. Following the presentation of evidence on this issue by the parties, the Board

decided in their December 28, 1983, design decision that the Applicants had not presented sufficient evidence on the issue of stability, including the safety significance of the unstable designs and an explanation of whether or not the problem was promptly detected by the Applicants' design QA organization.

So in response to the Board's two orders, the Applicants committed to provide a detailed description of the evolution of the instability issue, and these are provided in the motion for a summary decision positions on stability.

That's basically the background of the issue. What I would like to do is first address the Staff's response to the Applicants' summary disposition motion.

The Applicants discuss the nature of instability in the context of individual pipe support and piping system design. The Applicant referenced the ASME code, Subsection NF, Appendix XVII, Paragraph XVII-2221(a) which states, quote,

"General stability shall be provided for the structure as a whole and for each compression element,"

end quote.

So according to the Applicant, there was not just one form of instability. Stability for each

compression element -- that is, individual pipe supports -- can occur due to column buckling or rigid body instability. And the instability discussed in these hearing', according to the Applicant, was the rigid body instability.

The Applicant also discussed general stability in terms of piping system stability. Although an individual pipe support, when viewed unattached to the piping, may appear to be unstable, the relevant consideration is whether the entire piping system with the pipe supports attached to the piping is stable when considered as a single system.

Furthermore, the Applicants did not believe it was necessary to explicitly address the stability of piping systems in piping analyses because through the normal design process, the piping designers achieve a system which will stay within the specified deflection limits, assuring system stability.

That was basically a summary of what the Applicant had said in his motion for summary disposition. The Staff position on that -- before I get into the Staff position, I would like to discuss a little about the concept of instability and its consideration in the design of piping systems and individual pipe supports.

There has always been a problem in defining "instability." I think we've recognized that. The Board recognized it in its design decision. There have always been examples given of what an unstable pipe support is, but no one really came out with a definition. That was one of the difficulties that the Staff had in trying to review the Applicants' summary disposition motion because, in reviewing the hearing record, we never found that anyone ever accepted one definition of what an unstable pipe support was.

Basically we're talking about a static instability. And the textbook definition can be understood as follows -- this is taken from Elementary Structural Analysis by C. H. Norris and J. B. Wilbur. To quote:

> "If a system is displaced slightly from its equilibrium position, does it tend to return to its original position, or does it tend to displace further when the disturbance is removed? If it returns, the system is stable; if it displaces further, the system is unstable,"

end quote.

Now, that's the textbook definition of "static instability."

All right. Now, pipe support instability, as addressed by CASE and Applicant in this proceeding, does not really fit this classical textbook definition of instability. As I said, the Staff review of the record on that case is concerned with box frames and U-bolt pipe designs is the potential ability for the box frame or U-bolt to rotate around the pipe or slide along the axial length of the pipe due to a loose or unpredictable clamping mechanism between the pipe and the support.

CASE has also characterized the unstable support as a three-bar linkage which, of course, cannot accept the load in compression.

The Applicant has defined pipe support instability in terms of, one, a collapse or a buckling of a column or, two, rigid body instability where a support can carry no load in compression.

The second definition was this presented by Cygna in the April and May 1984 hearings. But, of course, Cygna's definition has changed considerably in their February 19, 1985, letter.

The Staff finds that instability of pipe supports as discussed in these hearings is related to the overall condition of a pipe support being non-functional; that is, unable to perform its intended function.

Now, the Staff believes that instability of an individual pipe support should be defined as the capability of a support that shifts to an unqualified position; that is, a position other than the position assumed in the piping stress analysis which could significantly affect the validity of the piping analysis results.

Now, that's a very broad definition of instability. Instability of a pipe support could lead to failure of the piping system by various failure modes, including instability of the piping system itself.

That was basically a discussion of the definition of stability. Maybe I should stop there and get any feedback from CASE at this point.

MR. DOYLE: I don't think I have anything to add to it. I think what you're saying is an accurate statement. Once you get rotation, then you do not have the same condition that was assumed for the stress input.

MR. TERAO: Okay. What I would like to discuss next is, in the Applicants' summary disposition motion, there was a discussion of industry practice regarding consideration of stability and piping and pipe support design process.

Okay. In general, the Staff agrees with the Applicants' discussion of standard industry practice regarding consideration of stability. However, the Staff does not regard this discussion as being relevant to the situation regarding the pipe support instability at Comanche Peak. The Applicant stated in its statement of material facts, Paragraph 1; quote:

> "Instability of a particular pipe support, when viewed in isolation from the piping system, is of little or no significance. The relevant consideration is whether the entire piping system and associated supports are stable when considered as a single system,

end quote.

Now, for standard industry practice related to pipe support design -- that is, when one uses your standard pin-to-end supports together with conventional pipe clamps -- the Applicants' first statement is valid. For this situation, a pipe clamp with a support, when viewed without the pipe, appears to be unstable. It's not self-supporting, in other words.

I think this was substantiated by Mr. Doyle. He noted in the transcript that STRUDL cannot analytically model a two-pin strut compressior without the pipe because the analysis will result in unlimited

rotations at the pin joints, and this results in an unstable condition.

However, the Staff notes that while the STRUDL analytical model cannot calculate its condition of pin struts without the pipe, the problem which arises is the limitation of the analysis and the analytical model due to the decoupling of the pipe from the support and not necessarily the fact that the pin strut attached to the pipe is an unstable condition.

The Staff does not believe that there is any disagreement on this point by CASE. The Staff would also note that industry practice dictates that pin struts do not have to be analyzed using STRUDL because these struts are classified as component standard supports which have been previously qualified by a load rating method by the manufacturer.

And the same principle applies to other component standard support items such as clamps, extensions, brackets, and U-bolts. The standard industry practice dictates that the application of component standard supports in a conventional manner precludes the need for subsequent analyses based on years of previously established and a proven design.

The Staff's concerns stem from the fact that many of the pipe support designs at Comanche Peak

represent either an unconventional application of the component standard supports which have not previously been proven to be acceptable, or the use of unconventional support designs.

It should be noted also that when one connects the pipe clamp to the piping system, the Staff concurs with the Applicant that the relevant consideration is whether the entire piping system and associated supports are stable as a single system. Again, I'm talking about a conventional type clamping support.

The Staff's understanding of the Applicants' statement, that each individual pipe support is not required to be self-supporting or self-standing if it is not attached to the pipe, with which the Staff agrees. The Staff would not necessarily concur that if a particular support were unstable when viewed with a piping system, that there is little or no significance if the system as a whole could be determined to be stable.

And it was in this context of standard industry practice that the SIT Report made its statement at Pages 27 through 28. I won't necessarily read what the SIT Report said at that portion, but what the SIT Report on Pages 27 and 28 was referring to was that a cinched U-bolt with no gap would function similar to a

pipe clamp.

And the SIT Reports also stated that for a non-rigid box frame -- that's the box frame on pin-end supports -- which could potentially rotate around the pipe, the SIT Report addressed the proposed modifications to prevent the rotation of the box frames around the axis of the pipe in order to assure system stability. So the Staff does not believe that the SIT Report was incorrect in what it had said on Pages 27 and 28.

For piping systems, the SIT Report was referring to the fact that system instability cannot be determined using established piping stress analysis techniques but can be assessed most effectively by piping and support designers using good engineering judgment and based on years of experience and common sense rules for supporting piping.

Now, the Applicants stated in its Statement of Material Facts, Paragraph 2:

"Stability of piping systems is not as explicitly addressed in piping analysis. However, it is not necessary to do so because through the normal design process, the piping designers achieve a system which will stay within the deflection limits and

thus will be incapable of the instabilities at issue here,"

end quote.

The Staff does not concur with the Applicants' above statement, that staying within specified deflection limits for piping or supports will maintain system stability. If a piping system were supported in a manner which resulted in an unstable system, then that system, if displaced slightly from its equilibrium position, would tend to displace further, per the textbook definition of static instability.

Furthermore, the Staff is not aware of any specified deflection limits for piping thermal expansion at Comanche Peak which can also cause system instability.

The Applicants in the above statement have incorrectly relied on the validity of the piping analysis results to predict the piping and support deflection in order to assure stability while the analysis itself is incapable of calculating unstable piping system behavior and large deflections associated with pipe support instabilities discussed in these hearings.

Thus, the Applicants' justification of staying within the analytically predicted deflection limits to

assure system stability is not valid. And, as a result, the Staff found -- I believe it was imperative that the piping engineers assure system stability by reviewing the piping and support configurations. And we mentioned those back in the February 26th and 27th meeting.

To conclude: The discussion on standard industry practice, the Staff review of the Applicants' discussion on industry practice regarding consideration of stability and piping and pipe support designs includes the following:

The Staff finds that unstable pipe support designs at Comanche Peak do not conform to standard industry practice; that is, the unstable designs are unconventional designs.

Furthermore, although the normal iterative design process is adequate for ensuring the stability of piping systems utilizing conventional pipe support designs, the process is not adequate for ensuring the stability of unconventional pipe supports which have not been adequately reviewed in its initial design conception.

Thus, the Staff finds the Applicants' discussion of industry practice for stability and piping and pipe support designs is irrelevant. The relevant consideration is whether the basic performance

requirements have been adequately considered in the initial pipe support design in order to ensure the functionality of the pipe support and overall acceptability of the piping system.

That concludes my basic discussion of the overall issue of stability. I can get into some of the more specific examples given in the summary dispositions, but at this point let me stop and get any feedback from CASE.

MR. DOYLE: I can't really think of much to add to that. The point you made about using standard components, I made in my initial summary disposition, that if you do have a double pin, strut, snubber, or what have you, with a conventional clamp, while the double pin, if you've undone the clamp, would collapse, attached to the pipe with up and down stream supports to assist, there would be no instability. It is only in the unconventional where we address it or where I address it and what have you.

MR. WALSH: In regards to the cinched U-bolt, that not being clamps, at the time the SIT Report came out, an attachment has not been tested, and there was no verification of the program in place to verify that the bolt had sufficient torque to hold it in place.

And from what I understand right now, is the Applicant has gone in there and painted the bolts. So if they're going to go in there and torque them now, they're going to be getting false readings.

So the items they tested in Unit 2 do not necessarily mean they were torqued the same way in Unit 1. And to go in there now and torque the ones in Unit 1 will give false readings because of the paint.

MR. TERAO: That's because of the paint on the threads themselves. And if they torque them, they will get a false reading of what the bolt torque is?

MR. WALSH: Correct.

MR. POSLUSNY: Are there any other comments?

MR. LEVIN: Chet, do we have an opportunity to clarify things that they've mentioned?

Let me make sure I understood at least one portion of your discussion. It had to do with the concept, which I think is very fundamental, that in order to qualify a piping system design, you want to have evaluated it and its expected behavior completely and understand the way that we believe it will behave represented in the piping system model. And that concept I think there is full agreement on.

And I think that's inherent in your definition also. And also inherent in the definition is the concept that what we're really dealing with here is the system. That's what we're after -- I mean, you know, meeting certain performance requirements of the system.

But if we could just hypothesize something -- and it's not that I have anything particularly intended or particular configuration in mind -- but if we had fully evaluated a system and there were a particular component whose behavior may exhibit an individual basis, things that people felt might represent instability, but the system as a whole still met its performance requirements, is that in your mind still -- I mean, how does that fit within your definition of "instability"?

Is that an unstable situation, if, in fact, we could agree that we analyzed that condition and we could get agreement that its behavior was adequately represented in a model?

MR. TERAO: Well, I think the difficulty there, Howard, is that with many of these unstable designs, there is a question of whether or not the assumptions used in the piping stress analysis are valid, whether or not one can assume that the support is as modeled in the stress analysis.

If you can somehow demonstrate that the analytical model is appropriate then, of course, yes, you can look at system stability.

MR. LEVIN: You've clarified my point.

MR. TERAO: But I think the difficulty that we're having is that we believe these designs, because they are unconventional, tend to invalidate the type of assumptions used in the stress analysis, and it's very difficult to analytically show in a model how these pipe supports are going to behave.

MR. LEVIN: I recognize that some of them may be difficult to represent analytically. And we may not have, you know, a full -- it may be very difficult for us to come to some kind of agreement, or anybody, as to how to do that for certain support designs. And I think we've recognized it very early.

And certain types I think we're going to want it -- for that reason -- is correct. It's just the easier solution path. But I believe there may be a subset that doesn't quite exhibit such difficulties in either modeling or it could be -- maybe we could learn something through a test or something that would tend to qualify its behavior so we could represent it in a model.

Okay? The objective is, though, to get the

information in front of us that provides the proof, if you will, that, in fact, this is how it will behave; and, in fact, given that, this is how it should be represented in the system model.

So I don't think it's a black/white. I think there are certain particular pieces of hardware that are. We can look at them very readily and say it just isn't appropriate to try to do anything, either detailed analytical studies or testing, because of the nature of that configuration is not going to get us anywhere. It would still leave very many open questions as to our ability to be analytically represented in the system.

MR. TERAO: Right.

MR. LEVIN: But I believe there are some possibly -- at least I want to allow that conceptual possibility -- that we could develop some better improved understanding as to the behavior and possibly, you know, as we reconcile behavior as we believe it will be out in the field, in revised stress analyses, that that avenue be left open. There are a lot of tools in the box, and that's still one of the tools, I believe anyway.

MR. TERAO: I believe Don Landers mentioned that in his report, in his discussion on as-built reconciliation. He believed that it was

necessary for an experienced piping designer to look at some of these support designs in conjunction with the pipeing system to assure that the assumptions he used in his analysis have not been negated.

MR. LEVIN: And I just wanted to assure you that that exists in our program, and we intend to do that.

MR. WALSH: Excuse me. This is Mark Walsh speaking.

Gary Krishnan who was the site stress leader, group leader, we have in the record, and he could not tell an unstable support if you showed it to him because he's not a pipe support designer. I take that to mean that even people below him would not be able to look at a support and determine if it was stable or unstable, and they would still continue to analyze it, if it was unstable, as a stable support. So you cannot rely on pipe stress analysis or the person doing the analysis to model in if it's a stable or unstable support. That's going to come out of the pipe support group apparently.

MR. TERAO: Well, maybe I didn't make it clear. I wasn't inferring that only the piping people should look at it. Of course, the program should include both experienced piping and pipe support designers working in conjunction in looking at the pipe

supports and the piping system, not only the piping designers.

MR. DOYLE: One thing I would like to state, though, is in the case of all of these double-pinned struts or snubbers with either a box frame or a U-bolt with a gap, I don't think there is a prayer of saving those because they are unstable in and of themselves.

MR. TERAO: I would concur with that.

On the other hand, maybe the Applicants should address this. But I'm under the impression that they fixed all the box frames with gaps so that all of them either had zero gaps and all the U-bolts which had gaps in them, U-bolts on trapeze designs have been cinched up. So . . .

MR. DOYLE: But at this point in time, we're not certain that cinching up is a viable solution.

MR. TERAO: I agree. That's another --

MR. DOYLE: That's another can of worms.

MR. TERAO: But as far as what you've just mentioned with U-bolts and frames with gaps around them, I believe those have been corrected.

MR. DOYLE: In other words, we're not discussing that particular aspect of the instability problem.

DR. CHEN: I will pick up this point, Jack, when I come to discuss U-bolts.

MR. DOYLE: Oh, okay.

MR. LEVIN: Jack, I just wanted to add one thing. I believe that the particular types of hardware you mentioned are under serious consideration by us and strong candidates for modification.

MR. DOYLE: I'm think I'm lost.

MR. LEVIN: You indicated particular pieces of hardware that may exhibit properties, you know, possibly the support to get in a position that would be indeterminate. And you mentioned box frames and single struts, things like that, and that's the type of configuration that we're prioritizing right now and taking a very serious look at. And that's one of the things at the top of the list. It's one of those things that kind of jumps at you first. And we're taking a serious look at it, and they are strong candidates for being corrected.

MR. WALSH: This is Mark Walsh again. Earlier I said something about a QA audit, technical audit. Why wasn't a QA technical audit catching these unstable supports up to the Cygna Report that came out a few weeks ago? Why didn't someone from Texas Utilities or Grinnell or Gibbs and Hill, say,

"Hey, this stuff is no good," through a technical audit, if that did exist?

MR. TERAO: I could only speculate on that, Mark.

When one reviews a support design, especially in the bulk that was transmitted in the as-built process, if one looks at the drawings without going up to the site and looking at the supports themselves, there are just too many details in the support design to look at. And stability, of course, is one of them.

If the person had the support design drawing and went to the field and looked at it, he may spot those kind of things. But because they are unconventional, it is very difficult to look for those kinds of characteristics in a support. In fact, that was one of our conclusions, is that the design review required under ANSI N45.2.11 was really not sufficient to catch those kind of unstable characteristics.

It is very unique to Comanche Peak, and it's very difficult in this nuclear industry to have someone look at a support characteristic that no one else has ever looked at before. So it is a very difficult thing to catch. But now that we're aware of it, we're hoping that at least now the support designers know what to look for. So initially it was very difficult to catch

those kinds of things because of the unconventional designs.

MR. LEVIN: Dave, amplifying on that -- and I would like to say a few things.

No. 1, you know, the process that existed is somewhat water over the dam with respect to Unit 1. And the appropriate thing to do with Unit 1 is to deal with these problems and correct any that exist.

I certainly hope that in the process of our investigation, we'll learn some things towards the answers to your question that we will factor into Unit 2 as well as, if they're appropriate, in terms of what we learn problematically, into the operations phase of the two units.

So I think it's an appropriate question from the standpoint of lessons learned, cause, and trying to correct things in the future. Relative to the specific hardware in Unit 1 and trying to correct it, I think it may help focus our investigation. But the important thing with Unit 1 is, in fact, to make sure that the quality of the design and construct of the product is acceptable.

MS. ELLIS: This is Juanita Ellis again.

I would like to make just one comment just to throw in for whatever it's worth.

I understand what you were saying about the difficulty in identifying these things. But at the same time, once the problem has been identified -- which it has been in these hearings sometime ago -- then it would seem to me that this is the kind of thing that people would be more on the alert to look for.

And I remember specifically -- I've seen Jack look through drawings and Mark look through drawings, and there is unstable support, you know. And it seems to me that it has been very slow in coming, that the Applicants have really looked at these problems and identify the problems.

I just wanted to mention that because I remember specifically, you know, when flipping through drawings that we received, say, on some other motion for summary disposition, Mark would flip through those and say "Oh, here is an unstable support."

MR. TERAO: But to address that, Ms. Ellis, it gets back into the difficulty that no one really defined what an unstable support was. So even though you clearly knew what an unstable support was, the Applicant may not have agreed with that definition and was looking for maybe a different type of instability.

MR. WALSH: This is Mark Walsh again.

In regard to what the Applicants wanted defined, what we were talking about was engineering mechanics, not English. The thing was mechanically inoperative. They may have not realized it, and maybe that's why they're having a problem.

If they cannot recognize a problem now, how would they be able to recognize it when they get an operating license? They've got to look now -- we look at these problems in the Applicants' position, as maybe their position when they get an operating license. They're not going to come out and say, you know, "We've got thousands of supports unstable." They're going to come out and say, "We've only got 15." And that's the why they're going to operate that plant.

MR. TERAO: I can address that. The Applicant -- it is both a question of English and mechanics, and both of them are important. From the mechanics point of view, you have to understand -- at least from my understanding of the record -- that the Applicant relied on his engineering judgment to justify the mechanics of the support. Now, of course, the Board ruled that was not appropriate, and the Staff would concur that with unconventional designs, that is inappropriate, too.

But it wasn't totally just that he did not

understand the mechanics of it, but maybe his judgment differed from what your judgment was. One can't deny that there is friction of some kind between a U-bolt or a box frame. But your point is well-taken, that it's uncontrolled and there is too much uncertainty involved, although you cannot deny that there is friction there. But the Applicant relied on that friction.

MR. WALSH: And he had no basis for that reliance. That's how I see it. They had no tests; they had an unconventional design; they had no method of proving the thing would work; yet, they went along with the idea that it was okay.

MR. LANDERS: Excuse me. Could I ask a question that addresses the going forward with respect to the stability problem? Is it acceptable to step in here?

MR. POSLUSNY: Go ahead.

MR. LANDERS: One of the points that you brought up, Howard, with respect to asking Dave about black and white issues with respect to stability, you said there are some that perhaps cross the line. I needed just a little bit more input on that, if you have it now, as to what you're thinking because I see a philosophy with respect to your approach in that and I would like to . . .

MR. LEVIN: Well, I guess fundamentally what we're talking about, Don, is that when we get into a position where we feel that we have understood the behavior of a particular support configuration, where that understanding is derived from an analytical investigation or a test or whatever -- okay? -- that if that can be represented in a conventional piping analysis, that that be an avenue that's open to us.

You know, just like we know how to -- I think a clamp and a strut pin-pin configuration is a conventional configuration. We know how to represent that, and there is enough input on the record that we have confidence as to how you deal with that in a stress analysis.

MR. LANDERS: I had not heard your statement with respect to the fact that when we get to the point that we understand --

MR. LEVIN: Absolutely.

MR. LANDERS: -- through the test or an analysis that would be acceptable to all of us.

MR. LEVIN: Yes.

MR. LANDERS: Fine.

MR. DOYLE: But I would like to add one thing to that. Many of the tests that have been done in the past and many of the analyses done in the past by

Applicant have always been uncoupled; in other words, they will prove that the clamp will create friction, there is no doubt. However, the clamp now introduces several new factors. Are the new factors also going to be taken into consideration?

In other words, there would be a study to determine any adverse impact from whatever modification is required because we have noticed in the past that an Applicant has had a tendency to jump off the deep end when the fix fails. As an example, the clip angle -- I don't know if anyone was at the Cygna -- but the clip angle failed, wouldn't function. The bumpers are not too swift. The bracketry for the same one that now has the bumpers was bound up and picked up or in this moment that . . .

So we would be assured that any modifications or any acceptance goes beyond just an uncoupled analyses and would determine what adverse impact would result from the fix or modification or as is.

MR. LEVIN: I agree conceptually, Jack. I mean, it's not a very good fix if it doesn't work. I think that's what you're saying.

MR. DOYLE: Yes.

MR. LANDERS: Could I again add something?

I think that's probably the most important part of my draft report, that you can't separate issues, you can't separate a support from a system, you can't separate a portion of the support from the whole support. And I would hope that if the Staff doesn't accept any other part of my report, they will accept that part.

MR. DOYLE: I have been saying the same thing for three years, that many of the issues that I'll mention later in and of themselves may seem insignificant, buy when coupled in a half a dozen to a support, the support could actually be in trouble before you apply the design.

MR. LANDERS: I would go beyond that. If you can't separate the support from the --

MR. DOYLE: That is true. That is correct.

MR. LANDERS: It's a system.

MR. DOYLE: One of the problems seems to be everybody thinks that their pipe is delivering the load to the support; whereas, it is the reverse -- that is, the actual fact. So you have to look at the total picture in order to see what has been presented.

I concur.

MR. LEVIN: Don, in that regard, I know

the Staff hasn't taken a position, but we're very quickly evolving to a position and a program. And I might just add that I personally concur with that aspect of your report. And it's our intent to integrate many, if not all, of the factors that you identified in your list in terms of a system evaluation, as part of our program. And that will be done.

MR. TERAO: Let me just briefly run through some of the specific examples that were given in the motion for summary disposition on stability. If you have any questions or want to discuss it in detail, then we could discuss it in detail. But what I would like to do is just basically go over what the Staff has found with some of these specific examples and the modifications to them.

The first support is your basic box frame with single strut. According to the motion for a summary disposition, those box frames with single struts which had gaps in them had all been modified, and modification consisted of one of three different modifications. The first modification was to add a U-bolt to the box frame; that was what you were referring to at the Cygna hearing.

MR. DOYLE: Yes, sir.

MR. TERAO: I would like to defer that to

the discussion later on when I talk about cinched U-bolts.

The second one is the use of index lugs on the box frame. With the index lugs, what the index lugs apparently were intended to do was to prevent the box frame from rotating around the pipe itself. The Staff found that to be an acceptable modification to prevent the rotation of the box frame around the pipe; however, they were also concerned about any out-of-plane seismic motion which would disengage the frame from the lugs themselves, and we're back to an unstable condition where then the frame, if disengaged from the lugs, could rotate.

It wasn't really clear in my reading of CASE's response to the Applicants' summary disposition motion whether you, Jack, recognized what these index lugs were for.

MR. DOYLE: Yes, we recognize that.

MR. TERAO: You seem to say that the frame could still rotate around the pipe, even with the index lugs.

MR. DOYLE: No. It is a walking problem.

MR. TERAO: Staff actually did go out to the site and look at this support. I do want to point out that there is only one of this index lug

modification in Unit 1 that we're aware of. That was the only one that the Applicant has found.

MR. DOYLE: I was only aware of it from the standpoint of Cygna's concern over it. Cygna had -- I believe it was Cygna -- Cygna had got involved in the index lugs.

MR. TERAO: I don't recall the index lugs being addressed by Cygna.

MR. WALSH: Do you have the diagram of this index lug that I could look at?

MR. TERAO: Yes.

MR. DOYLE: I've been involved in so many hearings, I can't remember anymore. But at any rate, I was aware of the index lugs. I was not aware of how many or if, in fact, they were installed.

MS. ELLIS: I believe in Cygna's February 19th letter, they mention that all three of these fixes have been completed.

MR. DOYLE: That's right.

MS. ELLIS: Right. But I don't know if they ever really said anything about index lugs.

MR. DOYLE: Anything about it in the summary disposition, probably didn't put in there. The only recollection that I have that I was sure of was Cygna.

MS. ELLIS: Yes.

MR. WALSH: This is Mark Walsh here. I'm looking at Drawing CT1-008-S22K. The lugs that are indicated on this drawing appear to support the frame and do not restrain the frame from rotating.

MR. TERAO: Okay. But if you look carefully, I think I noted --I circled it in red -- there are four notched plates that are welded to the frame to which the index lugs themselves fit into. In other words, the lugs are welded to the pipe, and the four notched plates are welded to the frame and the lugs fit into those four notched plates.

MR. DOYLE: I think Cygna pointed out that they're only on one side, so you could get walking.

MR. TERAO: You still can get walking, I agree; but the rotation is still taken care of.

MR. DOYLE: Yes.

MR. LANDERS: If you don't get walking.

MR. TERAO: If you don't get walking.

MR. DOYLE: Right.

MR. WALSH: I recognize that now.

MS. ELLIS: For the record, this is Exhibit F-1 from the September 24th, '84, Applicants' letter, Section F on stability.

MR. TERAO: The third modification was

the addition of a strut; in other words, making a single strut box frame into a double strut box frame. And in some cases they became triple strut box frames.

I would like to discuss that later, too, because the fourth category are the double strut supports, so I'll discuss that. But with respect to the use of snubbers, the Staff found that the Applicants' discussion really didn't address the snubbers. The Applicants' modification, when using snubbers, can still walk along the length of the pipe. And the Applicants' discussion only addressed the limitation of the double struts.

The second example given in the motion for a summary disposition are the U-bolts with single struts with gaps. The U-bolt with single struts with gaps, apparently there are two modifications done. One was to snub the U-bolt, and the second one was to add the stability bumpers.

The Staff basically agrees that the use of stability bumpers was not acceptable because support could cock against stability bumpers and thermally constrain the pipe from expanding. So even if the analysis showed that the support was not necessary, we still believe that it's imperative that those stability bumpers be removed.

With respect to the snug U-bolts, I'll get into that next because the third one are the U-bolts with single struts without the gap; in other words snug U-bolts.

Basically this issue, the Staff has not completed our review because it interfaces so closely with what Paul Chen is reviewing; in other words, the use of U-bolts on the pipe itself.

But from a stability aspect alone, perhaps we could have a discussion on what your concerns are with the use of U-bolts from a stability aspect.

MR. DOYLE: With or without gaps?

MR. TERAO: Without gaps.

MR. DOYLE: Well, without gaps, we again get into the problem of an uncoupled approach. First, once you cinch the U-bolts, particularly at the loadings that they're discussing now because of the walking problem, you're approaching the limit that the manufacturer has indicated that that particular U-bolt is good for. Now, that is prior to the pressure in the pipe, which is a minor contributor, the thermal, which could be a major contributor, and the design loads which, while not additive, will increase the load.

So therefore, now, even if the U-bolts prove to be a good system for establishing stability, you

still have the problem of qualifying the U-bolt because you are now outside of the manufacture's LDS. The U-bolt is not qualified.

In addition to that, as was pointed out, you have the pipe. The pipe is now receiving the effect of the load induced by the cinching, the thermal and the pressure constraint on the pipe itself. These are additive to the MNS of the pipe under whatever conditions it is determined.

Particularly -- the one that concerns me the most is the cinching because that is a sustained load. That particular load will be there throughout the life of the plant, or the fix is no good. So I have a feeling that the allowables will no longer be similar to what they are for faulted conditions or thermal where you get into -- what it is, 1.25 SC, SH? I have a feeling we're in the area of sustained loads, or there will have to be something established to qualify higher loads than are currently existing for sustained loads.

See, this is again a unique problem. Once you cinch that U-bolt, 40 years, whatever the loads induced into the pipe or whatever the loads on the U-bolt, whatever the loads on the frame that supports it.

MR. TERAO: The reason I brought this up -- I agree that those are concerns. I won't say those were

the concerns which related to stability. Those concerns are being looked at by Paul Chen.

MR. DOYLE: Right.

MR. TERAO: In this discussion of stability, I saw no problem with code violations for using a cinched U-bolt to prevent the rotation of the support around the pipe. I believe this is what the SIT Report was saying, too, that at that time, just the fact that you cinch up a U-bolt, you will establish a friction between the type of a U-bolt -- the SIT Report was relying on that friction to prevent the rotation of the support around the pipe.

MR. DOYLE: Well, I concur that the cinching of U-bolts will prevent rotation. My only statement is that we can't drop it at that point.

TERAO: I see. Fine.

I would agree that Staff also has other concerns about the use of U-bolts on large bore pipes -- not related to stability.

The fourth category, this is double-strutted supports, double-strutted frame supports. The Applicants' basic argument with the double-strutted frames was that the two struts now prevent the frame from rotating around the pipe axis. The Staff has had several meetings with the Applicant where we also

expressed concern about the out-of-plane citation of the support, walking along the length of the pipe to an unqualified position.

And we asked that the Applicant identify all double-strutted supports. And in the September 24, 1984, letter, the Applicant did provide us with 44 double-strutted supports. And as I mentioned before, the one concern is that the Applicant still has not addressed the use of double-snubbers because the snubbers can extend as the frame walks along the length of the pipe.

The Staff is also concerned with a subcategory of these double-strutted supports which is the multi-supported frame which has four piping systems going through it. Again, the Applicants' summary disposition motions and supplements to it really did not address the Staff concerns brought up at the previous meetings, including the dynamic interactions of the frame and the four piping systems, the twisting motion of the frame. So basically at this point, there still is not enough information provided to the Staff to address our concerns.

Also in the 44 supports, the Staff noticed that there were some double-strutted frames which did not have a zero clearance gap on all four sides. The

zero clearance gap were only provided on the two sides; and on the other two sides, there was a gap. The Staff also believes that those supports are unstable, similar to what's the Cygna concern was because now you have a gap on two sides of the frame, the support frame can now cock itself. So we find those to be potentially unstable, too, and those had not previously been identified.

Also among the 44 supports, there was a support which we mentioned at the February 26th meeting which was a triple-strutted frame resting on a structural steel. Apparently, there was a vertical pipe and there was a box frame around it which -- three supports all in one direction. It appeared to be quite a heavy support that probably slid down the pipe, and the structural steel was added to prevent the support from sliding down. We have concern with that because of the out-of-plane excitation of the pipe can impact that structural steel. So although it may not be a stability concern, it is the concern with the modification to the stability.

Those were basically the specific examples given in the motions for summary disposition. And at this point, I would like to ask CASE if they have any other examples of unstable supports that have not been

addressed by the Applicants or the Staff?

MR. DOYLE: Yes. There is one in particular. I was going to mention also the one you just mentioned of a gang hanger. There are about four or five in that one there. A specific is CC41-710-A63, which is triple-strutted and also has thermal movement. But that one again has the same walking instability, if we can call it that.

Then in addition to that, there is another one which is a single trunnion running perpendicular to the run pipe with a horizontal strut so that the delivery is eccentric to the line of action.

I have got some pictures of it here, I hope. Yes, here it is there. That's the one that I told Juanita over the phone.

MS. ELLIS: It's the one that was mentioned toward the end of the meeting on the 27th. It was mentioned specifically in the transcript.

MR. DOYLE: There are at least three of them in this set of drawing, related action; it's eccentric.

MR. POSLUSNY: Could we get the drawing number for the record.

MR. DOYLE: CC2-011-A63 -- can't read the last letter -- "K" I guess.

MR. BECK: I want to make sure I get the specific identifications, Jack, of the ones you just mentioned. I think we've probably got them, if they were read into the record before, but --

MR. DOYLE: Yes. I called Juanita -- oh, she didn't have the support numbers.

MR. BECK: That's fine, then.

Perhaps this is an appropriate time to comment. As Howard alluded to earlier, we're looking very closely at more than a few supports. There are a number of supports that from a stability perspective are not candidates for adequate analytical representation, and those supports will be either modified or removed and replaced with those which can be analytically represented.

That identification process has proceeded to the point where we have identified some hundred few-odd supports that we definitely are going to modify or remove. Included among those are the gang supports, for example, that we talked about earlier, a number of single-strut box frame supports.

Until we have done our QA on this list, I'm not going to mention specific support numbers, but let me just say that it's going to include that whole family that you've talked about earlier today and that have

been discussed on the record before.

We'll identify with specificity which ones those are, either in a letter in the very near future or as part of our comprehensive submittal in early April -- more likely in a letter prior to that time, just to make it specifically clear which supports those are.

I wish I were at the point now where we had done the QA check sufficient to lay the paper on the table and put it in this transcript. We just simply haven't gotten to that point yet. But I would certainly like the record to reflect the fact that we are doing this.

And it's very important, given that fact, Jack, that we get those specific supports identified to see whether we agree with you or not.

MR. DOYLE: I'm sure you will.

MR. TERAO: I've got one question about that support, Jack. I agree that there are some concerns to be addressed regarding the eccentricity of the loading which can induce torque to the pipe. But my question is, why is that considered an unstable support? I agree it's an unstable system or it's a system that is not accurately represented in the piping analysis. But why is that considered an unstable support?

MR. DOYLE: In the pipe stress run, the

load is delivered through the center line. Actually, the pipe is a line from node to node, and the loads are delivered along this line here. If you deliver a load along this line here (indicating) -- particularly there is a kick in this one -- then you can get rotation. You look at it that way.

MR. LANDERS: I think -- I would agree with Dave, that that is not a supporting stability problem. It's the problem of matching the support that's installed to the analysis that is done; that, in fact, the analysis doesn't represent the offset of the support.

MR. DOYLE: That is correct.

MR. LANDERS: I think that one is a different issue in my mind. It's not an instability issue; it's more a matching of the analysis to the as-built situation.

MR. TERAO: I guess the difficulty I have is, if we threw that in the stability hopper and it doesn't fit our definition, then I would say: What is your definition? Why is that support unstable? I agree there is a concern there, but I don't agree it's a support instability concern; it's a system instability concern.

MR. DOYLE: Yes, right. I want to keep

that one for --

MR. TERAO: That's an interesting concern, too, because the torsion that is induced into the piping may not necessarily be accounted for, even in the stress analysis.

MR. DOYLE: It also will have effects all the way down the line. Once you hit a --

MR. LANDERS: Why do you say that?

MR. TERAO: Well, from the equations --

MR. LANDERS: It's mx, my, mc-squared, square root of. You don't separate torque out. It's conservative but, in cases like this, it covers you nicely.

MR. TERAO: Okay. I agree.

MR. WALSH: With regards to the Applicants' comment about fixing some of these unstable supports, I'm curious if they're going to go to conventional designs or unique designs -- for example, the stability, bumpers would be a unique designs; whereas, if they had gone to a clamp for the fix, it would have been more a conventional fix.

MR. BECK: The modifications, Mark, will eliminate the question of stability for the supports that are on the list.

MR. DOYLE: Those will also be dictated

by space.

MR. BECK: The specific modifications will be individually dependent upon what's there and what is the most efficient means of doing it. So I can't answer the question specifically until you get down to the actual individual supports.

MS. ELLIS: I guess from a layman's viewpoint, John, the problem I see with that is that if it's a unique fix, we may be back talking about that unique fix next year about this time. We would like to avoid that if we can.

MR. BECK: The fix will be adequate. What more can I say?

MR. DOYLE: I think a unique fix is not the critical factor. It is if they address it. I mean, the fact that it's unique doesn't bother me.

MR. BECK: But it has to be adequate by definition, you know. We're not going to do anything that will leave room for argument. Let me just put it that way.

MS. ELLIS: Good.

MR. TERAO: That basically concludes my discussion on stability at this point. Maybe ask if there are any more comments to be made by either the Applicant or CASE?

MR. POSLUSNY: Would you like to take a 10-minute break?

MR. DOYLE: That sounds good.

(Brief recess)

MR. POSLUSNY: Dave had one more point for the record before we finished up with him.

MR. TERAO: The other summary disposition motion item that I had was ASME -- AWS/ASME on weld designs. But because that was formally submitted to the Board, our Staff response, that is the Staff position. So I won't be discussing that today.

MR. WALSH: I would like to comment on something you stated in your response to the Applicants' motion there, and it related to what the Applicants called the compensatory requirement. I addressed it at some length because it was in their motion and in the affidavit.

The compensatory requirements that they were referring to, they are not following. It was an attempt by the Applicants, I believe, to mislead the Board into showing that they are using a conservative value, that that number is not being utilized by the Applicants, which I believe you indicated the point is very relevant in regards to how the Applicants are handling these

motions of summary disposition and what their attitude is towards a safe design and proving that they have a safe design.

That's all I would have to say about your response on that.

MR. TERAO: Well, my responses were intended to address the technical issue. I really don't want to address that particular aspect of it. We recognize -- in fact, the Staff even asked the Applicant whether or not those compensatory requirements were still being followed. When we found out that they weren't, we just dismissed them, did not follow that portion of the summary disposition motion.

MR. WALSH: I believe that it's part of the NRC duty to require the Applicants, though, to be truthful and not attempt to make misleading statements, technical or otherwise. This was a misleading statement by the Applicants in an operating licensing hearing. The Staff should have followed up on it and found out why were they doing things like that.

MR. BECK: I would like to comment for the record -- John Beck -- that we came here this afternoon to participate in a technical exchange, not to be subjected to pejorative comments by CASE about misleading statements or anything else that you feel

should be discussed. And to that extent, I object, and I want the record to reflect that I object to that pejorative remark.

MR. WALSH: Well, maybe the Staff here can find out what the real problem is here as far as these misleading statements that were written in there that was not even relevant.

MR. POSLUSNY: We'll take a look at the transcript when we get it back -- exactly.

John Fair.

PRESENTATION BY JOHN FAIR

MR. FAIR: Yes. This is John Fair with the NRC Staff.

I have several of these summary disposition motions. Luckily, some of them are somewhat less technically complex than the ones that Dave just went over, so I'll try to go over them in as brief a summary form as I can.

The first one has to do with friction forces. And essentially the crux of this was that two of the Applicants' design groups an assumption in calculating the support lo ds. And that assumption was that for cases where the piping motion was small, less

than 1/16th of an inch, they could neglect this friction force in the support calculation.

CASE objected, that the Applicants -- and I'll try to paraphrase you -- did not have an adequate basis for making this assumption. So in order to resolve the issue, what the Applicants did was to essentially make two arguments. One, that via the code rules they had some additional reserve to accommodate stresses due to friction; and, secondly, these stresses would be fairly insignificant such that if they were added to the other stresses in the pipe support, they could still be able to maintain stresses and loads within allowables.

To support this, they selected a sample of six pipe supports which were supposed to be the ones that would be representative of the worst cases; that is, fairly short and stiff type of supports.

In the analysis of these six supports, it turned out that one support had an error in calculation of a bending moment. The Applicants agreed that there was an error in this calculation, went back and redid the analysis. When they redid the analysis, they changed the method of analysis for this particular support.

Now, CASE has pointed out -- they've argued with the method of analysis. And I essentially agree

that the Applicants didn't submit anything to justify that particular analysis assumption. And that assumption was that this was a bending load between an I-beam and a base plant. And the Applicants' revised assumption was that there was an even bearing between the beam and the base plate such that the negative portion of the moment would be taken out by direct bearing on the plate.

I didn't agree that the Applicants submitted anything to justify this assumption; and therefore, my position at this point in this summary disposition motion is that the two assertions made by the Applicants -- No. 1, that their evaluation showed the friction forces to be fairly small and not significant and, No. 2, that even including these forces, they were able to meet applicable allowables -- I disagree with both of those assertions.

Turn it over, if you have any --

MR. DOYLE: I have one thing, in the particular case of open section or the Y flange or an I-beam, particularly on short ones, due to a shear lag, it's actually on either a flange from which the member rests that probably will see the entire friction load.

And another point is, as anybody who has ever participated in a hot functional test knows, when you

start out with stress outputs that say you have a deflection of .060 in the specific direction, when you get to hot functional testing and you're now working with a fully coupled plant, it is not necessarily so.

For that matter, in many cases, the thermal movements of the pipe will go in a direction opposite of what you have anticipated due to impacts, et cetera. So the only time that I could ever see that the 16th of an inch could be considered as insignificant is, as I said at the hearings themselves, if I did an analysis and I wound up with a stress ratio of -- I'm going to say .6 -- and found that I had failed to include friction by engineering judgment and in so marking it on the calculation, I could write it off because I would be fairly certain that there would be no condition where the one-third increase in load that I would be receiving due to friction would affect the final safety of that particular support. But I don't think in any case would I ever allow it to just go totally unaddressed on generic basis.

That is about all I have to say.

MR. FAIR: Well, I would like to try to read into your comment here. What I hear you saying is that regardless of whatever analysis the Applicants did for the motions that they calculated from the plate

stresses, that you wouldn't agree with it anyway?

MR. DOYLE: Not as a generic solution. In other words, what I'm saying is, I wouldn't tell a group, "Forget about friction if it's less than a 16th of an inch, period," because there are instances where the 16th of an inch could be critical because we've all been involved with supports where we were running stress ratios of .9, .98, and we try to massage them as much as we can to keep them from getting stress ratios in excess of one. And in a case like that, there is a high probability that the inclusion of friction would then run it over the limits.

Additionally, on real short supports, if you have a 6-inch deep member and it's only a foot long, then you don't have a flexural member. If you're riding on the upper flange of the beam, the friction load is delivered to that upper flange and will be carried to that upper flange. It will never get to the lower flange. So that the effect on the weld particularly -- I'm mostly concerned about the effect on welds as opposed to the structural member.

And those are the two areas of concern that I have with just a generic negating of the losing of friction.

MR. LANDERS: Don Landers.

Jack, based on what you said, one of the hypotheses was that designing supports to know your ratio of .9 on the allowable, do you find that is a common practice in a design process, or is that the situation where I'm trying to now reconcile something I've found in the field?

MR. DOYLE: That's where I would see it.

MR. LANDERS: Okay. All right. Therefore, in a design process leading up to that point, is it reasonable in your mind to establish some cut-off point on consideration of displacement versus friction loss?

MR. DOYLE: It has been done in other plants.

MR. LANDERS: Okay.

MR. DOYLE: And I feel easy with it mostly because they don't have a number of other elements that are neglected. I know what they've included, and I'm satisfied with what they've included. And then comparing the 16th -- if I don't feel comfortable, I'll put it in, regardless of what their document says. And generally most places I've been -- well, let me say at least half the places I've been, they've included it regardless; and half the places don't include it it. I've been in places that I will

include it every time regardless of what they say because I'm afraid of their other numbers.

MR. LANDERS: I just wanted to clarify that there is a difference, I think.

MR. DOYLE: Yes, yes. Many times we'll get as-built loads, and we have a stress ratio and we take the loads here and we come up with a factor and we multiply it, and we say, "Well, it's .8, so that's as far as we'll have to go."

MR. WALSH: I have a few comments in regard to frictions, more or less to do with the weld. My concern now with this is how the Applicants handled the analysis. Now, we have found that they change their assumptions, and they don't consider pressing forces on that weld on this particular support. But this support has been modified. I think we've seen a drawing indicating it was unstable. They got rid of the friction forces. But the supports where they have now qualified them, because they don't consider this compression forces, that hasn't been addressed by the Staff.

And the Applicant is coming up with a new plan. And in that plan, they should discuss how they are going to handle these calculations where they don't consider the compression forces in welds. You might say

it's a new issue, but it came out of this particular motion and was not one that Jack and I discovered until we saw their calculations.

Jack and I, while we were down there, we didn't have much of an opportunity to review the calculations. Therefore, we hadn't really said, "Well, this is a generic problem they have down there." But it appears that that's how they passed a lot of their welds, based that they were doing this on motion for summary disposition.

MR. FAIR: Well, I disagree with your statement that the Staff hasn't pursued it. I think we've asked the Applicants at least twice, in two different meetings, to provide both a justification for that assumption and, secondly, to clearly spell out what their criteria is supposed to be for that evaluation.

MR. WALSH: I'm again sorry to imply that you didn't pursue it that way. My indication was that there has been no effort to go back and look at other calculations to see what was done to pass the weld or show that it was acceptable by the Applicants on other supports, as well as in regard to some qualifications.

I'm not that familiar with their welding procedures, but I believe they are allowed a gap between flange and bearing, and it would not be in violation.

I'm not sure what the gap is. There is an allowable gap and that's why I feel it's not addressed in any code we have on compressor forces on the welds.

MR. FAIR: Well, I guess I don't agree with that statement either. I believe that there are provisions in the code that do address these situations. And they're extracting from the AISC but in subsections. Appendix XVII they pulled out the criteria for bearing joints on columns. And there are a couple of criteria in there -- depending on whether you're looking at great bearing stress or bearing stress -- but the critical point in the specification is that you have to have finished-to-bear item in order to take credit for bearing stresses between the beam and plate.

And that specifically is the question I asked in the meeting a couple of meetings ago, whether they have any justification for that assumption and did they specify this joint as a finished bearing joint.

MR. WALSH: During construction?

MR. FAIR: That's correct.

MR. WALSH: All right. I agree.

MR. POSLUSNY: Other comments?

MR. FAIR: I guess I would like to now try to seek a clarification from CASE on their response to the Applicants' summary disposition motion. And it

has to do with the appropriate allowables for the evaluation of the load combination and considering these friction forces.

There were two areas in which CASE took issue with the Applicants' analysis allowance. In one case, it had to do with what they called their yield share criteria for the base material of being .6 S-sub-Y as opposed to .4 S-sub-Y in the AISC specifications.

And the second additional comment that CASE had, they were concerned that the Applicants weren't using the provisions of Regulatory Guide 1.124.

Now, I would like clarification as to what was being argued in this particular response by CASE as to whether the Applicants are required to go to the AISC specification or that the ASME code in conjunction with the Regulatory Guide is inadequate. And it was not clear to me.

MS. ELLIS: Without seeing this, I think we would almost need to take a look and get back with you on that.

MR. WALSH: Do you know what page that was on in the affidavit?

MR. FAIR: I don't know exactly what page it was on, but in terms of the difference between the ASME code -- I guess this goes back to some of the

original arguments, that there is a provision in the ASME code, when you're looking at stresses due to the strained, free, and replacements that allows you to increase the normal allowable stresses, on top of that, the Staff has a regulatory guide that puts some restriction on that.

Now, it wasn't clear to me whether you were arguing that the ASME code criteria, coupled with these restrictions, were inadequate and therefore you needed to go to the AISC which was a little bit more restrictive.

MS. ELLIS: I think we would have to look as that and get back. I think we really need to take a look at that summary disposition and see if we understand exactly what we're talking about.

MR. WALSH: You're saying why didn't we refer back to the AISC code?

MR. FAIR: As opposed to the ASME. I did bring copies of your submittal also if you wanted to take a look at them. I can pull them out.

MS. ELLIS: We'll do that at the break.

MR. FAIR: Okay.

MS. ELLIS: I'm talking about being just strictly off the top of my head. I may not know what I'm talking about, but I kind of think that if I recall

the particular answers, that what was being said at the time was that you do have to address 1.124, but if you didn't have to do that, then the other requirements, there are other requirements that still would have to come from -- I believe that that's right, but I would have to check back and see.

MR. WALSH: But you're asking why didn't we look at the AISC code over the ASME?

MR. FAIR: That's correct.

MR. WALSH: I don't remember that portion.

MR. FAIR: Any other questions on the friction forces?

The next issue I had was backing values from the NSSE. This started out as a very narrow issue. I believe the original issue had to do with a particular support and the fact that for some reason, the OBE loads came out larger than the SSE loads which would contradict logic since the SSE is greater.

In addressing it, the SIT wrote in their report that there were no problems with the loads, the OBE load being greater than the SSE load because of the damping specified for the building. And they put in two values of damping which were pointed out to be inappropriate damping values per Guide 161.

The Applicants put a response in, in which they attached the computer run of that particular stress problem when the issue was raised. In reviewing that particular analysis that the Applicants have put forth, it appeared to be a different -- later run than the original run that was in question by the SIT evaluation.

Now, Dr. Chen had retained some documents for comparative purposes that he originally had reviewed during the SIT inspection. And when I compared the two documents, they were different, although in reviewing the input specter, they seemed to be fairly similar but they were not exact. Therefore, I was unable to draw any conclusion on the original analysis that was reviewed by the SIT team since the documents did not have anything that clearly identified which damping was used in the specter input.

And at the point I am right now with the Applicants, I'm awaiting their response. And I believe that they are gathering all historical documents associated with that particular stress analysis problem to clearly define which dampings were used and when they were used.

MR. DOYLE: Essentially then, this is still an open area?

MR. FAIR: Yes.

MR. DOYLE: That's all.

MR. FAIR: Any other questions?

MR. WALSH: No.

MR. FAIR: The next issue I had, had to do with the section properties of the tube steel members. And it came up in two separate -- I call it two phases of the issue. The first phase had to do with the fact that the Applicants had three different texts which they were able to pull out the member properties and all three of them were different. And these different properties were based on what was the assumed corner radius of the tube steel sections.

The SIT originally reviewed this and found that they had no problems with what the Applicants were doing; however, CASE also brought up a point with the corner radiuses that could affect the weld thread area, depending on which section that you assumed in the analysis.

Now, the Applicants have stated that their tube steel sections conformed to the Eighth Edition of the AISC specifications. And therefore, if this is the case, there would be only one set of property values that could be unconservative from the point of stress calculations in the member itself. Also, the Staff had evaluated the concern with corner radius and weld thread

area, and that was Mr. Tapia who had submitted an affidavit on this subject quite awhile back now.

In reviewing some work that was going on with Cygna, I ran across a response by the Applicants to a Cygna question when calculating thread area for welds from these tube steel sections.

It appeared that the Applicants had changed the criteria on the method of calculating the thread area from what Mr. Tapia's affidavit contained. And therefore, the last meeting, I asked the Applicants to clearly define the criteria that has been used for this calculation, the basis for it, and all changes to the criteria. Therefore, this particular aspect of the issue is still open, waiting for Applicants' response.

As far as the member properties, CASE has argued that certain of the tube steel sections used by the Applicants conform to the Seventh Edition of the AISC specification which essentially gives lower member properties than the more recent Eighth Edition.

I am unable to understand the bases of why CASE thinks that the Seventh Edition property members are more applicable to the tube steel sections of Comanche Peak. I would like to request, if there is some reason or basis on which you feel that there are tube steel sections with corner radius and member

properties that are more in conformance with Seventh Edition than Eighth Edition, to tell me.

And the reason I'm asking is, as part of this review, I did go personally through the facility looking at the thicker tube steel sections, the ones in which the corner radius would make the greater differences; and in all cases, it appeared to me that the corner radius on tube steel sections were approximately the 2T assumed by the Eighth Edition.

MR. WALSH: I can respond to that.

Early on in the design, Grinnell, I believe, had utilized some of these tube steel shapes. This was back in '78. I believe at that time the Seventh Edition steel was being used. And that's why they were using Seventh Edition member properties. The steel had been purchased prior to the change-over within the steel industry. Those members would have the Seventh Edition radius which changed based on the size of the member.

I forget at what time the steel industry went to a 2T uniformly for all steel members. But the present steel they're purchasing, assuming that it has been milled since like 1980, will have the 2T. But if they go out and they use steel that was purchased prior to the change-over, that steel would be with the Seventh Edition.

And there is no indication on the drawings when that steel was purchased. They could look it up. But the drawings, a lot of them were at issue in 1978, that indicate the steel was purchased in that time frame.

MR. FAIR: Well, is there a specific reason or document that tells you that there was actually a difference in tube steel, a change in the actual properties of the tube steel when the AISC changed their specification?

And the reason I ask is, is because the material specification in both instances would allow you to go up to what was assumed in the Seventh Edition, a value of 3T, and whether there was a change in the code on its assumptions or whether there was actually a steel change between those editions.

MR. WALSH: There was a steel change in the milling of the steel, fabricating of the steel. That's the change. There was a physical change. That's why the member properties changed. For someone to be in compliance with the Eighth Edition, they have to be using the 2T. The Seventh Edition varies.

And someone buying tube steel back in '78, you know, they would be buying that steel -- larger sections with the 3T. That's where the concern was.

The Applicant, you know, they can go out there and measure the corner radius of the tube steel member or verify that it was milled to the dimensions for the Eighth Edition. But if it's old steel, the Seventh Edition would have the larger tube steel members, 3T radius.

It's not included on the mill test reports. It's a member property problem. It's like a Y flange, dimension for a Y flange. You get a mill test report, it's not going to indicate what the dimensions are on the Y flange.

MR. FAIR: Were there any cases at the facility where you actually saw some tube steel sections with corner radiuses of 3T?

MR. WALSH: I never went out there and measured it, no.

MR. LANDERS: What I'm hearing is that the basis of this is that prior to the Eighth Edition issue of the AISC, industry made tube steel in a certain fashion; and after the issue, they made it in a different fashion. Is it possible that the Eighth Edition reflected what the industry is doing? Do you know that to be a fact or not?

MR. WALSH: No. I do not know the reason for the change in the edition. I know the Applicant --

this is a point I think I -- or I should have made clear in the affidavit -- is the Applicant used a Lefland or Welded Steel Institute, tube steel properties, with no justification if they were higher values. That's the one with the 1T. And they didn't bother to look at -- this is the generic type of thinking that problem -- didn't realize that maybe that steel doesn't exist, when they used it, didn't match the properties of that to what was being out in the field.

And to be more specific, when I was working in che industry, this became a concern. We were using three different member properties. And it was around January, Landley Hoghouse decided to go to the Eighth Edition instead of using this Welded Steel Institute values.

It was John Finneran that informed me what was actually out in the field, and this never came about when I was working. When I left in June, I still had not heard what steel are they using out there. But the problem of going a whole year using these higher values and not recognizing that they have to reflect what's in the field is a design problem that should be looked into more than just -- I was working on the STRUDL. That was not my responsibility. I wasn't designing those forces. I just saw that type of problem.

MR. FAIR: I understand what you're saying. We're still trying to understand what are the appropriate properties to be using at this facility and why -- I think everybody agrees that the other set of higher properties were not the appropriate properties because the Applicants switched back to the Eighth Edition and did some reevaluation.

MR. WALSH: I believe the properties that should be used for steel milled after 1980 would be the Eighth Edition for all three type supports groups, not just PSE. ITT and NPSI, they were putting steel in and purchasing steel after 1980. That's when the mills were doing the, you know, producing properties conforming to the Eighth Edition, the numbers they should be using. It's as simple as that, I think.

If that's what's out there, Eighth Edition, they should be using Eighth Edition. And if there is no Seventh Edition used on that plant, then they'll just use Eighth Edition everywhere.

MS. ELLIS: Does that answer what we were --

MR. FAIR: Yes, I guess to the extent that it can be answered.

The fourth issue that I had, had to do with safety factors. I think in the original meetings that we had on the summary disposition motions -- that is,

between the Staff and the Applicants, which was quite awhile back -- I had come to the conclusion that this was a fine analytical study, but it was not particularly useful on this plant to resolve the issues because it was a bunch of generalized types of studies which are in line with some other things that have been produced in the industry. But other than that, that's about as far as I took the review of that particular issue.

MR. POSLUSNY: Comments, anyone?

MS. ELLIS: No.

MR. FAIR: The next issue that I have has to do with the effects of gas or the bolt hole tolerances. I would like to defer that into the section with the Richmond inserts.

MR. DOYLE: Could I interrupt for one minute?

MR. FAIR: Certainly.

MR. DOYLE: What about the over-sized holes for the bolts going into the Richmond bolts?

MR. FAIR: That's what I just asked if I could defer.

MR. DOYLE: I tell you, I'm foggy.

MR. FAIR: And with that, I'll just leave it open as to whether anybody has any questions, comments, et cetera.

MS. ELLIS: No.

MR. WALSH: Nothing we say is going to make . . .

MR. FAIR: The next item I had, had to do with generic stiffness. And simply put, the concern is that the Applicants used a set of generic stiffnesses in the pipe stress analysis, equal assumptions; however, they used a different criteria to design the supports which was the deflection guideline with the load output from the pipe analysis.

These two assumptions are not exactly the same; and therefore, you have a concern as to whether there is a match-up between the two analytical processes. We also, the NRC Staff, had a concern with this and with the Applicants' results in their summary disposition motion. Therefore, we requested that the Applicants go back and reevaluate piping systems, looking for cases where the supports could be particularly soft. And the basis for looking for these supports being particularly soft had to do with the supports that had the lowest loads from the piping analysis and what you would expect based on a load deflection criteria to come out with softer spring rates.

At this point, the Applicants had given us a

screening criteria for performing the evaluations, but they have not given us any results of these evaluations. So at this point, it's still open.

MR. WALSH: Do you know when you'll be getting this?

MR. FAIR: I have no idea.

MR. WALSH: I haven't seen that screening criteria that the Applicants were using.

MR. FAIR: I believe that was in the September submittal, the final large submittal that the Applicants made to the NRC Staff.

MS. ELLIS: Okay.

MR. DOYLE: In this evaluation they're doing, are they considering, particularly in the containment where we have these tube steel frames that are supported on A36 threaded rods into Richmond inserts that are effectively a bearing type connection and also exhibit, just from the tests alone, the very high, obviously, shear displacement which will tend to soften the support?

MR. FAIR: I would have to defer that question to the Applicants because I don't know what specific supports their screening criteria is going to yield for this reanalysis effort.

MR. DOYLE: It would be well if they

include some of them, I would think.

We're most concerned in that particular type of a screening effort is not a soft system, it is a random system. In other words, if we have a system that's all soft supports, that doesn't present really critical problems. It's where you have very stiff supports and intermingled you have soft supports. Is this the type of thing you're looking for?

MR. FAIR: I think that the screening criteria is supposed to be looking at both cases. Now, the Applicants have agreed with what you've said, that the case that's likely to be a problem is a soft support in the middle of two stiff supports.

MR. DOYLE: Yes, right.

MR. FAIR: However, the other case I agree is not a problem. If the supports are all soft, that may very well change the total load input.

MR. DOYLE: Well, that's true, particularly if it goes more than 10 percent below generic.

MR. FAIR: That's correct.

MR. DOYLE: What I probably should have said is less of a problem than you could run into with two million pounds an inch on each side of 50,000 pounds an inch.

MR. FAIR: Well, I guess my opinion is, either one could be a significant problem.

MR. LANDERS: Again, it's this system concept of the fact that we have something attached to the end of that pipe. A soft one could in fact be a --

MR. DOYLE: That's right. I do what I tell everybody not to do.

MR. FAIR: I guess -- Dr. Chen just reminded me. It appeared that -- at least your position was that this 10 percent number which was an argument which was put forth by the Applicants, that if they were within a factor of ten of their assumed generic studies --

MR. DOYLE: Yes. Right. Single order of magnitude, I found no problem with that. Many of the places I have worked, as long as you're within one order of magnitude you don't even get concerned about it. If you go beyond the first order of magnitude, you go talk to the pipe stress people, What we call "confirmation required." You get confirmation that you're -- I forget what that is -- but they'll go so much over the first order of magnitude before they get excited.

MR. FAIR: I guess I would agree. It depends on what the generic stiffness was in the first place.

MR. DOYLE: Yes.

MR. FAIR: If it were stiff enough, then an order of magnitude lower would not be a big problem. But if it were soft to begin with, then it may be a problem, even less than that.

MR. DOYLE: Right.

MR. FAIR: Any other questions, comments?

The next issue is U-bolts intending to act as one-way restricts acting as two-way restraints. Hopefully I said it so that it's clear which ones I'm talking about.

In this particular case, the Applicants had U-bolts on rigid frames where they intended them to act in the strong direction of the U-bolt and assumed that they would take no load in the weak direction of the U-bolt. The basis for their assumption was that these movements in the other direction were so small that there was enough gap in the U-bolt so that the U-bolt would never see a load.

They have said that they had identified approximately 70 cases, gave the deflections output from the computer analysis of the piping, and identify eight cases where they expected the deflection of the pipe to exceed this assumed amount. They did some analysis, some seismic reanalysis of a couple of cases and concluded that stresses in loads would still remain

below allowables.

Now, in order to make this conclusion, the Applicants had to do some actual physical testing of a couple of U-bolts to come up with a load rated allowable that was higher than the original manufacturer's allowable.

In confirming whether the Applicants' assumptions were correct, I went out to the field to measure the gaps on these particular eight U-bolts where the deflections were the greatest and was unable to confirm the Applicants' assumption that a gap existed in this particular direction, that the U-bolt didn't intend to take load.

And therefore, I disagreed with the basis of the Applicants' analysis. The Applicants went back and did a reanalysis and submitted it to us on the September submittal. The reason the Applicants did a reanalysis was because the original analysis included seismic only with the assumptions that the gap existed and was larger than the thermal load; and therefore, the original thermal analysis was still valid.

In the reevaluation effort that was given to us, it was unclear to me that the Applicants had gone through and evaluated the new loads on the supports, including the U-bolts, to determine whether they would

be within acceptable limits. And I requested that the Applicants go back and reverify this.

The bottom line conclusion that I had out of this at this point is that the Applicants originally had no basis for making the assumption that these U-bolts provided no lateral support.

The issue as to whether there is a problem with the existing U-bolts is still open.

MR. DOYLE: I don't really believe we can comment on that one until the issue is resolved. At least I can't.

MR. FAIR: Any comments from --

MR. BECK: No. We understand your position, John.

MR. FAIR: The last issue is Richmond inserts. And there were several issues with Richmond inserts, and I'll try to group them as the Applicants did in their summary disposition motion into three more general categories.

One had to do with the actual capacity of the Richmonds and the basis for the allowable on the Richmond. Another category of issues had to do with the design assumptions you used to model the joints between the Richmond inserts and tube steel connections. And the third general issue had to do with bending loads on

the bolts that were going into these tube steel connections into these Richmond inserts.

As things stand right now, we have a concern with the Applicants' modeling assumptions for the Richmond insert tube steel connections. They have done some evaluations in their summary disposition motions and at our request have gone back and looked to see if these evaluations covered all cases that existed at Comanche Peak.

And they have identified some cases where they had to do some additional analysis. There was also a part of the summary disposition motion that was very confusing to me, and that had to do with the Applicants' discussion of when they released what we'll call the torsional moment and when they released the bending moment along the tube steel.

It is my understanding at this point that the position is that they always model the bending moment as a released condition and that, depending upon the designer's judgment, the torsional moment would be either released or fixed. And I'm awaiting a clarification on that particular point and a particular discussion in the Applicants' affidavit.

If the case is that on the frame structures that the Applicants in some cases assumed that these

moments were fixed, I don't think the Applicants have presented enough basis to justify that the stresses in the tube steel members and the inserts are adequate.

My opinion is that the appropriate modeling assumption is assumption for these frame structures. Now, I distinguish between the frame structures and the ones on which the Applicant has a long length of tube steel with the inserts along the length of the tube where the only method of stability is to take reactor loads out and forget them. And this indeed in my opinion will occur. They will eventually react the load out in torsion.

However, the Applicants' analysis currently has identified the problem, especially with cases where the insert is offset from the center line of the tube steel members; and therefore, you get a very short couple to react the load out, and you primarily have to take the load out with bending of the bolt.

The Applicants had identified some items in which they calculated fairly high bending stresses in the bolt and had said that they were going to modify or correct these situations. However, there are still some concern between the Staff and the Applicants on the evaluation criteria for which bolts to be looking at; that is, the Applicants developed a formula based on the

results of their finite element analysis. It was a formula which you won't find in any of your standard industry code. And at this point in time, we're not in agreement yet with the Applicants that this evaluation of criteria was adequate.

Specifically, we've asked them to go back and evaluate the results of their evaluation of these tube step members for bending stresses and loads in the Richmonds, considering assumptions or field installation procedures such at bolt hole angular and bolt hole gaps, et cetera, and determine that their evaluation conservatively considers all those cases.

And that's at the point where we are on Richmond.

MR. WALSH: Will the NRC be using any of Cygna's questions or comments that they recently submitted to the Applicant, in the the NRC's evaluation?

MR. FAIR: I will be reviewing them, yes. I have not really had time to understand the basis of their comments or what points they were trying to bring out. I've seen their comments.

MR. LEVIN: John, with respect to TUGCO's development of an interaction formula or method of dealing with the bending and tension and shear in a bulk, for example, was your concern with that approach

conceptually or the value, for example, of the acceptable interaction, whether it be 1.0 or 1.75 or whatever the number may be? I mean, given that there is not ready guidance and codes on these kinds of things, I'm trying to find out if --

MR. FAIR: That's specifically my concern, was the fact that TUGCO had developed a higher limit than you would normally use for bolts. The basis for this was two-fold. One was the fact that their finite element analysis showed that the method that they were going to use for these calculations of bending was conservative compared to the finite element analysis, and they were going to screen the bolts at the field based on the more standard type of calculations and not the finite element analysis.

The second basis in their affidavit was the fact that you were looking at bending in these bolts and that the normal allowables were strictly in terms of tension and shear, and that if you go to the ASME code or Structural Steel Code, they generally allow higher stresses in bending than they do in direct tension or shear.

So, yes, it's the basis for the increase in the allowables.

MR. LEVIN: So it's more toward the

allowable versus whether or not one can add bending into the interaction equation, per se -- I mean, you would like to see the bases for the specific value, the allowable, as compared to a conceptual problem with whether bending could be included in the interaction.

MR. FAIR: Well, since we have the situation, it has to be included, some method.

MR. LEVIN: Okay.

MR. POSLUSNY: No other comments?

MR. DOYLE: Are you going to get into the stiffness of those bolts?

MR. FAIR: That was the end of my comments. I'll leave it open with you.

MR. DOYLE: Yes. Well, I have two major concerns within A36, and one of them is they're not recommended for dynamic applications. And the other, many of the supports, the distribution of shear and tension -- tension being the lesser of the two -- but the distribution of shear is based on the concept that all of the bolts are in contact equally. And particularly for some of the ones at Comanche Peak, there were a multiplicity of bolts -- it could be 12, 15, 18 bolts in a single frame. And because of the nature of the beast, we know that there is no time when we will have all of those bolts actively engaged. So

therefore, some of the bolts are getting higher load than would be indicated by the STRUDL analysis. And I was wondering whether or not that was looked into.

Additionally, there were other supports -- I had a couple of them in my summary disposition, one of them being a Class 1 support -- where you have a single piece of tube steel hanging off two Richmonds and then a cantilever hanging off of that. Effectively, the bending of the bolt renders the entire support far softer than the analysis would indicate because the analysis indicates that those two points are literally fixed.

Beyond that, that's the only two comments I have in regards to --

MR. LEVIN: Jack, I have a question with respect to that comment. Is there anything with regard to these particular connections that -- I mean, I think what you explained might generally be true for bearing connections, but is there anything that would make that particularly different here on this plant?

MR. DOYLE: I don't understand.

MR. LEVIN: In terms of the sharing of loads between bolts.

MR. DOYLE: Yes, because the entire support -- before the entire bolt pattern becomes

effective, a number of bolts are going to have to displace perhaps as much as an eighth of an inch. So in addition to the support stiffness factor, you have to take the ratio of the sum of the --

MR. LEVIN: Am I to understand your concern being more with the impact of that on softening of the system or the fact that there may be a different load distribution to bolts?

MR. DOYLE: There would be a different load distribution, depending on the total load of the support. But the tests indicate that the shear displacements of those bolts were rather horrible. For that matter, when you get up around a 16th of an inch, you've almost reached the limits of the allowable for the particular bolt and shear.

MR. LEVIN: Is that particular tests or tests in general?

MR. DOYLE: No. That was Applicants' Exhibit 142. Anyway, they did a test at the request of --

DR. CHEN: It was 142, it was another --

MR. DOYLE: Oh, well, at any rate, there was a test done that showed very high displacements.

MR. LEVIN: I'm trying to understand, though, if that was in a specific test of a particular bolt or that was a general trend noted in the entire

testing program.

MR. FAIR: That was four particular -- as I recall, four particular bolts, tested out at very large deflections at the bolt allowable load in shear. At the last meeting I asked an explanation of those particular results.

I, too, would like to ask the question, now that the subject has arisen, as to the concern on the sharing of loads on bolted connections, as to whether this concern is particular to the Richmond insert, tube steel connections, or whether it's in general for base plated anchor bolted connections?

MR. DOYLE: No, because most of the anchor bolted connections are friction. They prescribe torque. And a friction joined connection, until you reach separations, there is no shear involved, although they may be analyzed as if there is. In fact, there is none.

On this tube steel, again we're into a unique design. The friction is indeterminate because the torque is indeterminate. They can't torque down like they can on a base plate. On a base plate, you can take it up to whatever is required according to your particular specification or the manufacture's recommendation.

So it is only for those connections which are essentially either indeterminate or vary in type but not for base plates in general.

MR. FAIR: I would still like to pursue it a little further. In terms of general bearing connections, is it your position that it should be the practice to analyze each individual bolt separately within the tolerances of the gaps around the bolts?

MR. DOYLE: I'm not sure exactly what you're saying. However, if there was a practice in place that would have assured the lesser gap, then of course we would have less problem. But currently the condition is such that you could actually have to deflect some of the bolts an 8th inch before the remainder come into action.

MR. FAIR: Well, I guess I would turn it around. If they would be used with the AISC which are considered standard hole sizes which would cut the gap effectively in half from an 8th to a 16th, would you still have a concern?

MR. DOYLE: Myself, yes.

MR. LEVIN: Well, Jack, then I guess -- I was interested in that same line of questioning, John.

I think people recognize differences between friction and bearing connections. So if we get to just

looking at bearing connections, what I'm interested in is understanding whether or not the concern is, in fact, with industry practice, or there is something that has been done on this project that particularly would exacerbate it.

And you're saying that going in the direction of a larger hole size would, but I am interested in your thoughts on the point that John just make on the 16th, and the fact, you know, that that's a tolerance adopted by AISC, and --

MR. DOYLE: Yes. But you've got to remember that AISC, for the most part, wherever they do have large shears like at base plates and things, they put in shear keys, so they're not relying on the bolts to take shear load.

In the remainder of the structure, except under the new SEAH codes, they took an equivalent horizontal shear into the building and therefore that really structures subject to dynamic conditions. So here we have a unique case where we have the supports suspended off of tubes which are bolted to Richmonds with an indeterminate friction qualification; and yet, every bolt in the pattern is treated as if it is receiving equal load.

And I marked under a number of the supports

out there where there were problems even at that, bringing the bolts into qualification. And had they been done as the real world, which you would never do, you would probably find that several of those bolts went way over the allowable.

MR. LEVIN: Is there any indication that the connection doesn't perform?

MR. DOYLE: The fact that it's A36.

MR. LEVIN: I mean any experience. That's what I'm after.

MR. DOYLE: Well, that's what I'm saying. I have never seen that particular type of support used anyway, except for perhaps, you know, a coathanger or something. Every plant I have ever been in either used embeds or surface-mounted plates or through bolting or something of that nature, and I can't recall of any that weren't using friction type joints.

MR. WALSH: The other point that I would like to make is with regards to the AISC code -- and I don't know if it was submitted in our answer on gaps or on the Richmond insert -- but the AISC code, as far as oversized holes and base plates, was written with the assumption that you have got a heavily loaded column, that column has sufficient press, of course, on that base plate that you don't even really need anchor bolts,

you put the anchor bolts in just the same.

And I believe it was one of those motions that the Applicant was relying on, saying, "Well, we could have made the holes even bigger."

That is not the case. If that's how the Applicant really feels about it, it's either a question of judgment again of the Applicants to rely on that type of premise.

MS. ELLIS: I would like to ask you, Paul, I believe that the test, wasn't that attached to an affidavit of yours? I sort of believe it was, but I'm not positive about that.

DR. CHEN: I don't remember, but I think -- let me look through my, quote, boxes unquote, and I'll get back to you on that.

MR. DOYLE: One of the major problems with that particular type of connection, again, if you uncouple it and you look just at the shear, eventually all the bolts in the pattern will share the shear. But you've got to recall that some of the bolts at the point you get to where the load is fully distributed, have higher shear loads than was anticipated. Now you must add the tension load and also the interaction of bending.

But I don't recall having seen -- the closest

thing I can recall to that type of a situation is where some people were using Unistrut. I disagreed with that strongly, too, because again you're getting to bending of the little ears.

MR. LEVIN: What about a situation where it's just simply a base plate?

MR. DOYLE: How is that again?

MR. LEVIN: Just simply a base plate configuration and a bearing connection. It seems to me implicit in the concept of that type of connection is the fact that there will be some redistribution of loads between bolts and in the concept of that --

MR. DOYLE: Yes. But first you have to displace two or more of the bolts to the point where the remainder become effective.

MR. LEVIN: Yes.

MR. DOYLE: When you do that, you have got shears or shear stresses in some of the bolts that are considerably higher than you had anticipated. Those could be the bolts which also are taking the majority of the tension, if you have a couple in that direction. So now you're well beyond the allowables established for that particular bolt.

MR. LEVIN: Okay. I understand your point. That's why I asked the question before about the

question of experience. And it seems to me that people as a matter of practice have accepted that, possibly supported by the fact that those types of connections do perform in a certain way. And I was curious as to how you believe that would impact the overall integrity of such a connection.

MR. DOYLE: Like I say, I haven't seen it done. All the building columns I worked on, if you receive 30 percent of the friction, then you go to shear keys in the diretions it's required.

MR. LEVIN: Okay.

MR. POSLUSNY: Are there no more comments on Mr. Fair's items?

If we can think of anymore, we'll take a quick break.

(Brief recess)

MR. POSLUSNY: If we could get started. Okay. Ready.

Paul Chen is going to continue.

PRESENTATION BY DR. PAUL CHEN

DR. CHEN: Okay. I have four summary dispositions. The first one has to do with forced distribution and axial restraints.

This summary disposition is concerned with dual snubber or seismic restraint types supports and forced distribution and lug type supports. The concerns relate to piping stresses and loads on the supports.

To expand on bit on that, the concern is related to the rotational restraints offered by the dual type supports, the effects of offset masses in the piping analyses, CHEME stresses and trunnion type supports and local stresses due to the attachment. I would like to ask at this point whether or not that covers the concerns that you have?

MR. DOYLE: There are a couple of other points. One of them is particularly in reference to snubbers. Snubbers are generally set for a specific -- they're acceleration sensitive, so they're set for a specific g loading -- for example, .02 g. Any gain type, whether it's 2 or 8, like they've got in the upper lateral restraint, like of a snubber arrangement, you're never going to get exactly, precisely .02.

And most often, they assume the total load on

one support. If they can't make it by that, they'll go to 75 percent. And I think the manufacturers by test have shown that if you go below 60 percent of the total load on one support, then you're going to find yourself in real trouble because the two snubbers will not lock up simultaneous.

The second problem is, when you have two snubbers or two struts and they're attached to literally different frames of the same frame system but independent frames -- for example, say you had a cantilever off the wall a foot long, same tube steel coming up off the floor ten feet long, you have a differential in stiffness which will affect the loading distribution. And this, of course, is why most places like to try to make it pass with a total load on one. If you can't do that, many of the places I've worked at, you can arbitrarily adopt a 75 percent. To get down to .6 and below, you have to get confirmation required type of thing.

So besides the thermal rotation, if it's on a horizontal run above the "Y" axis and "X" number of ratings or what have you, have those two considerations, you should also go into snubber loadings.

MR. LANDERS: Jack, is there anything in the procurement process that would alleviate some of

that concern?

MR. DOYLE: With respect to --

MR. LANDERS: With respect to procurement of the snubbers and specification thereafter.

MR. DOYLE: I don't think that, to the best of my knowledge, they never can get two snubbers to lock up precisely at the same time. They come very close, and it is a precision piece of equipment. But due to the fact that you are dealing with an acceleration and the reaction of the snubber to that acceleration will vary so that the snubber, within the limits of human capability, I don't think they could ever get two snubbers to lock up precisely at the same time. The result is, the (unintelligible). And once you start getting rotation and the acceleration picks up, the other one will lock in which is why in many of the plants I've worked you are allowed to go down to .6 of the total load but I don't -- it's possible, but I don't think I've ever worked at a place where they divided 50/50.

MR. LANDERS: That was precisely my question. Going down to .6, do you know if in those instances there are very specific procurement requirements with respect to those two snubbers, that that was defined to be accurate?

MR. DOYLE: We only buy them in pairs. In that respect, yes, if you have a dual snubber set-up, you should buy them in pairs. But as far as CPSES, I have no idea what their procurement did about that. If you don't buy them in pairs, then you're going to have even more problems.

MR. LANDERS: Okay.

DR. CHEN: The Applicants' motion for summary disposition does not address these two concerns that you brought.

Additionally, the arguments presented in the summary disposition are contrary to what the Applicants had committed to two years ago to the SIT team. The motion contains analyses for (unintelligible) piping system which supposedly show that if the rotational restraints of a dual snubber installation are considered, it has very little effect on the piping stresses, but that support load will increase by a factor varying between 2 and 3.

The Applicants proposed new allowables and these new allowables are based on the -- assuming that the rotation is a secondary effect and that increase on loads can be treated as secondary loads, basically what they were proposing was that the allowables be increased by three times what the allowables were. And we would

disagree with that. The analyses showed that the loads increased by around 2 and 3; the proposed allowables increased by 2 or 3. Basically what they were saying is that there is no problem, and I have a problem with that.

The load type supports -- the information is based on inspection of 29 supports -- showed that in each instance there were always two lugs which were fairly equidistant from the support structure. A maximum distance between the structure and the nearest lug was about 1/16th of an inch.

They then did elastic-plastic analysis of a lug attached to a pipe and displaced the lug by 1/16th of an inch and said that it was also indicated that the plastic deformation localized. I cannot accept that analysis because it does not address what happens on the separate loadings. This analysis just shows what would happen in the case of a one-time loading.

Some of these comments were passed on to the Applicant in a meeting we had about a month ago, and I haven't heard anything back from them as yet. Basically this item is --

MR. DOYLE: Also, in a substantial earthquake, you could have a fairly large number of zero crossings. So the fact that you're displacing a

particular lug a 16th of an inch -- well, coming back to that 16th of an inch, again we end up somewhat similar to the shear on bolts if we're displacing a 16th of an inch. I wonder what the BLR would have to say about that in reference to the pipe or what does anybody have to say about that?

DR. CHEN: They prepared some results of analysis for piping system when one lug was loaded -- or, rather, a pair of lugs was loaded. And the result of that analysis indicates that the piping stresses are acceptable. The results of those analyses are still open as far as I know. I'm not sure.

MR. LANDERS: Could I say something?

I think I addressed that in my draft report. That's where they do an elastic-plastic analysis.

DR. CHEN: Yes. You mentioned this was unacceptable as an analysis.

MR. LANDERS: Yes.

MR. WALSH: In your discussions with the Applicants, did you find out why they decided not to do what they said they were going to do?

DR. CHEN: Well, the meeting at which this was said was a meeting to relay to the Applicants some of the concerns that we had regarding the reason for some of these positions. I have not gone in to find

out why what they're proposing now is different from what they proposed two years ago.

MR. WALSH: I'm concerned with maybe they forgot. It was one procedure, and then someone came along and said, "We need to consider this." And by that time, it was too late to go back and start doing reanalysis on piping systems. You know, maybe it's a problem with the quality assurance program, the piping stress analysis; people that were really going to work didn't realize that they were committed to modeling these types of restraints.

DR. CHEN: That could be. But more along these questions related to QA, I would hope that in Applicants', quote, get-well plan, unquote, that all of these kinds of things are going to be considered.

MR. WALSH: No, because I remember that was a dead issue as far as modeling struts and then getting this motion to find out that they're not going to do it, I would be surprised.

DR. CHEN: That in a sense they were going to model and they were going to --

MR. WALSH: Sure. It was no longer a concern.

DR. CHEN: That was my understanding two years ago.

MS. ELLIS: I guess what we're saying is that that sort of thing needs to be analyzed as well, something you can tell us in the report, how all this came out.

DR. CHEN: It has QA obligation as well.

MS. ELLIS: All right.

DR. CHEN: Any other questions or comments?

MS. ELLIS: That analysis that you mentioned, was that part of the September '84 information which the QF provided for the Staff?

DR. CHEN: Yes, it was -- well, just a minute. Which analysis, elastic-plastic analysis? The elastic-plastic analysis was part of the original motion, and the analysis for the -- just two lugs loaded is also part of the original motion, I believe.

MS. ELLIS: Okay. Just wanted to be sure.

MR. DOYLE: One other point, and that is the lugs not only are sometimes spaced differently, you think, actually along the pipe, but also on angularity so that the net result on the clamp can be more significant than would be apparent on the surface because if the angularity is such that you're way out on the end of the particular lug or trunnion or whatever it

is, then you have induced moments and shears into the welds and into the pipe which would not be so if it were a perfectly square surface.

DR. CHEN: More along those lines, Jack. Recognizing that nothing is going to be ever built perfectly, what kind of tolerances would you see as being acceptable?

MR. DOYLE: I don't see a tolerance. What I see is usual industry practice. And if you have four lugs, then count two, sort of an arbitrary situation.

DR. CHEN: And in the case of just two lugs --

MR. DOYLE: -- you count one. If you're going to do a Bjillard type analysis additive to M and S and P, existing pipe, then again we use about the same numbers, try to do it all which is conservative, approximately 75. We can get authorization to go to .6.

MR. LANDERS: Where in the lug would you put your load?

MR. DOYLE: When you are overloading under those conditions, then you would put it something like Malcolm Hobbit does with bolted conditions. You put it inside of the -- if you're coming down on a support, you would put it at the tangent point plus,

say, a 16th, because you're going to get some local deformation at that point so you would go to the tangent point of a tube. If you had a trunnion coming out and you're sitting on a piece of tube steel, you go at least to the tangent point plus a 16th of an inch.

Generally, based on what I know, we go to the center of the tube which is even more conservative. But if we get into a real bind, we'll back off a little.

MR. LANDERS: What if you had a pipe clamp with a lug?

MR. DOYLE: How is that again?

MR. LANDERS: A pipe clamp with a lug.

MR. DOYLE: A pipe clamp with a lug? You put it to the center of the thickness of the pipe clamp. But if you start trying to take advantage of all four clamps, you are going to take it all the way out to the worst possible condition.

MR. LANDERS: What about a situation where construction in welding lugs on pipes uses jigs; that is, if they are going to put a clamp on with lugs, they in fact have the clamp there and put it in place?

MR. DOYLE: If you specify on the drawing that you want these particular lugs to be within a specific plane, then you would use that plane, whatever it is.

MR. LANDERS: I just wanted to ask that because you said you wouldn't use tolerances. In fact, in situations where you can control a construction --

MR. DOYLE: For that matter, in submarines they do that all the time. That's exactly how they put their pipe supports up; they jig them in.

DR. CHEN: The second motion I would like to discuss is with regard on local stresses on displacements. This motion covers four topics: zero clearance box frames, stresses and anchors in piping systems, local deflections, and depression in the walls of tube steels.

The last item is the depression in the walls of tube steels, was covered in Applicants' motion, according to AWS versus ASME requirements. What is in this motion is basically a summary of what was in the other motion. And Mr. Terao, I think, has addressed all of those concerns.

Did you have any questions on that?

MR. DOYLE: Are they doing the analysis on the basis of AWS Section 10?

MR. TERAO: That's our understanding, yes.

MR. DOYLE: Well, if they're doing that on that basis, obviously there can be no question.

DR. CHEN: The zero clearance box frames, Applicants are relying basically on two analyses, one which was done by Cygna for a zero clearance box frame. I think that goes in the -- I forgot -- it might have been under SI system. And they also presented some analyses on three supports -- two or three, I forgot exactly -- well, the methods of analyses, and there were also analyses I find at this point unacceptable. Some of my comments have been transmitted to the Applicants, and we haven't heard anything back yet, so that's basically an open item.

MR. DOYLE: Still open. I didn't agree with the analysis at all.

DR. CHEN: The same comments apply to stresses and anchors and piping system. That's still --

MR. DOYLE: Constraining thermal?

DR. CHEN: That's correct.

For local deflections, this is somewhat related to the issue of generic stiffness, and that's still open.

Cinching down of U-bolts, as I understand it, CASE's concerns relate to unusual design issues, A36-A37 material, for the use of those materials, questions related to stability and stresses and the pipe and the

U-bolt itself and local deflections and stresses.

Is there anything else that --

MR. DOYLE: Yes, well, specifically on relaxation, there is no information on A36. The closest they have had, DS60, and it's not really related to A36 material.

But, in fact, that is the lesser of the problems. It then comes to one of these cumulative things where you have cinching first, VF terminal. You have pressure. All three of these are contributing to high levels of stress. And also you have bending of the U-bolt which is one of the analyses Applicants never even considered the bending because you have to take that U-bolt, conform it to the configuration of the pipe.

When you consider all of those, they will have an effect relative to the various differences on the U-bolt type and whatever it's connected to, whether it be a plate and what not.

DR. CHEN: Is this true, that Applicants' motion does not address bending, stresses on the U-bolt itself?

MR. DOYLE: Yes. Those can be as much as four times the actual.

DR. CHEN: In fact, most of the

measurements -- well, all of the measurements, if they're taken on the U-bolt, were just taken in straight portion.

And as to the U-bolt, some of the cross pieces, I do not believe that configuration was tested nor analyzed, were sufficient to cover a broad range of --

MR. DOYLE: There are many cases of the plate and the cross piece.

DR. CHEN: I think that was brought up in a meeting that we had with them.

The motion basically covers the results of an inspection for torques. I mentioned in the meeting with Applicants that that is still an open item, some of our test program and analysis program.

The torque versus free load tests, questions relating to conforming, which you just mentioned, as it relates to bending in the U-bolts was mentioned.

For the friction tests, the results of the tests indicated that there could be problems associated just with dead loads, that which could be more significant than some of the problems that we're looking at.

The thermal cycling, there are givens, rested heavily on relaxation effects, as you pointed out. There is very little information available on relaxation

and the kind of U-bolts we have.

Pre-tests, the test data was still inadequate for the 32-inch U-bolts. And for the two dynamic tests, the normal vibration and similation test and seismic loading similation tests, I have a lot of questions related to the results of the so-called unofficial test.

And have you had a chance to look those over?

MR. DOYLE: No, not really. I didn't look over the tests, and I found a lot of problems.

But do we have anything on that?

Generally their test procedures don't seem to follow the ASTM requirements for one thing.

MS. ELLIS: We submitted some information on that. I'm not sure that we submitted everything. I won't go through the whole spiel again that you've heard so many times about the timing constraints.

DR. CHEN: Again, basically the U-bolt issue is open. And so this is a result related to stability or the stiffness; they're still open. I think in a meeting that we had with Applicants, I think I pointed out that there were several concerns which were raised in CASE's proposed Findings of Fact which were not addressed by the motion.

The last motion I was involved with was differential displacements in large frame wall-to-wall

and wall-to-ceiling pipes works. Again, during a meeting with Applicants, I mentioned some concerns which were not addressed in the motion. Some of these relate to wall-to-floor and -- well, the question as to whether or not the wall-to-floor and wall-to-ceiling supports are more critical or terminal in as-built conditions.

Maybe you can explain to me why you consider those more critical than wall-to-wall and floor-to-ceiling.

MR. DOYLE: I don't really consider them more critical, but the displacements within the wall vertically -- you know, taking the vertical component -- would be far less than the slab to which it comes out and ultimately attaches. So whatever the displacement of that slab is, will be taken up in the frame itself. And there was never no consideration obviously given to that.

But as far as is it more critical, I've never run no firm numbers on it. In the first place, I don't have the displacement history of the plant.

DR. CHEN: The reason I asked that question is because in the proposed Findings of Fact, I think it was stated that these were more critical, the wall-to-wall and floor-to-ceiling.

MR. DOYLE: Probably what I was thinking

at the time is, if you're going from floor to ceiling, particularly in any given plane, you probably would get compensating displacements, although differential.

DR. CHEN: Would that not be unacceptable?

MR. DOYLE: Yes, that's why I mentioned it. But I don't know if it would be more serious. I think it would be less serious than when you come off a wall and you come off a ceiling, you got the ceiling coming this way and in that direction the wall moving very little. But the wall could be moving this way while the slab is moving very little this way (indicating).

So you could get larger displacements, particularly as you pass the points of curvature in the wall and the slab, you get out into the area where the deflection is actually occurring.

MR. LANDERS: You would have to have a large span restraint is what you're saying?

MR. DOYLE: Yes.

MR. LANDERS: You would have to get away from the wall on the slab and away from the slab on the wall quite a ways?

MR. DOYLE: Yes, try to get to the point of neutral --

MR. LANDERS: Yes.

DR. CHEN: There was a problem also in the Proposed Finding that treating wall-to-floor and wall-to-ceiling supports as building supports, as common practice.

MR. DOYLE: How is that again?

DR. CHEN: I think it was stated that these kinds of supports were usually treated as building supports.

MR. DOYLE: They would be considered as a building support without a slip joint. If they have a slip joint, they're just a post, but if you tie solid from the floor to the floor above, you are going to pick up building load because you're going to get time displacement, you're going to get whatever lag-load is put up there. And it's going to act as if it were a building column. MR. WALSH: In regards to that, go out to the D-FW Airport. I think it's the new Terminal 3E or 2E, whatever the new terminal is. In pre-cast, the pre-cast numbers failing the shear to support them, they argue is in tubes just like pipe support from floor to ceiling. Out there they're using it to literally support the building, tube steel members to be used in the concrete tower.

MR. LANDERS: With respect to that,

really all you're saying is that if there is any displacement, it should be considered the fact that it's referred to as a piece of building steel or is inadequate for sure. In this case, the Applicant would -- in any case any applicant would prefer to call any piece of steel NFR.

MR. LANDERS: But the recommended concern is making sure that the broad --

MR. DOYLE: Yes.

MR. LANDERS: What would call --

MR. DOYLE: Call them anything.

DR. CHEN: More along those lines: Considering that the differential displacements both are of the order of .006 of a flange, would you consider the slop at the attachment point as being significant or not?

MR. DOYLE: No. You have to take worst case; you have to assume that they literally got that thing in tight. Additionally, from the time they got it in relatively tight until the time we get concerned with .006, you are going to get time displacement which occurs rapidly for the first few years, but it still goes on.

At that point -- now you've got .006. And if you want to get back with me, you've got a hell of a

load on the column.

MR. LANDERS: What time displacement?

MR. DOYLE: Of the concrete. When you first put it in and pour the forms, you get a certain displacement. As time goes on, you'll get additional displacement. It's rather rapid for the first year or so. In the next five or six years, you're getting some, although it's not as much as you're getting in the first five years.

MR. LANDERS: Have you seen this kind of time dependent displacement in the nuclear power plant with the kind of reinforced slabs that we have?

MR. DOYLE: No, I have never seen it.

MR. LANDERS: Have you seen the displacement occur over -- I can understand what you're talking about with a simple poured slab on a tray, but when you're talking about a slab that, in fact, only has concrete in it so you won't fall through and catch yourself on the rebar, I'm a little concerned if you're worried about time dependent displacement of a slab.

MR. DOYLE: Well, if you put the column in and get a craftsman and he puts it in rather tight, it doesn't take too much beyond tight to make it fully engaged.

MR. LANDERS: I understand. That's

another issue. But I want to make sure about this time dependent displacement.

MR. DOYLE: I'm not talking of 10,000 or 15,000. I'm talking about they put the plate up here which they usually try to get it all snug; otherwise, then they're going to have to pull the bolts and bend the plate. So they try to get it as accurate as they can or as tight as they can.

Now, you get even a tenth of a thousandth, now you're tight. Now you get the seismic displacement of even a thousandth of an inch on a large column, which is essentially a pipe support but now is going to take building loads. You have to account for what will occur.

MR. LANDERS: Okay.

DR. CHEN: More along those lines. In the case of zero clearance box frames, where the pressure displacements -- and again, this order of magnitude -- would you say that for a conservative time, that they should assume that the stresses in loads will occur for infinitesimal displacement?

MR. DOYLE: I'm lost. You say box frames?

DR. CHEN: Yes, where the differential displacements again of similar magnitude, are you saying

that for conservatism the analysis should assume that there is no slop between the pipe and the box spring?

MR. DOYLE: Oh, I see what you're saying. You're getting thermal growth.

DR. CHEN: Right, at the unstable support.

MR. DOYLE: Okay. Yes, for conservatism, because Applicant himself assured us that there is no tolerance, then there was no tolerance, zero net zero. I guess what they do is really wedge it in there.

DR. CHEN: Well, in the case of the zero times box frames, they did assume that zero meant zero?

MR. DOYLE: Yes.

DR. CHEN: This summary disposition is basically open.

MR. DOYLE: Yes.

DR. CHEN: That's all I have.

MR. POSLUSNY: Okay. Before we change topics, I wanted to see if Mr. Walsh or Mr. Doyle wanted to add anything or make any comments in general about this first section or perhaps --

MR. DOYLE: What's the next section?

MR. POSLUSNY: The next section we're going to talk about is cable tray supports.

MR. DOYLE: What I have is a list. The

vast majority of it is stuff that has been discussed, and there are points, and I put it into various categories. Some of them we have discussed already.

What I would like to do is read it into the record and then it's all in one place, and then Applicant can look it over, he can disagree. Like I say, a lot of it -- not a lot of it, but there are many points which could be classed trivial alone, which means that when some of the major points are cleared up, then these might go away, some of them might go away. But they can't go away until we get the plant into a condition where when you read a stress ratio of .6, you can be fairly certain that that is the stress ratio in which case you can now write off.

The first one I cited was stability. And we had:

(a) We had box frames both with struts and snubbers.

(b) We had U-bolts with struts and snubbers. And these are all in the condition that they were back three years ago; this is before all the fixes.

(c) We had U-bolts with two struts; for instance, trapeze type of item.

(d) And then we had the one that I was showing earlier, the clamp with a one trunnion eccentric

to the pipe, which is more of a system stability in that the pipe has to torsionally support -- it works in combination with the pipe.

And then there was -- under (e) I put structural frames of the gang hanger type which was strut supported and had thermal displacements out of plane. And the one I listed as an example is CC41-710-A63. And that is in our 669B.

Over the time we have had fixes on these. The first fix was (a) brackets that were placed at the upper end of a particular strut which was supposed to stop the strut from rotating and thus prevent the instability. But unfortunately, a strut is so long and the brackets were so small that we picked up some horrendous moments, at least in our finding.

And (b), they put clip angles in to hold the U-bolts around box frames; and they were, of course, overloaded, particularly when you talk numbers like 84 pounds of torque.

And (c), they had lugs to stop the axial but it didn't stop the walking.

(d) They instituted a double strut program to stabilize the box frames. Then they still ended up with an axial to the type of an instability in the finding.

(e) They placed bumpers to replace the strut bracketry.

And then (f), they shimmed the box frames to a zero inch gap which created a thermal impression in the screen.

And (g), they went to cinching up of the U-bolts, and that we all know is still an open item. Even if that one is solved as a method of solving the instability problem, there is still the problem of qualifying the U-bolt for the loads which are not in the manufacture's LDS.

And then Item 2, the loads not included on the support, I have:

(a) The self-weight excitation of the hardware is not taken into account.

(b) The swing angle of struts and snubbers were down under five degrees, is assumed negligible.

(c) Friction loads, when they're under a 16th of an inch.

(d) The load differential, including the snubbers, which is what we just went over, due to stiffness differential in two independent frames; and also the inability of the snubbers to lock up precisely at the same acceleration.

(e) The actual section properties,

particularly for the large holes for one and a half inch diameter Richmond bolts which is mostly all removed from the extreme "Y" distance.

And (f) was hanging the supports literally off of Richmond bolts, particularly in the case where they are using only one tube and then cantilevering or hanging off of that single tube.

3, under Hardware: The crossbars were used for cinching down U-bolts, not necessarily tube steel as has been discussed in the analysis. One of them may be noted on CC-008-006 where we had a span of 14 inches, 7,500 pound load, on a three-quarter by 3 inch bar, piece of bar stock, which obviously was overstressed. I believe they replaced that two or three years ago.

(c) Double axial restraint -- looks like I have repeated this one -- load distribution relative to stiffness.

And (d), thermal rotation about the "Y" axis in the horizontal run, delivering different distribution and intensification to the snubbers, struts.

4, under Richmonds:

(a) Excessive deflection of the bolt in the Richmond, alters the stiffness of the support.

(b) Bending in the bolt.

(c) Bearing joints are not acceptable for

dynamic loadings.

(d) Shear loads are not based on proper distribution since all bolts are not active concurrently.

(e) The Mz moment in the tube steel which induces prying action was not considered.

(f) Incorrect procedures for coupling of torsion from the bolt, particularly, as Dr. Chen pointed out, on those with eccentric holes.

And some of the local effects that are not considered -- now, these are some that were brought up earlier. I think these have been repaired -- 16-inch diameter diaphram, CC-08-709.

(b) The tube wall of an 8 by 8 by quarter inch tube steel yielded.

(c) The failure of a W6 by 12 light beam at CC-028-039; that was a stability problem also.

(d) Failure of the plate -- I already mentioned that one. That's that one with the three-quarter by 3 inch.

(e) Torsional problem with CC-107-008. That's the one the load went up 660 percent.

I think all of those -- plus there was the failure of the clip angle, and I think those have all been fixed.

(f) Calculation -- oh, in their calculations, we found this particularly with the Phases 1 and 2. They have supports as short as three inches and four inches deep, and they're analyzing them flexionally; whereas, the real problem was in the clamp. The flexural analysis of such short beams is not conservative, most generally.

Rationally because of the shear lag, you'll literally have a plate with a gusset behind it. Where this will particularly show up is not so much the support itself but in the weld, because the load is delivered to the weld. You have to have yielding in the area where it is loaded before it will move on up the line.

We had a problem with the skewed welding. One of the problems, Applicant in the closed angles of less than 60 degrees, as opposed to the open angles, Applicant analyzed them as a groove weld which, in fact, they are.

However, you run into a problem there. You are putting a vast amount of weld material relative to base plants with a very short phased diameter. So they exceed the usual criterion of face of weld over depth of weld, keeping it between 1 and 1.4. This can create a problem with internal cracking, and internal cracking is

something that visual inspection will miss. There is no way to pick it up.

When we were looking at the same Cygna calculations, we could find no calculations for the pads which were integral with the piping nor the effect of trunnions on such pads.

There was several supports. If you would like to see an example, I've got them with me. When they take a piece of tube steel and they put two flared double welds to a base plate horizontally -- the two that's sitting horizontal to the base plate -- they would put a bracket for a strut or a snub-up and load it. They analyze it as a beam. Unfortunately, the l/d of those is usually less than 2.

But, worse than that, what we're into here is more of warping, and they never considered that as -- side walls are taking all of the load ready because we're delivering a load from here down to the welds down here. You have to transfer it to the side walls. And every one of those I've seen have been analyzed. I have two or three if you would like to see what they look like.

Punching shear: The Applicant at one time thought that if he made the cumulative thickness of throat area of the weld equal to the thickness of the

tube, that he had no punching shear problems. But as Dr. Chen pointed out and, Mr. Terao, I believe you said that they are now doing the AWS Section 10 procedure.

And U-bolts, when they're put in as one-way support but ultimately end up with a two-way load on them, in addition to the load from the one or two-way, you still have friction there. And I have never seen friction considered on any of the U-bolts at Comanche.

And unfortunately, once you get into friction, particularly if it's on a hot plate, you could also actually get into a binding and create yourself an anchor.

Cinched-down U-bolts, the loads exceed the manufacturer's LDS allowables and therefore have to be requalified if they're to be used for procedures other than acceptable to the manufacturer.

The bolts, the U-bolts that pass through tubes, particularly on thin-walled tubes and are bolted to the far side, you have a pull-through problem and a very serious local problem there. For that matter, that's the one that took the Kansas City Hyatt walkway out. The bolts just literally pulled right through.

Also this could have an effect on your pre-torqueing. But if you yield that area, the plant has to last 40 years. And if it gets intermittent loads

and plant transients, you could yield much the same as we found with that PSA phrase by quarter. You yield the walk and you have got your present load and you are back unstable.

And loads on the pipe affecting local stress, half the hardware plus the clamps are actually acting as masses on the pipe, and particularly where you have U-bolts that are hung on large structural box frames, beams, Y flanges, et cetera.

Another area is angular struts. We found some that were angled as much as 39 degrees, but there was no component. If they were taking a vertical load downward and they were sitting at 39 degrees, there was no horizontal component considered.

The cinched-up U-bolts, we never received what the effects -- the total effects are, particularly insofar as the bending at the upper portion of the bolt. You've got the old 1931 Sealy where they used to have to analyze the chain links, you know, see an example of what occurs when you try to bend a curved beam.

And the box frames, of course, I didn't agree with most anything that was done on the calculation there because in the first place they had temperature at the area where it contacted the pipe varying outward. They had a different temperature at the top of the box

beam varying outward, so you had differential temperatures top-to-bottom which induces bending which was never considered in the formula. You have gaps, air gaps which are infinitesimal, but in many cases they equal more than the thickness of the steel that you're considering as far as heat transport, but there are many things that didn't look too swift about that.

Additionally, I think there are more severe problems in the box beams and the thermal pressure and loading is at the welds, right at the major section itself.

And then the one that Dr. Chen was discussing, there are many anchors that have opposed trunnions and then they are locked fairly tight. As the pipe expands, all of the thermal expansion is taken up as loads within the structure itself. And they have been considered recently by the Applicant, but I don't know what I have to say about them. I wasn't particularly pleased with the approach.

And then the one we just got through discussing, building loads on support members wall-to-wall, wall-to-floor, ceiling-to-floor, et cetera. I don't think I have to expand on that. We just went through. And also they act as a seismic restraint.

Stiffnesses, the actual stiffness versus the generic stiffness. Many places, what they're doing to solve that particular problem, rather than worry about the actual stiffness, is to use a lower generic. Of course, the trouble when we get into these higher generic stiffnesses, go through and sample and find out where any soft supports are.

And then undersized welds, that's a rather amusing one because there is no code that says you have to comply by the code as long as you develop a code of your own, one that's acceptable, and particularly in the case of undersized welds.

The purpose of the provision in AWS, ASME, AISC, they all have the mc -- everybody carries the same provision, is for a pre-qualifying weld. If you do that, then you can visually inspect it, and you're on your way. If you don't do that, it doesn't mean you have to take the weld out, you don't have to make a wash pass. What it means is, you just have to go into a volumetric examination. If there are no cracks, the weld is great. That's the end of it.

That's the same as with plug welding. If you can see the plug welding was done improperly and it's proper, the weld is fine. You don't have to go and render it out and put a whole new weld in. You have to

do a volumetric because on many occasions the cracking turns the material to shrinkage.

Then the welds where you attach tubes, particularly circular tubes and you drop below the one-third Beta consideration, it is not really sufficient to say, "Okay. I won't count the welds out of this particular area," because it will receive stress.

If it cracks, then you have a whole new problem again. You have a problem of cracked propogation. You have a notch. In your weld, you start with a notch. So you just don't disregard the fact that, "Well, it's not going to work but it's going to break out there so I won't count it."

And we addressed Appendix XI as long ago as -- in fact, in my summary disposition, I heard nothing from the Applicant in any respect on how they are complying. I brought in two supports which had rigid frame or something wrapped right around a Class 2 girth weld, but they had changed it so that was the end of it. We had never heard if that was a real problem or if that was an isolated incident.

MR. TERAO: Excuse me, Jack. Is that Appendix XI or Section 11?

MR. DOYLE: Section 11.

DR. CHEN: You're thinking of the

inspection requirements?

MR. DOYLE: Yes, right, in-service inspection requirements.

And anybody that's read the first motion found that -- well, Cygna found that 78 percent of the calcs they went through all this iterative process in which we were assured, you know, and done by the best people in the world, wound up they had major calculational errors even at a point where one of them had to be -- one of the supports had to be reworked.

And as I just got through mentioning, I found that the upper lateral restraint was no good. The guy went through 40 pages of calculations and blew it. And the moment restraints, they used the wrong k factors for the attachments to the concrete. And, of course, there was a diaphragm, et cetera.

So I found that there was a large number of calculational errors. And the thing that's so alarming about that particular problem is, they were alerted to the fact that, you know, we were on their back and they were going through a number of iterations to check to make certain that there were no problems; yet, the problems somehow slipped through. So that if they intend certification -- like I say, Cygna found that 78 percent -- actually, the number is much higher than

that, but there were a lot of trivia. I didn't even bother to catalog them.

But of the problems that could be considered significant, particularly generic, 78 percent of the calculations contained problems in fundamentals. The guys didn't know how to calculate a weld on a line basis, composite section, fairly serious problems.

That's about all I got to say. But at least it will put it all in one box. You don't have to go through 15,000 pages -- there are a few new items here, but really not that many. Anybody that's been around is aware. We get lost in the shuffle, we'll mention something, then we get onto something else and that gets forgotten. Two years later you say, "My God! I got Appendix XI, forgot all about that one."

MR. TERAO: I would like to clarify one thing, Jack, on the punching shear where I may have misrepresented what the Applicant is doing. The Applicant is using or has used Section 10 of the AWS code to evaluate punching shear on those supports which had a chord thinness ratio greater than, I believe, 10. Those were the supports that the Staff were concerned with. Punching shear is a problem.

MR. DOYLE: Well, most generally you are correct. If you take a 2-inch and put it into a

12-inch, almost regardless of what the thickness is, you're in trouble. If you take an 8-inch and put it into a 10-inch, most generally you don't have a problem.

MR. TERAO: Maybe we should discuss that a little bit more because the punching shear that you have just mentioned -- in other words, the Beta factor of, say, a 2-inch support or 2-inch tube steel on a 10-inch or a 12-inch support does not reduce the punching shear capacity on that support. The critical element is the chord thinness ratio, which is the ratio of the tube steel thickness --

MR. DOYLE: D to D.

MR. TERAO: -- and not the ratio of the small area onto a --

MR. DOYLE: Yes. Then it's only .6 of that.

MR. TERAO: -- larger support?

MR. DOYLE: Yes. That is .6 of that.

But where I have found from my own personal calculations, where I get into the most problem is when I did get wide -- when my Beta ratio was under five. Then all of a sudden I started getting into problems. And, if they were straight angular, you pick up the same to assist you.

MR. TERAO: Well, as I read the Section

10, then, that Beta factor increases and can only increase the allowable; it does not decrease the allowable?

MR. DOYLE: No. We just start out with a .6 times the D/2T, divided into FY, times AFB2 -- whatever it is. That gives you your allowable. And in many of our cases where we're using thin wide members, the allowables got down so low that we ran into a problem. So whenever I get into a Beta of under 5 with a wide thin member, I generally wound up with problems.

MR. TERAO: I would agree that if you had wide thin members, that Beta can exacerbate the situation.

MR. DOYLE: Yes.

MR. TERAO: But if the chord thinness ratio is below 7, even the paper by Toe Crack (phonetically), which, of course, gave the basis for punching shear, says that the material has its full punching shear capability, has its full shear capability, and that the Beta factor can only help you; in other words, when you have Beta greater than I believe .5, then your allowable can actually increase.

MR. DOYLE: Yes, sir.

MR. TERAO: For Beta less than 5, you follow the Beta view of Section 10. It just says use a

factor the 1.0.

MR. DOYLE: Yes.

MR. TERAO: It does not decrease your shear capacity.

MR. DOYLE: Yes.

MR. TERAO: Another area that I just wanted to ask you a question on as an engineer. It has to do with the five degree swing angle you mentioned.

MR. DOYLE: Yes.

MR. TERAO: Recognizing that construction and design, you have to have tolerances, the five degree swing angle appears to be standard industry practice --

MR. DOYLE: Oh, yes.

MR. TERAO: -- for accepting a support installation. Why is there any particular concern on Comanche Peak for installed less than five degrees?

MR. DOYLE: Oh, no, that's not the point. There is a component. Because you're at five degrees, you pick up .085. You pick up .085 of whatever your vertical load is, which puts a horizontal load into your structure which in many cases is not even included in the calculation. Somebody has the idea that since the codes or the guidance says you can go to .05, no sweat, then that means you don't include that component. But that component can be the straw because .085, if you've

got a Size C, BIET 211, 4,500 pounds down, just to round it off, you've got 450 pounds which is eccentric to the centroid of the beam which is not included in the analysis. That all gets back to your weld.

Do see what I'm saying?

MR. TERAO: Yes. I understand what you're saying but I also -- I don't understand why the situation is any different at Comanche Peak than at any other plant.

MR. DOYLE: Well, most places I worked, they include -- whatever the swing angle is, you take the component and put it into the analysis.

MR. TERAO: I guess the point I'm trying to make is, in any other plant, the five degrees is a tolerance, is it not, if not a tolerance that people have that they don't have to evaluate the effects if this board is installed five degrees or less than the design shows?

MR. DOYLE: No, no. Most of the places I've worked, whatever that angle is, you take the component it will deliver at the bracket, put that into the weld for the bracket and put it into the tube steel or whatever you're attaching.

MR. TERAO: That seems to defeat the definition of what a tolerance is.

MR. DOYLE: No, but the tolerance is not to eliminate loads. The purpose of the tolerance is to prevent you from binding up the strut between the bracket and the pin.

MR. TERAO: Well, that's the manufacturer's tolerance that he imposes for that particular problem. But I'm speaking of the industry tolerance to reconcile design and construction.

MR. DOYLE: No. I'm talking of the swing angle which is incorporated into the design to overcome the thermal movement. It is not a tolerance for the field. It is put in to compensate for a condition which exists, the fact that it's going to move that away. So what you do is, you offset in your design. Now when you heat up the pipe, now you are vertical.

Do you see what I'm saying?

See, if you put -- if we went out here to Comanche and put every single strut plumb, as soon as the plant heats up, now you have put a component in there you don't want.

So what they do instead, just about -- I can't think of any place I've ever worked where they don't do it -- you find out what the pipe movement is and you offset it so that your bracket is now setting where the node point will be at hot operating conditions.

MR. TERAO: Is the concern, then, that in addition to the five degree tolerance, you can have thermal movements which can exceed that five degree tolerance?

MR. DOYLE: Because you tell the man to set it at a specific angle in order so it will thermally come back over to where you want it. If you don't do that, then under hot operating conditions, you've got the thermal movement carrying the clamp out from under the center of the particular support. So now you've got this angle in here during hot operating conditions which is putting the component into the support.

That component can be pretty healthy; like I say, .085 times whatever the vertical load is. If it's a larger support -- say you have got 8,000 pounds, got 800 pounds, and that's not only a bending moment here, it puts a torsion here, puts a bending moment on the weld, creates all kinds of additional loads in the system.

MR. TERAO: All right. I think I understand what your concern is.

One more area which has to do with the bending on the upper curved portion of the U-bolt. You said that the stresses can be four times larger.

MR. DOYLE: No, no. I said, you know, I

think. I don't know. Man, I'm pulling from years and years ago.

MR. TERAO: Several factors larger?

MR. DOYLE: Yes, much larger. That, of course, is peak.

MR. TERAO: Right. That was the point I was going to make. That is a peak stress.

MR. DOYLE: Right.

MR. DOYLE: Of course, it's derived analytically. If that peak stress, of course, shows very high stresses, one might assume that the analysis is telling you that the U-bolt is going to fail. But the Applicant has done testing of the U-bolts in -- this is testing by ITT Grinnell in that summary disposition on U-bolts acting as two-way constraints. They have actually tested the U-bolts to --

MR. DOYLE: Well, before we even start, we know that. All we have to do is go to ITT Grinnell's handbook and it says right down there at the bottom of the page that these loads that you're recommending and in the LDS these loads that you're told to use have at least five-to-one safety factor.

When I go to build this building right here and I put a piece of steel in there, I am allowed to go to 22, 24 ksi. I know I can put 26, and it ain't going

to fall down. You have to justify going over what the LDS says or what the building code says. If I come down to the Building Code Committee and tell them, say, "Hey, I just built a building, and it's 28 ksi. But that's okay because I know and I can prove and I can show by tests," they don't care.

MR. TERAO: I guess that was the point I was trying to make here, is that when the Applicant has now gone to lengths to test these U-bolts to tensile failure, there appears to be a good justification why the peak stress on these U-bolts, at least in the curved portion, should not be a concern and one should then look more at the test results to tell you where these U-bolts are going to fail, rather than the analysis.

MR. DOYLE: In the first place, I don't think too much of the test results. To do a proper test result, the first thing you have to do is get the actual tensile capability of the steel. What you get from the plant is a high speed test. It gives you the upper yield limit for starters. So it's not necessarily the yield point at which you are concerned.

Second, you have to have the exact physical properties and mechanical properties, and then you have to ratio that. You multiply those two factors, then you come up with another factor based on the fact you only

did so many tests. You multiply that.

Now, you're sitting at your ultimate logical load. Then from that you work backwards and find out what you rely on.

But the fact that his U-bolts went up to such and such doesn't surprise me at all. They also deflected an inch and a half or two inches before they failed.

MR. TERAO: I think I would like to clarify that point, too.

We have discussed this with -- as far as the inch and a half and 2-inch deflection -- we have discussed this with the Applicant and Grinnell and those that testing that you -- those test results that show those deflections were not only of the U-bolt. A U-bolt itself, at its tensile, a Florence U-bolt at tensile only deflected about a quarter of an inch.

It was really the test set-up that deflected an inch and a half to two inches that was shown in those plots different.

MR. DOYLE: So in other words, we're not testing to the stiffness of the U-bolt, we're testing for the combined stiffness of several items?

MR. TERAO: Well, it was tested to the tensile failure of the U-bolt, but I just want to

caution you not to be misled by what those plots show. This is not the U-bolt --

MR. DOYLE: That presents me with a second problem: How can I answer things when they don't send me the right numbers?

MR. TERAO: Yes.

MR. DOYLE: We -- not me.

See, you're correcting me on something I know nothing of.

MR. TERAO: That's true. I agree. We just found out about it just recently.

MR. DOYLE: How can I answer it? I mean, I am given a mass of information, and I worked my tail off to answer it. And I am working on the wrong information? This is insanity. I'm spinning my wheels.

MR. TERAO: Well, if you recognize that the plots themselves were intended to show where the U-bolts failed, the load at which the U-bolt failed; that is valid. We can use it for that. But I just was trying to caution you not to be misled by what the deflection was telling you. It was not only the U-bolt deflection, it was the test set-up deflection; whereas, the clocks can tell you where, at what load the U-bolts failed.

MR. DOYLE: Yes. But here again, we're

faced with a dilemma. I don't believe that ITT can be considered an independent testing organization. I mean, in order to evaluate the results of the tests, we have to know precisely what went in and precisely what we can buy because if I put in steel that tests out at 70 ksi, but I can buy steel that will actually come out 54, 55, 56 ksi, then how can I rely on the test results?

So I have to have what the manufacturer guarantees as his size and his ultimate capacity, SU, and the numbers that are based on this item, not the one he tested on.

We tested -- I don't know if you're acquainted with International Nuclear Safeguards which were the first ones to make snubbers, dry snubbers. We tested their snubbers up at the 300 area or at the FFTF.

The numbers they gave us in several cases were off by a factor of four because they were tested improperly. They were tested in the first place horizontally which already established which way they were going to fail, which mode they were going to fail. And then they bounced off the table, which gave them a third point which is what we finally found out.

Also, the test data didn't supply enough information that we could determine that on our own. That's why we had to go and actually do all of the

measurements to find out that this is exactly this, the yield of the material or the element of the material is exactly this, and get all the exact precise numbers. Then we did it in an environmentally-controlled area.

And when we come up with the new numbers, as you're well aware, we throw them all out. So in order for me to evaluate what somebody is telling me, I would have to have the precise information of what he did because I can only go from what I got. What I got told me it was deflecting all over the place.

This is the first time I heard about that.

MS. ELLIS: I think that Jack has hit on one of the things that I wanted to comment on at the end. I might as well go ahead and mention it now, and that is that one of the things that we need to get is the same facts and documents the Staff has seen on some of this stuff. And in a lot of cases, we don't know what you've seen. We're not operating from the same data base that you are in some of the instances. That's a big problem.

MR. TERAO: I do want to point out, you have everything that we have. What I was referring to with this last testing is something we just found out this week, and the Applicant is sending that in, and you'll get a copy and we'll get a copy. But it was

something that we found out from an informal discussion, that then we told the Applicants to document it and make sure that you get a copy and we get a copy. But it take a long time for us to get this particular point out in the open.

MR. DOYLE: No. But you see, my point is that I did something -- what? -- five months ago. Now all of a sudden I find out I didn't have all the criterion.

MR. TERAO: I understand. But as far as Ms. Ellis' concern, you have everything we have.

MS. ELLIS: Or will have it.

MR. TERAO: I think you have more than what we have, a lot more.

MR. DOYLE: I do. I've got rooms I can't get in.

MR. BOSNAK: Jack, I had a point there that I wanted to clarify, to make sure I understood what you were getting at. It was in the area of undersized welds. Let's just say that the minimum size is 5/16ths, and I go along with the full weld gauge, and I assme you're talking about full welds?

MR. DOYLE: Yes.

MR. BOSNAK: And I find that it's a couple of mils under. What were you getting at because,

as you know, it's very difficult to volumetrically examine a full weld?

MR. DOYLE: No. I'm not talking of a couple of mils under. What I'm talking of, where the engineer puts on the drawing, "Use quarter inch." And if a quarter inch is incorporated in the field, it's in violation of whatever one you want to use -- ASME, AISE, AWS --

MR. BOSNAK: So you weren't getting at the fact that it might be --

MR. DOYLE: No. For that matter, you can be a 16th of a inch under for 10 percent of the wall length. Beyond that, there is discussion now in the industry that if it's good for 10 percent under for -- or a 16th under for 10 percent, why not give it tolerance and say that a quarter inch minus a 16th, plus or minus a 16th?

So I don't get overly concerned with a 16th under. Where I really get concerned is where you've got a 2-inch plate, two and a half inch plate and all of a sudden somebody has got a 3/16th weld on there, you've got a heat sink in there that won't quit. You could crack the roof.

MR. BOSNAK: Okay. I understand what you were saying. Before it sounded like you wanted to

examine almost any undersized welds, volumetric or --

MR. DOYLE: No, no, no.

MR. BOSNAK: Okay.

MR. DOYLE: I'm talking of the intentionally installed -- by "intentionally," I'm talking about some designer gets up there and has got a 14-inch Y flange, 426-pounds, and, "Here. Use a 3/16ths. That's all I need." You can have some very serious problems, be it heat effective zone or the roof.

MR. BOSNAK: Right.

MR. DOYLE: But what I was saying is, though, doesn't mean you're going to put a wash pass over it or cap it. All you have to do is prove it didn't crack. If the weld isn't cracked, the weld is fine.

In the aircraft industry, a lot of times they'll have a -- they'll take a piece of thin plate to a big thick piece of plate, very thin, then weld on there. But they make sure it's not cracked. If it's not cracked, it's a great weld.

But if you go in and they put less than the same volume of medal back on as a cap or wash pass or whatever you want to call it, you've intensified the problem. You have done it twice now. So if you didn't crack it the first time, you could have cracked it the

second time.

Like I say, a 16th inch doesn't concern me. But if you've got a 16th inch under and then somebody goes a 16th under that, now you've got a real serious problem.

But the codes do carry a little fat. One of them is, is that 10 percent you can be under.

MR. BOSNAK: Okay.

MR. POSLUSNY: Do you have any comments?

MR. FAIR: Yes. I had a couple of clarifications.

Going down your list, you mentioned something about holes for Richmond inserts and section properties.

MR. DOYLE: Yes.

MR. FAIR: You didn't comment and I don't believe it was mentioned in your response to their summary disposition.

MR. DOYLE: They probably didn't respond to it. That's a long standing argument. What they do is, you take a piece of tube steel, you go put a one and a half inch diameter bolt through there so they cut out a bolt hole that's maybe one and three-quarter inch. That's all your extreme fiber is gone or a large portion of it.

And you'll find that if you analyze it, it

comes out as much as 40 percent under that. That is the section property you thought you had.

MR. FAIR: Is your concern that they don't consider the bolt holes in the stress calculation at all?

MR. DOYLE: That's right.

MR. FAIR: I recall reading that from the original Findings of Fact. And I think it was back about a year ago I had asked the Applicants to give me a sample of a calculation where the bending moment was at the location of the hole and they did submit that. And they did -- at least the calculation they submitted, a sample calculation doing a code type of stress evaluation. That is, if there is a certain percentage of the fibers gone where the hole location was, they recomputed the section modules at that location.

MR. DOYLE: I was out there for over a year and a half, and the only analysis they took was the one right off of the STRUDL. I have never seen one analyzed for a hole -- I'm not going to say there aren't, but I didn't see any calculation. As a matter of fact, I've only seen a dozen, two dozen calculations.

MR. WALSH: In regards to that very problem, I think it was a year and a half ago, in one of the affidavits that we turned in or something -- and

maybe it was from the Findings -- that the Applicant noticed it and John Finneran wrote a potential 10 CFR on that item. So he may have pulled it out of the findings, I don't know. But it's only been since that time the potential was written that they did start considering it. Before then, they did not consider it as far as I know. There was no evidence in the potential 10 CFR that they had, and I forget what the closure was on that.

MR. FAIR: Just to understand --

DR. CHEN: I spoke to John about that, and I called it 50.55(e) related to this issue. It's my understanding that it's a slightly different issue. I'm going to have to dig up the 50.55(e) to see exactly what it is.

MR. FAIR: I just want to follow up on the question. Since you hadn't mentioned it in response to a summary, I had presumed that you had been satisfied on this particular concern.

MR. WALSH: In regards to that statement, we were only given a week essentially to respond to this, and this covered a lot of territory. We just couldn't sit down and cover everything we wanted to. It was just impossible.

Like the design QA, I just came to a point I

just had to quit because it was not enough time.

MR. FAIR: I understand that. I just wanted to follow up on it.

In discussing the modeling assumptions for the Richmond insert tube steel connection, the Applicants have stated that they generally considered what you called the Mz moment pin connection in the model. And therefore, unless you had a continuous beam where you had two loads giving you an additive moment, you might generally not get large bending moments at the location of the inserts. And I just wanted to know if, when you were doing these calculations, and you came across a high bending moment or did you come across a high bending moment at the location of the insert where you didn't look at the stresses?

MR. DOYLE: No, we weren't doing that. All we were doing, we were doing the STRUDL import. If the numbers at the back of the page looked all right, that's as far as we went. If it came out that you were getting 100 ksi, stop right there. But if the numbers were all right, we didn't get concerned over any of the problems with the design of it. That wasn't our function at all.

MR. FAIR: Were there many instances of cases where there were high bending moments at the

location of the --

MR. DOYLE: We didn't even go that far. All we were concerned is that it go over stress. And I don't think in retrospect that I would care. I had already created quite a few waves out there. I don't think I have ever cared to create more. So I just looked at the stresses. If the stress is okay, that's as far as it went.

I was in a particular spot. The guy I worked for wasn't particularly interested in looking at anything.

MR. FAIR: Just to follow up in a similar light: Another issue you checked off your list was not accounting for the Mz moment.

MR. DOYLE: Yes. I am speaking locally "X" axis and the "Y" horizontally, assume a horizontal number.

MR. FAIR: Which is the bending moment along the tube steel?

MR. DOYLE: Right, the one that would prying, what we generally consider prying.

MR. FAIR: Now, the Applicants' motion was an attempt to demonstrate that that effectively was a pin connection for the majority of their tube steel insert locations using their standard families.

MR. DOYLE: It's a pin connection mainly due to the fact that the bolt is yielding under the load so it will come away from the wall. So you can develop a Beta angle in the tube steel. I can see that. But then, again, they cut you back to another problem, that bolts into the Richmond are rather soft.

MR. FAIR: I guess it's a degree of relativity.

MR. DOYLE: No. But added to all the other softness factors, some of the supports are extremely soft. But particularly if it's one tube with a support coming off of it.

MR. FAIR: That's the torsional moment you're talking about?

MR. DOYLE: Yes.

MR. FAIR: Which is different from --

MR. DOYLE: No. It's different from other one; that's true. But what I'm talking about is one of the main reasons you can call it a pin connection, you can develop the Beta angle at that point where that bolt is going through because you're stretching the bolt out. You don't have the whole fix.

MR. FAIR: I agree with you. That's what their analysis was attempting to demonstrate.

MR. DOYLE: Okay. Now we get back to how

soft that bolt is. You develop that Beta angle in there. Do you see what I'm saying? If you have got a piece of 6-inch tube steel that is hardly even moving and you have bolts that are literally moving all over the place, the bolts are the key to what the stiffness of the support really is because you've got bending on these bolts, you've got shear displacement on those bolts, you have got stretch due to the tension between them.

MR. FAIR: Let me back it up a little bit.

Have you still a concern on the Mz moment?

MR. DOYLE: I don't know because I'm faced with two problems. If I say no, I'm not, and eliminate that one completely, now we get into a strange argument over the other problem of stiffness. So the two have to go together. It's much the same as the old thermal problem and the stiffness problem. If you take advantage of the weakness of the one, you get into a problem on the other.

In other words, first, before we start deciding what is not significant, before I ever started I said a lot of this stuff is not significant provided that we get enough of the information into the support analysis so that now we can say, in fact, it is

insignificant.

For instance, you take self-weight excitation of the steel itself, that puts about 5 or 10 percent on the weld so that you'll find that the weld generally ends up to be the critical point. If you take the mass of hardware, and it happens to be a pretty big support, you'll find that adds a percent.

If you take a swing angle, that adds another percent. And before you apply any load, you're using up 25 percent of your allowable. So for me to say that, to get the swing angle, okay, now we got that out of the way, forget the self-weight. You get that out of the way. Pretty soon you have got nothing left. The support is fine.

But in the meantime, you've discarded the cumulative effect of a large number of minor problems in addition to some serious ones.

MR. FAIR: I would still like to get back to the one point.

MR. DOYLE: You are not going to get an answer. You have my answer.

MR. FAIR: I would agree that it would be inappropriate for the Applicants to compute a stiffness, assuming you had a joint and it was fixed.

MR. DOYLE: But they were willing to do

it to prove that there was a theta development in there.

MR. FAIR: But back to my question. The Applicants have done this evaluation to determine whether or not prying exists at that joint for their standard span lengths. Are you still in disagreement that you think that prying will exist?

MR. DOYLE: I am not going to say anything on prying until we find an answer on stiffness. At such point, it may be that we have to change all those bolts to 325, maybe a different problem entirely, because I have explained my position. There is enough displacement that you can develop a theta angle. If you develop a theta angle back here, you have no prying.

But for me to say that, "All right. I drop it, all concerned with it," you are not going to get that out of me because there is another factor involved which is stiffness.

MR. FAIR: I'm trying to get out of you whether you still had a concern after their submittance, after their calculations, on whether their proof was adequate or not, but there was not, in fact, prying.

MR. DOYLE: No. But what you're trying to do is, you're trying to put it in piecemeal. See, I know how Applicant thinks. As soon as I say that, that's the end of that. Forget the joints; the joints

are fine.

MR. FAIR: Well, the Applicants may try to put it in piecemeal, but the Staff may think about it differently.

And as I said in my summary on Richmond inserts, I still had a concern with the Applicants' evaluation being adequate for the frame structures where they assumed the torsional constraint as fixed.

MR. DOYLE: Uh-huh.

MR. FAIR: Which was both in terms of stresses and stiffnesses.

MR. DOYLE: That's as far as I can really go with it. I can state that if we have a weak bolt and that bolt moves up, then we have no Beta, and we are simply supported, but contingent on the fact that we now have a very soft number back there that's yielding --

MR. FAIR: I would agree that it's softer than the tube steel as far as deflection. I don't necessarily say that that means it's a soft -- very soft strength.

MR. DOYLE: It could be, particularly if you've got a short couple this way and now you're moving out here. It could be that even the dead bolt is in trouble because your support may be moving down enough that you've relieving load on this support and placing

the supports up and down.

MR. FAIR: Are you going back to the torsional stiffness?

MR. DOYLE: No. I'm going back to the stretch on these two bolts. Just for argument sake, move this point out one inch, and you've got a cantilever coming out there, now your support out here magnified by the ratio of these two, you could be relieving half your dead load.

I think what I'm saying is essentially the same thing the Staff is saying. It's open until I get all the answers to that.

MR. FAIR: I was trying to determine whether there was something specific --

MR. DOYLE: No, no, just part of the overall -- before I started, I said a lot of these are trivia, a lot of them -- if I had at least 25 percent of them on a given job, I could probably write them all off because I would have stress ratio here of .1. I've seen them for .0. I've seen them less than .1 for stress ratios.

All of a sudden, I got a self-weight excitation. The guy forgot to do it. I am checking it. I ain't going to make them to do a calculation over for something as stupid as that. There is no way in the

world that that is going to be ten times as big as the design loads. But I'm not going to go into a group and say, "From now on, everybody can forget self-weight excitation," doesn't mean a thing. Stress ratio of .98, now it's important.

MR. FAIR: Let me go to one other -- I don't think we'll go any further with this one. You brought up again on your list the bearing joints --

MR. DOYLE: Uh-huh.

MR. FAIR: -- not being acceptable, and I think you have to --

MR. DOYLE: No, no, I never said --

MR. FAIR: From seismic events.

MR. DOYLE: Yes, right.

MR. FAIR: It's seismic events that you're concerned with.

MR. DOYLE: Well, dynamic load, water handling, steam handling, although those are usually one shot in one direction, but they still damp out. So you have to consider those, too.

MR. FAIR: Now, let me get it clear as to exactly what's the basis of the concern on bearing joints and dynamic or seismic events.

MR. DOYLE: Well, it's not the joint as much as it is the particular bolt that's holding the

joint together. One of the things wrong with the joint is, you're only going to get higher damping values in the specter because of the damping effect of the varying joints. You're also going to get a higher peak.

So the joint itself is unpredictable. You don't have a predictable joint. Remember the Japanese are very concerned about damping factors. They're going to shake some of their plants pretty hard, and they already are using half.

MR. FAIR: The concern is unpredictable; yet, the loads, you may underestimate them?

MR. DOYLE: It's unpredictability of the effect of the joint on transferring the load from the building to the support.

MR. FAIR: Do you have a concern of potential fatigue problems with the bolts?

MR. DOYLE: It's not a fatigue problem. It's --

MR. WALSH: Excuse me. I think you got that out of the -- we didn't say that. We haven't addressed this. --

MR. DOYLE: I don't address the fatiguing problem; it's up more than 20 zero crossing.

MR. FAIR: Well, I just wanted to get exactly what your concern was with the dynamic -- is it

you're under-predicting the load on particular bolts?

MR. DOYLE: That's what I'm saying. It's unpredictable. I don't know. I do know that it is not going to act as they have said it would. That is input at a fixed point, which for a friction joint you can establish what that fixed point is. You can go to a certain point, and you know you're going to get separation on parting, so you know what that joint will do.

You don't know what these various other joints will do. You don't have the foggiest notion. They can sit there and just jump up and down every time the sign wave changes -- probably will, don't know.

What I'm asking for is -- you are asking me the question I am asking them: What happens at that joint? Because it is a --

MR. FAIR: So your concern is more the unpredictability rather than the -- you have some concern with fatigue failures -- --

MR. DOYLE: No. I am not concerned with fatigue failure. It probably could be a problem. I don't know what the limit of vibration -- what the vibrational situation is on the particular pipe out there. But I do know that sometimes it gets interesting what happens to those pipes, high frequency vibrations,

low frequency vibrations going on for years, poles, anchor blots onto the wall eventually. I never got into that. I don't think so.

No. I was just concerned with two factors: One, A36 is not in any way, shape, or form recommended for dynamic loading; and, two, if that is a bearing connection, unpredictable, capable of moving over an 8th of an inch, all kinds of locations and everything else, how come it's analyzed if it were a fixed portion with a million pounds or whatever stiffness.

Do you see the point? So what is happening is, you're asking me what I am asking the utility, what is occurring there? Because I have seen test reports on joints where they set out on a friction joint, vibrate it, loosen it up, vibrate it again, loosen it up, vibrate it again. And from that, they develop response factors which had much higher damping values. But they also had peaks on some of them. And that's one of the reasons why, for dynamic loadings, you should be able to predict the action on that joint.

MR. FAIR: Well, you said two things that kind of contradict each other. One is, if you have a higher damping factor, you should be lowering the response.

MR. DOYLE: Not necessarily.

MR. FAIR: The peaks you're worried about are very localized impacts?

MR. DOYLE: Yes. On bolts that are already designed to take 18, 20 -- what's the lowest we went? -- we have had as many as 40, 50 bolts sharing equally all of the load that was put on them. We know better than that. It's not going to happen.

So I think the only thing we've ever said or alluded to or what we're trying to say is, you can't predict what's occurring when you transfer the seismic loading from the structure through the supports to the pipe because there is a weak link there.

But to answer your question, I can't tell you the magnitude of the problem. I wouldn't even attempt to.

MR. FAIR: How was this concern different from bolts, let's say, and struts and snubbers with bushings and gaps such as that?

MR. DOYLE: Well, we get fixed k from the factory. There is a k rating on every snubber and every strut.

MR. WALSH: Stiffness does change through the psi values. For an item like the snubber, it decreases a considerable amount because it takes into account that dead --

MR. DOYLE: But to answer your question, we got numbers; we have no numbers of that joint.

MR. FAIR: Fellows, is your concern more the softening of the stiffness due to this additional joint flexibility?

MR. WALSH: It could be soft at one support and the next one, instead of being loaded in shear, it may be resting right there above the floor or maybe he just hung from the ceiling. It's not going to want to lie flat. It's going to be the case that probably for use of a 1/16th inch deflection criteria, you end up with a soft support and hard support. The hard support may not be able to take it, and it goes back and forth and it's unpredictable.

If you're saying the support doesn't move at a certain point, it shouldn't move. It you're saying it does move and you're going to allow it to move, your 1/16th inch correction criteria will probably -- the support has already moved 1/8th of an inch before it even starts acting, since the 1/16th inch deflection criteria is peanuts. See what I mean?

MR. FAIR: Well, the deflection criteria is really a backwards stiffness criteria, the way it was used at this facility?

MR. DOYLE: Yes.

MR. FAIR: And we have that as an issue in itself.

MR. WALSH: I think this was mentioned in part of our response to that. I think this thing with the soft support/hard support is also included in one of the responses. And I think you get into some gaps. The fact would be, it was just a simple three-span support, but you can see that now.

THE REPORTER: Would you talk up a little.

MR. WALSH: I think I'll stop.

MR. FAIR: I think I'll stop, too. Those are the only notes I jotted down for clarification.

DR. CHEN: Just one question. Sometime or other you said you had a few more in your head. Are those out on the table now?

MR. DOYLE: Yes. There are probably a couple more buried, but that's about all I can think of.

MS. ELLIS: As I mentioned before, Jack and Mark tried to, you know, come up with what they could based on what you told them today. But at the same time, I think we need to have the opportunity to come back later after we've had a chance to review the transcript and think about some of the other things and look at them. This shouldn't be construed to limit what

our concerns are about.

MR. WALSH: I do have another item in regards to these Richmond inserts, these tests that were preformed. I don't know if you included it in the replies, but Jack bought it up in regards to the testing of the U-bolt; and that is, the test materials they used. They tested the 736 rod and at yield point was 60 ksi. That's not specified off of the plant. They may get threaded rod out there that just meets stress, and that's out there, but that's not what was tested.

The same thing goes for the tube steel member they utilized for their test. That tube steel member -- and the Applicants already said a lot of this high strength tube steel, that that's what they used for the test. Then what's out in the field has got to be at least that or better, and it may not be on all the supports. They have had some questionable supports out there because the tests did not reflect what was out there.

MR. FAIR: You can take the results of their submitted material property values and extrapolate them in a backwards fashion, to what the expected -- the worst expected case may be.

MR. WALSH: Then you start losing the purpose of having a test if you can just extrapolate.

And then their 1.75 stress ratio was out the window. Their testing was to verify so that they could use normal analytical techniques to approve these designs. And what they tested did not match what was out there, and that's where they have a problem.

MR. FAIR: I'm not catching your point.

MR. WALSH: There are too many elements. The concrete is stronger in their test; the bolts are stronger, the tube steel is stronger. So all these items that they tested, if they're stronger than what is out in the field, how can you say, "Well, decrease this this much and this this much and this this much," or increase, for example, deflection.

Now, I don't know what the deflection will go up when concrete strength goes down because the concrete is not as strong. And that's what, you know, the requirements was for 4,000 pound concrete. I think they tested 5,000 or something like that.

Now you've got, you know, a large increase in strength which is not reflected, but there is a difference. And like Jack -- the increase is not linear, it could be the square root of the concrete strength. You know, there are a lot of variables involved, and the only way to get around that is when you test, test the weakest point. And then when you

install it in the field, it will be greater, it will be stronger so you know it will work, instead of getting your good steel and testing it. I don't know if it was good steel. I didn't see any material properties in their report as far as the strength of the steel or concrete was in there.

MR. FAIR: I guess I'm still missing the crux of your point. I understood from your submittal the difference between the tested concrete strength and the minimum specified concrete strength argument.

MR. WALSH: Okay.

MR. FAIR: And that's fine, that's no problem. As far as the other tests, when the torsional load on the Richmond inserts, the only one that I can think of is another test they were using, it was a test used by them to try to demonstrate that their analytical method was highly conservative. And what you're saying is that that's an inappropriate test?

MR. WALSH: Well, if they had the threaded rod with the yield strength of 60 -- okay? -- and they go out and test it, and their results are going to come out good. Now, if they go out there and they test the A36 rod and they had a yield point of 36, the results are a little different; you might even see a yield point on the test. It was nothing, you know.

That's what I'm getting at.

MR. FAIR: But for that particular test, even the Applicants aren't trying to use the results of that test establishing direct allowable?

MR. WALSH: They are, that stress ratio of 1.75 because of their test results. And that's how they came up with that new allowable. The allowable normally established in their code is stress ratio less than or equal to one. Well, also they're going to use 1.75 now as a test result, and then at the finite element analysis that it performed that is questionable.

MR. FAIR: Well, we have that as an open issue.

MR. WALSH: Correct.

MR. FAIR: But I just wanted to get the point, that the test itself wasn't one where you divide what they call failed or deflected load by a certain factor and said this is the allowable, it was, "We've got this method. And look at how much more strong this support is than what we calculate."

MR. WALSH: Right. So what you're talking about --

MR. FAIR: Which is somewhat different --

MR. WALSH: It's so much stronger because the materials are stronger because, see, what happens

when they get the material that's used out in the field, now are they still going to be able to say it's still stronger? That is what I'm getting at.

The results of that test could draw a conclusion that they could use a stress ratio of more than 1.75. If they go back and test it with weaker items, maybe they can't come up with that statement then.

MR. FAIR: As I said, we still have that as an open issue with the basis for the 1.75 anyway. But I wanted to make a clear point, that it wasn't -- the NRC Staff isn't looking at the results of that test and saying, "Hey, we can divide that load by a factor of 4 and show that everything is within allowables," or take that as an allowable. We're not looking at it from that point of view.

MR. POSLUSNY: I guess I'll let you go ahead.

We have one request. Would you have a problem if we made a copy of your notes to put into the record?

MR. DOYLE: You mean that thing I read?

MR. POSLUSNY: Yes.

MR. DOYLE: No.

MS. ELLIS: As I mentioned earlier, we primarily were interested in the issues anyway. But

since we were talking about summary dispositions, I think it probably ought to be noted for the record that there are three others. The Board said they were treating A500 steel information as a motion for summary disposition. So that is one.

The other one is the upper lateral restraint. And we would like to find out about that. And then the design QA which we understand won't be addressed until you get through with all the rest.

I guess the next thing is where is the upper lateral restraint?

MR. DOYLE: Oh, way up.

MR. POSLUSNY: We're going to speak to that. I know one of our consultants is working on that. I don't know what the status is.

MS. ELLIS: And here again, I might mention we have some concerns about that because I know -- I want to be sure again we're working from the same data base.

MR. TERAO: I think the difficulty there is maybe oversight on our part, but that was one of the summary dispositions that was given to one other person in the NRC. He contracted it out to Appropriated National Labs. And that was always treated as isolated with us, the four of us. So it did slip through the

cracks. We didn't bring the right person down to talk about it today.

DR. CHEN: If I might add something, said for the Commission or has been said fairly recently in response to that question related to Level B or Level C loads, it's still open as far as I know.

MR. POSLUSNY: Design QA we've covered.

MS. ELLIS: It's still open.

MR. POSLUSNY: Just one minor comment. Just bear in mind we've given you a, quote, status on each of our items, and they're not the official NRC position yet. Many things may change. I just want to make that clear.

MS. ELLIS: Right. I take it that you did pretty well find an answer as what your concerns are at this point. Right?

MR. POSLUSNY: (Nods affirmatively)

Mr. Beck, anything you would like to say?

MR. BECK: No.

MS. ELLIS: There are a couple of other things. Those probably ought to be discussed. One thing is that I want to say again that I think this is a very productive sort of meeting, and I think that it's long overdue and that it will help a lot when we finally do come to a hearing on -- hearings by mail or whatever

kind of hearings we end up having to decide the issue.

I think it will save a lot of time when we finally do get to the final point of this. I think that is very important and something to be desired by everybody.

I want to mention one other thing, too, that is a little bit of concern still to us, and that is that while we appreciate your efforts, you have got to realize by now that there is no way that Jack Doyle and Mark Walsh found all the design problems that there are at Comanche Peak. And I think it's pretty obvious from the ones that have been identified by just these two individuals, on a very limited perspective of what went on at the plant, that there are serious problems.

And I think also you have to recognize that the manner in which these have been handled has to be considered to be generic as far as the Applicants' manner of handling these. And if this is generic, I think the Staff has got to look much, much deeper than has been looked at so far.

I know that's just what y'all wanted to hear, that you need to do more work, but I think that's really almost mandatory at this point in time because I think it's been proved that there are some really serious problems. Otherwise, you have got to realize how is it

that after going on three years, these two individuals have -- why their concerns have not yet been answered?

I think one of the basic things that has to be recognized and the main reason for that is Applicants were not able to simply say, "Okay. This is what we did, and here's the calculation and documentation. This is why we did it."

That's all it ever would have taken and Jack Doyle and Mark Walsh's questions would have gone away. And that hasn't happened, and I think that's got to be recognized as a real underlying problem that has to be addressed by the NRC Staff.

Obviously, Jack and Mark can't look at everything. But to me, one of the most telling things that occurs through all of this with Cygna is the fact that the few things that were looked at that were outside the area that you normally -- such as the upper lateral restraint and the -- well, the cable tray supports, for instance, when those were looked at closely by these individuals, they found that there were just as many problems with those as there were with the other things. To me that's a clear indication that this really is a generic sort of problem, and I wanted to bring that out very definitely.

Another thing that I wanted to do with the

Applicants here, we have now asked in the February the 7th, at the end of the February 27th meeting, and now we're asking a third time today for information on just who the Applicants' team is, what their qualifications are, you know, all of this, all of these details. And the third time is a charm. We're not going to ask again. We're going to try to take whatever steps are necessary to get that information. I just wanted to make sure.

I guess unless the Commission has some other -- y'all have any questions or anything, that it might be well to take a break now and go into cable tray supports.

MR. POSLUSNY: Okay. Make this 15 minutes.

(Brief recess)

MR. POSLUSNY: Shall we begin.

Charlie Hofmayer, would you like to start or did you want -- Charlie Hofmayer and Rom Lipinski will both address the outstanding issues.

CLARIFICATION OF ISSUES

(Rom Lipinski and Charles Hofmayer)

DR. HOFMAYER: My name is Hofmayer, H-o-f-m-a-y-e-r, NRC.

MR. LIPINSKI: Rom Lipinski, L-i-p-i-n-s-k-i.

The purpose of this meeting that we want to take an opportunity to discuss with you, it is clarification of the issues that have been discussed during the meeting of November 7th.

The meeting of November 7th, there were some points made. And in order to make sure that we proceed in the right direction, we welcome this opportunity to meet with you. And we would appreciate your cooperation to clarify what you said during that meeting. It will make our work much easier to accomplish what we want to do.

I have in front of me some of the pages from the transcript of that meeting, and I am going to address these questions with Dr. Hofmayer as we go along.

Let's start with damage study. On Page 110, Mr. Walsh said something and I will quote.

"When they did that study, did they

consider 2/1 projectile; for example, going,
end of quote.

And the question that we wanted to ask you is what did you actually mean by saying "2/1"? Did you mean the projectory of the non-safety related components or did you mean the interface of Category 1 components with non-Category 1 components?

MR. WASSH: The projectory.

MR. LIPINSKI: Projectory. Okay.

Would you be more specific in your concern. In other words, you found any specific knowledge of this being not treated properly?

MR. WALSH: This was in regards to the control room, I believe, we were discussing at that point, and it was stated somewhere along the line that there was a damage study performed. And although I haven't got any proof of it, evidence of it, but because of the problem in the control room and them saying there was a damage study performed, I was wondering if they had considered that type of item in the damage study when we went out and looked at the plant.

MR. LIPINSKI: By "item," you mean the correct zone of influence directed in the damage study. Is that what you mean?

MR. WALSH: Yes. When they decided this was a non-seismic item, could it still fall --

MR. LIPINSKI: In other words, because you mentioned 2/1, what do you mean by that? Do you mean two vertically and one horizontally, or the other way around?

MR. WALSH: Two horizontally, one vertically.

MR. LIPINSKI: Two horizontally and one vertically.

DR. HOFMAYER: Just to clarify, your real question was whether to consider any horizontal motion.

MR. WALSH: Correct. That's the main thing, the way they treated it.

DR. HOFMAYER: You haven't specifically looked at the damage study at this point to have any specific concern. It was kind of a question in passing, I take it?

MR. WALSH: Correct.

DR. HOFMAYER: As you know, this issue stemmed out of the control ceiling question, and there is an action on the part the Applicant to relook at the damage study. As far as we're concerned, that is open. We just wanted to be sure there was nothing specific that you had in mind, that at this point you come to the

table and that you addressed.

MR. WALSH: Well, the other item would be the HVAC containment, treating that as a -- that was closed as part of the SSER which was published recently.

DR. HOFMAYER: Maybe we need to clarify that. I'm not sure what you're referring to.

MR. WALSH: There was a -- I believe it's an SSER that was written, that came out of the TRT findings, and it's wherein the last month -- or that's when I read it -- which closed it. But the concern I have is the HVAC did not actually have any -- the way I looked at it, it was not adequately braced and it would be acting as a projectile also.

MR. LIPINSKI: Well, again, let me interject here -- Rom Lipinski -- that your concern is again not the overall picture of this interfacing between non-Category 1 and Category 1 systems, but the projectory. Right.

MR. WALSH: Correct. You know, did they take that into account?

MR. LIPINSKI: Okay.

MR. LEVIN: Excuse me. Are you questioning the implementation of the program and the zone of interaction or the actual definition of the zone of interaction?

MR. WALSH: If they took -- if they had utilized proper zones. I don't know what the zone of interaction was -- that's where it is -- it was two horizontal, one vertical, or one horizontal, one vertical. I don't know what the criteria was.

But since there was a problem already with the control room, you know, that was based on a damage study on what was -- the damage study, you know, what did they use as a criteria for projectile?

MR. LEVIN: You mentioned 2/1. And to my knowledge, there is no one zone of interaction. That varies as a function of the elevation 1 is above -- the item might be above the floor and the floor that the item may -- the elevation that the item may be on. 2/1 most generally refers to the name of this issue in the industry. Sizing of 2/1 doesn't refer to the zone of interaction.

MR. WALSH: That's correct, and I meant to say 2/2/1.

DR. HOFMAYER: You know, basically, the damage study is an action that's still pending. And certainly that matter and how it can be treated, you know, what impact, you know, Category 2 might have, non-seismic might have on Category 1, if this question is to be resolved.

And we just wanted to make sure you weren't aware of more things about the damage study that come to light now and be folded into this review, you know. I understand your question, and that will certainly be incorporated into the review.

MR. LEVIN: Charlie, we indicated on I guess our recent meetings on the 6th or 7th that we had undertaken a third party review of the damage study, and assumptions such as the zone of interaction are included in the scope of that review.

DR. HOFMAYER: All right.

MR. LIPINSKI: Well, my part, I want to assure you that we have followed this rather closely and we are working on it.

Then shall we go to the next one?

DR. HOFMAYER: Yes.

MR. LIPINSKI: On Pages 115 through 119, there was a question raised of use of a preliminary study method.

And are you aware of that particular issue, I guess? Could you be more specific on misuse of this method anywhere in the implication?

MR. WALSH: Well, they did not use it on a cable tray supports.

MR. LIPINSKI: They did not?

MR. WALSH: And they have this dynamic amplification factor of 1.5 unless shown to be less. And when Cygna did their review -- and they retained their calculations they worked, that dynamic application factor had not been used.

Recently, with the Cygna meeting, that item, I believe, was indicated as Item 9, and I believe they indicated it was closed, and the Applicants would be using an amplification factor of 1.14.

MR. LIPINSKI: Could you be more specific about that meeting? When was that? You said recently? When was it?

MR. WALSH: It was in California last Thursday.

MR. LIPINSKI: Uh-huh.

MR. WALSH: There were no references indicated for that item. They were reported later. I haven't seen the justification yet for the 1.14 in lieu of 1.5 that is indicated in the FSAR that they would use. I have seen a preliminary report that was done by Gibbs and Hill saying that they could use a dynamic amplification factor of one. In fact, there is one part in there I think they said that was conservative.

MR. LIPINSKI: Do you remember when that report -- time of the publishing for that report or

whatever?

MR. WALSH: It was a draft report in May of '84. I went through some of the Cygna correspondence. And of September of '84, that issue of a dynamic amplification factor was considered an open item, depending on Dr. Bjorkman's conclusions or whatever.

I haven't seen anything from Cygna, how they came to a conclusion. The FSAR at the time of the Cygna report differs than what was actually out there. Cygna did not pick up this non-conformance to the FSAR requirements in more than one way.

In the FSAR, at the time the Cygna review assumed trays were flexible and supports were rigid. And they designed, using the equivalent static load method and utilizing the 1.5 factor above the peak. They hadn't done a dynamic analysis.

MR. LIPINSKI: That's what Cygna said?

MR. WALSH: That's what the FSAR had. When Cygna did their review, they did not realize that the supports are not rigid. They just went out and analyzed them as if they were flexible, and they assumed then the tray was rigid, still not realizing that the dynamic amplification factor had not been used. So even after they issued their report, the Applicants

revised their FSAR to reflect what is going on now.

Now, as far as the dynamic amplification factor, I do not know what the Applicants have done to the FSAR, if they're going to change it to say they're using 1.14 or they are using the 1.5.

DR. HOFMAYER: One thing I believe in the FSAR, the requirement is that they will use 1.5 -- okay? -- but they can justify a lower value. I don't believe that they are strictly limited to the 1.5.

MR. WALSH: Yes, but they had not used 1.5 until I brought it up. We don't know where in that plant, if they ever used it. It would appear to me Cygna just looked at cable trays that came out of Gibbs and Hill, and there was nothing to indicate that they had ever used the 1.5 factor.

DR. HOFMAYER: Or as adjusted might be used in the lower factor?

MR. WALSH: Correct. They're adjusted --

DR. HOFMAYER: What I'm saying is, you're not required to use 1.5. You're entitled to use something less if you can justify it?

MR. WALSH: Correct. I am aware of that.

Yes, you justify in advance, not after someone has done a review and someone has figured out you have to do this. And the supports that Cygna had looked at,

Gibbs and Hill reran them using a NASTRAN program to get the -- instead of going to the peak, they got the correct frequency, and they still had supports over-stressed, 7 percent, with utilizing the dynamic amplification factor.

MR. LIPINSKI: Excuse me. When you talk about supports, you talk about --

MR. WALSH: Cable tray.

MR. LIPINSKI: -- cable tray supports of the channels that -- ladder type members, these were over-stressed?

MR. WALSH: Yes. But, you know, they are doing it now and they're going back -- I don't know if they're going back and looking at their calcs for other supports. There has been no requirement for them to do that as far as I know.

DR. HOFMAYER: What do you mean by "other supports"?

MR. WALSH: Cygna didn't look at all the supports. They only looked at a select few. And the ones Gibbs and Hill ran, Cygna did not require them to rerun. From what I understand, it was at their own choice, and they picked them up. I don't know how they came up with a sample of which supports they would send back to Cygna to show there was no problem.

MR. LIPINSKI: You say of their own choice. That was Cygna's choice or Gibbs and Hill's choice?

MR. WALSH: Gibbs and Hill.

MR. LEVIN: Maybe I could clarify the record in this regard, first with a few comments on what Cygna has concluded in their current activities.

To the best of my knowledge, they have concluded that the factor of 1.14 has been established as an appropriate factor. However, they haven't halted their work at that point. It's going to be included when they look at the effects of other items, in terms of drawing their overall assessment.

I might add that we on the CPRT are going to be doing a similar activity. We'll verify the validity of the 1.14 and evaluate it along with the impact of other items that have been expressed in the cable tray area.

DR. HOFMAYER: I might add, you know, we don't have much more detail than you do in terms of the basis of the 1.14 or, you know, all of that information that recently came out of that meeting. But certainly we're far from beginning in any way to express a position on this matter.

MR. LEVIN: Charlie, the basis is in fact

calculations by Gibbs and Hill. And as I indicated, we plan in our third party verification activities to take a look as that calculation as well as the applicability of that dynamic amplification factor to the cable tray systems as a whole.

DR. HOFMAYER: The main concern is that in applying the equivalent static load method, there are several options that were given in the FSAR. One option would be to apply a 1.5. Another option might be to justify a factor of less.

The question raised is, when the designer did it, did he indeed go through that process to determine what is the appropriate factor under the rule? And that should be a function of your review, and it would be something we would follow up on in terms of why.

MR. WALSH: Yes. Well, see, my concern is more than just the cable trays. There are other structural items out there. Someone has got to look at, is the whole plant designed that way and can they use that 1.14 for the stairs or what other structural items that are out there?

MR. LEVIN: We have initiated a survey to identify all areas of the plant or all hardware that may have been designed using equivalent static methods and determine what dynamic amplification factor was used.

DR. HOFMAYER: At this point in the question of your concern -- okay. I can see where your concern came from, your review of what other people may have done and whether they did it or not. Have you looked at some cable tray analyses or reviewed anything that leads you to believe that 1.5 is necessary or what might have been done was not correct? Have you looked at any --

MR. WALSH: I have looked at the calculations where they did not use it. Now, these are generic designs. The FSAR at the time I looked at it, at the time Cygna looked at it, the FSAR said 1.5. There are no other studies to say they could use something less.

It's after the May hearings of '84 -- it is after the May hearings of '84 that this came to light. I received the calculations on -- I essentially started looking at them on a Saturday. We had hearings starting Monday. I had worked with Bechtel. We used 1.5.

MR. LEVIN: Mark, I think we have to separate out some of these issues. One, relative to our determination of the acceptability of the design, I think we're going to look, at, you know, the quality of the product. I think the question you're raising now is maybe related to recause in the design QA area but not

design adequacy. Okay? I want to be sure that we properly separate those issues.

MR. WALSH: Well, see, the problem that I see is, I had two days to look at this. And if I can find -- you know, if I can find something like this -- and I don't have all the calculations -- why didn't a technical audit pick it up? The other thing is, why didn't Cygna and why didn't Gibbs and Hill? It's their design.

It is like a design QA problem. It should not have occurred. It should have been picked up is all I can say.

MR. LEVIN: All I can tell you is that it's our intent to look at the generic implications of that if it occurred.

MS. ELLIS: I think what you said, Howard, is true to a certain extent. But I think also that the concern here also goes to the adequacy of what's out there because at this point in time we don't know, we haven't --

MR. LEVIN: Well, in fact I think that's the most important aspect right now, and that's what we'll deal with first.

MS. ELLIS: But also there is this continuing concern of how did this happen? And also

what else is out there that may have been the same way?

MR. WALSH: Those other items that I picked out in that two days in that 1.5 factor, that was not considered by the Applicant to the best of my knowledge.

MR. LIPINSKI: Are you saying it was not considered? In other words, they must have considered some factor, you mean factor of one?

MR. WALSH: Factor of one, sure. It's better than zero.

DR. HOFMAYER: Well, that's mischaracterizing it since they don't use the peak --

MR. WALSH: They don't, that's the problem. They do sometimes; sometimes they don't. When they did their reanalysis, they did not use the peak, and they did not use the 1.5 factor.

DR. HOFMAYER: They did not perform any frequency test?

MR. WALSH: They did perform a frequency analysis to determine what the applicable frequency was and used the appropriate g value.

MR. LEVIN: I don't want to get in a position of defending that in view of our ongoing activity of reviewing that, but there are differences between what one does in the design basis evaluation

where, in fact, yes, your criteria is to apply some factor times the peak; yes, in fact, you do that, as opposed to an evaluation where you're trying to verify the validity of some factor.

In that case, what I understand has been done is, a dynamic analysis was completed where, Charlie, as I think you were suggesting, values and item factors were known and response can be calculated also. What you might say is, that an equivalent dynamic amplification factor.

You know, there are differences -- you have to look at the purposes of that study. That study was a study and not a design basis analysis from the standpoint of, you know, trying to implement some criteria like a factor times the peak. It was to be used to actually calculate what that factor should be.

MR. WALSH: That's one of the reasons why I want to see the calculations. If they're using one assumption, and if they are going to verify something else -- and we've seen the studies having erred before. I just wanted to be sure that they concide with what I have seen in this other stuff that we have received from Cygna or the Applicants.

DR. HOFMAYER: I think I understand your concern.

MR. LIPINSKI: Go to the next one.

The next is that we understand the difference in allowable stresses for cable trays in considering building stresses containment. And that's mentioned on Transcript Pages 119 through -22. And we want to clearly understand why are you concerned about the stresses to each cable tray's design or cable tray supports are designed, different stress allowables in containment of stresses?

MR. WALSH: In their FSAR, under the containments, steel structures, maximum axial and bending stress in a member under the SSEOC Commission can only be .9 Xy.

DR. HOFMAYER: Could you clarify that.

MR. WALSH: It's in the Cygna issue, should you pick it up. But in the Aux Building, they don't have that stipulation. And the generic designs did not consider what happens when they use an increase of 60 percent for the allowable that the stresses go above yield.

MR. LIPINSKI: Well, it goes just about two percent, which is a very small amount. You multiply 1.6 times .6, you get just about 1.02.

THE REPORTER: Will you speak up, please.

DR. HOFMAYER: Maybe we could clarify the

reasoning because when I reviewed your concern, I looked up the FSAR, Sections 383, which is for structures inside containment and Sections 384, which is for structures outside containment, and read their criteria, their structural acceptance criteria for both steel for both sections. And basically they're identical except for one statement.

I didn't see any reference in there to the concern of (inaudible) but there is a statement on Page 3.8-83 which says that,

> "The steel is designed so that the maximum stress for any load combination which includes differential pressure is less than the yield stress, thus assuring that it behaves."

Is that the basis of why you believe that the cable tray design should be different in inside containment as opposed to outside, or is there some other criteria that I haven't seen that would lead you to that conclusion? I'm just trying to get an understanding of what we're trying to address.

MR. WALSH: I thought it was .9, the yield stress of the steel. It might be a different version, too, than what you're looking at. It may have been revised.

But just the same, if you looked at the Cygna Phase I and 2 report, the final report, they had the allowable stresses that they did use and they did see yield strength in the material.

DR. HOFMAYER: Unless the Applicant has, you know, a different position -- the last time we discussed this, where we raised this question, my understanding is that your criteria you believe are the same, are the same for both inside and out.

MR. LEVIN: That's my understanding also, Chuck.

MR. WALSH: They're a generic design; they can't be.

DR. HOFMAYER: My problem is, if your concern stems directly from this statement -- I don't know -- this particular statement which would require the steel to be less than yield on the basic and remain elastic, which will establish some difference in criteria, is really in there for a load combination which includes differential pressure. And if you read the current standing review plan where basically this criteria is almost identical, there really is no stipulation like that, that that type of requirement placed more for structures that would stand quite alone.

And this same requirement is there. Okay?

The Staff originally took the position and still maintains the position that when you design a structure for pressure load, they wanted to be assured that the structure would remain elastic. And that's the way I interpreted the statement in meaning. I'm not sure -- I can't put words into the Applicants' mouth. But certainly I don't interpret that the requirements are different for the cable trays inside containment or outside.

MR. WALSH: Well, see, there is the other thing, one of the load combinations --

DR. HOFMAYER: That's pressure load.

MR. WALSH: Without pressure, just the temperature effects is in the steel section of the FSAR. You have to include temperature. When you include temperature, you have a LOCA environment, you're going to decrease your yield strength of the material. I think Cygna is also trying to address that. Applicants did not consider the LOCA environment on the cable tray supports.

MR. LIPINSKI: You're talking now about behavior of the material?

MR. WALSH: Yes.

MR. LIPINSKI: Elevate the temperature. You are not talking about the structural -- the

temperature lows imposed on the structural members due to elevated temperatures.

MR. WALSH: Because you have a determinate structure, no stresses due to thermal, just the increase in the yield strength.

MR. LIPINSKI: So in other words, what you consider, that in spite of the fact that the criteria might be the same, the high -- the elevated temperature should be considered in the design?

MR. WALSH: It would be -- right. They are different structures and under different behavior.

Now, LOCA values would be considered in the Aux Building, but I would be less concerned with that because the temperatures aren't going to get as high in the containment; they won't. And considering a -- we have used that normalization process with a large SSE loading condition.

MR. LIPINSKI: Well, okay.

DR. HOFMAYER: I think that's a slightly different twist, but I'll think about it, I guess. But . . .

MR. LIPINSKI: Well, it just proves that it's beneficial to have this kind of a meeting so that we can exchange our interpretations, our views.

Do you have any specific knowledge about the

instances where yield stress was exceeded?

MR. WALSH: Well, see, when they used their normalization process, they didn't consider the SSE condition. But when they did the old condition, it was 7 percent overstressed. It's part of the record. There are CASE exhibits from the May hearings of '84 which we -- I don't know if they're --

MR. LIPINSKI: Do you remember what particular structures, structural members were designed that way?

MR. WALSH: I don't remember right now.

MR. LIPINSKI: But still we need something to prove out --

MR. WALSH: It was their calculations; it's a CASE exhibit in the record.

MR. LIPINSKI: Do you remember the number of the -- something to give us more --

MR. WALSH: Somewhere between 900 and 1,200.

MR. POSLUSNY: Maybe you can check on it.

MS. ELLIS: We can check on that.

MR. LIPINSKI: In other words, you don't refer just to cable trays, you refer to any structural members in general. Are you talking about cable trays in particular?

MR. WALSH: During that meeting, I'm pretty sure I was just talking about cable trays. But, sure, why not? They didn't consider LOCA on the upper lateral restraint. They didn't consider it on the cable trays. So why not? You know, we haven't looked at that many calcs. There has been, I think, three of them.

MR. LIPINSKI: Yes.

MR. WALSH: Three areas.

MR. LIPINSKI: I guess we can go to the next item about the seismic gap. Do you remember the specific door opening that you made the reference that there was integral part of the one building or part of the other building?

MR. WALSH: I don't know where it's at. I know they were attached going through a door. It appeared to be attached concrete to concrete.

MR. LIPINSKI: Yes, but that was --

DR. HOFMAYER: Well, let me clarify that. When you say "attached," first of all, was this a door that you went through, the Containment Building?

MR. WALSH: No, no. It was in safeguards.

DR. HOFMAYER: You went through a doorway in the Safeguard area?

MR. WALSH: Safeguard.

DR. HOFMAYER: Actually, as you walked through the safeguard area, you would see the gap between those two buildings, potentially see the containment wall, so you could observe the potential gap between the Auxiliary Building and the Safeguards Building?

MR. WALSH: Yes.

DR. HOFMAYER: But through that doorway, you couldn't necessarily see the Containment?

MR. WALSH: Right. I saw there was a gap on the floor. Let me see.

But above the door, it is a gap.

DR. HOFMAYER: When you say "connected," do you mean --

MR. WALSH: Concrete to concrete, as they used the Containment as a form -- that's how it appeared. But it bowed up. I don't know if there is any rebar in there or not.

DR. HOFMAYER: You couldn't have mistaken that for the --

MR. WALSH: I could --

DR. HOFMAYER: I guess what potentially in that particular location, with the air gap would have been closed, potentially not open. Are you saying by design, they were physically joined?

MR. WALSH: By construction, they were joined. I don't know what the design would have been, but I really can't pinpoint the location. Sorry.

DR. HOFMAYER: You realize, I think, certainly this is an issue that we have open right now in terms of the overall adequatecy gap, and the Applicant has a real expensive program to go back in and look.

MR. WALSH: The Applicants may have fixed it since I was there.

DR. HOFMAYER: If they have, you have no problem with that? For the matter, they may have -- as long as they provide an adequate air gap, that's what we're primarily concerned about.

So absent more specific locations, it's very hard for us to say that we will go and look. We can look at a particular location. We certainly would address the overall adequacy of the air gap as part of the overall open item that we have.

MR. WALSH: Isn't -- I may be mistaken. Isn't there a walkdown for it, so if there is, it would be caught there?

DR. HOFMAYER: Our concern was, it was sounding like something, when we first read this, as some kind of integral attachment to the containment and

structure itself.

MR. WALSH: It looked that way to me.

DR. HOFMAYER: We did go out and look around all the openings to the Containment, or at least three of the four, I guess. We didn't observe anything unusual in those areas. That's why we wanted to get a little more specific.

MR. WALSH: I can't. I'm sorry.

MR. LEVIN: I think it might be appropriate to clarify a few things. No. 1, there are locations where this type of material is permitted per design. And relative to the question of is material in locations where it hasn't been evaluated in design, as Charlie indicated, we have a program under my direction undergoing where we are inspecting all locations of the gap between buildings on site.

DR. HOFMAYER: There was one other item we left out on the cable trays, and that was the question that you raised about the holes drilled in the channels on cable tray supports.

As you recognize, it is an open item, and Cygna addressed this at some time. That is another matter that needs to be resolved. But again, I guess I would like to get a little more of your perspective of your knowledge of the facts of these holes.

Have you analyzed cable trays with big holes in them? Do you have any knowledge specifically that would help pinpoint where the potential problems are?

MR. WALSH: Well, when they drill holes, see, they don't use the high strength bolts to attach the trays to the channels. The Staff has permitted them to use A307 bolts there also. And then they take the bolt and they drill the hole in the channel, there is a reduction in the suction lines. And that -- I forget if it's over 15 percent.

When I did the Cygna, I had those two days over that weekend to look at it. I did a quick analysis in subtracting the hole, and I can't remember offhand what the results were, but it was a substantial amount, considering it's more than 15 percent of the flange area. And I haven't totally read or seen what Cygna has done, but it looks like they're looking at it very seriously.

Other parts of the hearing where they were using -- they were calling -- it's hard to say if they were plug welds or they were filling up misdrilled holes. I don't know if they did that on these cable tray supports. But the hole could be where the load is being applied by the Cygna in the case; in fact, their result on stress.

That is not considered by the Applicant for the reanalysis for Cygna in May of '84, did not consider the holes. I don't know if any of those beams or those channels would have been overstressed at those load points.

DR. HOFMAYER: The concern is clear. Again, I'm just bringing it up. I wanted to be sure there was not more to it.

MR. LIPINSKI: One of the things that was in that transcript was the concern about the ?an? plate be welded to liner plate and was overstressed -- I mean stressed at about 100 ksi if I remember right.

And then again we have to rely on your help because we went there and we looked there, and we couldn't find it.

MR. WALSH: Jack, do you remember that, those supports Jean was working on, and they told us not to model the plate that was attached to the liner? When Jean modeled it in, it was 100 ksi or something? Is that psi?

MR. DOYLE: Containment spray system?

MR. WALSH: May have been.

MR. LIPINSKI: On containment spray system, I remember that very well. And there is no -- to my knowledge at least, there is no (inaudible) at the

scrubber to liner plate. The member which is welded to the thick end portion of the liner plate is a built-up Y flange with the strong axis horizontal, with the web horizontal.

MR. WALSH: That's not the work I was talking about.

MR. LIPINSKI: That's what we saw. Then there is a vertical flange plate face that -- the end of that horizontal member, and the rectangular tubing is welded to that plate, and that is surrounding the vertical pipe which is a part of containment spray system.

Now, the only horizontal plate that we found was the plate that was supporting electrical conduits. That was a small plate about a quarter of an inch thick and about five to six inches in span, spanning from the liner plate. And the loading on the conduits are large, that it would be highly improbable that it would be stressed 100 ksi. So the basis for our difficulty, we cannot locate a plate. If you could be more specific then.

MR. WALSH: I'm going try to draw you a picture --

MR. LIPINSKI: All right.

MR. WALSH: -- of how I best remember how

this thing looked.

MR. LIPINSKI: We looked at the elevation close to the springline. I don't remember which elevation it is. There is a bridge going across the containment that went from one end of the bridge on the other. I looked at both sides of the containment.

MR. WALSH: At the time we were analyzing it, this is two tubes with 3/8ths inch plates -- I think it's 3/8ths.

MR. LIPINSKI: Is that a flange?

MR. WALSH: This is a containment line, this is an elevation they had further on.

This went between these tubes. They had another plate, I believe, that went like this that was part of the cantilever. I think it came out something like this. That was part of the -- that would be overstressed model.

DR. HOFMAYER: This is this liner plate?

MR. WALSH: Right. This plate here --

MS. ELLIS: Why don't you label each one of those.

MR. LIPINSKI: The elevation of this plate --

MR. POSLUSNY: This is off the record.

(Off the record)

MR. POSLUSNY: Back on the record.

DR. HOFMAYER: When you analyzed this, was this pipe that was not installed or were you analyzing as-built conditions?

MR. WALSH: That is for vendor certification.

DR. HOFMAYER: This was as-built conditions?

MR. WALSH: Yes.

DR. HOFMAYER: To the best of your recollection, does this contain spray system piping?

MR. WALSH: I don't know.

DR. HOFMAYER: As far as I know, that's the primary piping system --

MR. WALSH: Is that another one that starts with "vh" or something like that?

It was NPSI, if that helps narrow it.

DR. HOFMAYER: NPSI pipe support?

MR. WALSH: NPSI pipe support.

DR. HOFMAYER: Do you know whose pipe, whose piping?

Are you aware of any other -- Howard, are you aware of any other --

MR. LEVIN: Charlie, I'll look into it. To the best of my knowledge, you would be talking about

the containment spray line. But, you know, we'll take a look into it.

DR. HOFMAYER: As far as our review, we basically review the containment spray piping lines, look at all the detail. We don't see the detail. Angling up to the top of the containment, you know, looking up, see the supports, the lines go up vertically and then split and go up higher like this, like a u-shape in piping halfway up.

Are you saying that this type of support would be general support design for this entire line, or could it be one location?

MR. WALSH: There was more than one. Now, I don't know if there were ten of them like that; I can't say how many.

MR. LEVIN: Could you say when the line was installed?

MR. WALSH: Yes. The supports we were analyzing for NPSI were all as-built.

MR. LIPINSKI: Was that rectangular tubing?

MR. WALSH: Correct.

MR. LIPINSKI: Structural members?

MR. WALSH: Yes.

DR. HOFMAYER: Well, at this point I

think the next thing we need to do is look at the piping and look at the support detail to see if what you don't see visually shows up. As I say, that particular type of plate, you can observe the conduit. As a matter of fact, we saw some unused plate that looked like what you were describing. So beyond this point, I can't add anything to it. I hoped maybe you could add something to it.

MR. WALSH: It may be also taken down now because, you know, this is not a new issue. This has been out for over a year and a half. So, you know, the Applicants have not always been known to just come right out and say, "Yes, you were right. We're going to change this." And they could have gone out and just changed it.

MR. LIPINSKI: When did you see it? Maybe we can trace it down to what happened.

MR. WALSH: When I worked out there in '82.

MR. LIPINSKI: In '82. So in '82 it was still there. If it was taken down, it was after 1982 -- between 1983 and 1985?

MR. WALSH: It was before February of '82 because Gene was working with us at the time. He had analyzed it and gone back and analyzed it with a plate

in there. I took it over to NPSI and indicated to them what the stresses were, and they took like -- put a hold on it.

And it came back about a month later telling us not to model in that plate, that it was Gibbs and Hill's responsibility. And so we did not model it in, went back to NPSI.

MR. LIPINSKI: When was the last time that you knew of the existence of this plate?

The gentleman that you are talking about was after you left. Right?

MR. WALSH: No. He left in February.

MR. LIPINSKI: He left in February. And after that, you know that it was there or you don't know?

MR. WALSH: I didn't go back and pursue it. I gave it back to NPSI for their approval, whatever they wanted to do with it.

DR. HOFMAYER: Just one question: Was this the Unit 1 Containment?

MR. WALSH: Yes.

MR. LEVIN: Could you describe for me what you viewed your responsibility was in terms of your analysis, what your scope was?

MR. WALSH: At the time we were looking

at that, I was the group leader.

MR. LEVIN: And in terms of modeling these types of supports, what was it normally the practice of the STRUDL group to model?

MR. WALSH: Oh, just to use the best judgment -- I mean NPSI had their own design criteria which we were required to follow, and I'm not saying I agreed with it; I just did it. But something like that, it's part of the structure, so model a plate in, and that's what we did.

MR. LEVIN: The reason I asked that question, it was my understanding that there is basically a scope of supply change at that point. And as I understood the process, STRUDL group would transfer loads applicable to that interface to Gibbs and Hill for them to evaluate their own hardware.

MR. WALSH: The STRUDL group did not do that.

MR. LEVIN: I'm asking you to try to help clarify what, in fact, you did do.

MS. ELLIS: I think that's already been covered, Howard, in the hearings pretty thoroughly.

MR. WALSH: All I can say is the guy from NPSI told me that it was Gibbs and Hill's responsibility and they would take care of it.

MR. LEVIN: What I'm trying to find out is not how the process was supposed to work. It's my understanding that that, in fact, is so.

MR. WALSH: I don't know. If I can model in that plate and it's overstressed, so I don't care whose responsibility it is, it should be modeled in.

MR. LEVIN: At the time, did you have access to the as-built information from the point of view of the items within Gibbs and Hill's scope?

MR. WALSH: No. I did not have any listing like that. I was given a pipe support package to analyze, and I analyzed it. I mean, I didn't analyze the Containment Building when I did the support, if that's what you're getting at.

MR. LEVIN: No. You know, you modeled in hardware in that scope, and I'm just trying to verify whether that was on the basis of as-built information or if that could have anything to do with the stresses you calculated. Were you using current information for hardware beyond the scope of supply boundary?

MR. WALSH: I feel that any calculations that we were doing at the as-built or preliminary design or whatever you call is a calculation, and it doesn't matter what you call it. It has the same seriousness as if it was as-built or vendor certified or whatever term

is now being utilized. It was as-built.

MR. LEVIN: The important thing that I'm getting at, Mark, is that we want to be sure we're analyzing with the correct inputs and information, and I'm just asking you if you verified that you had it for this particular case?

MR. WALSH: At the time we were doing this, they were doing supports before they had the loads. And we could go back and look at it later, what the loads actually were. We were doing a calculation. If the thing was overstressed using the loads that we were given, it was overstressed. If those loads are final or preliminary, they were loads.

MR. LEVIN: Okay, granted that. But whether or not it's overstressed today may be dependent upon whether the configuration is the same, whether the input information you were using was the same, and that's why I think it's important to know -- as I think the NRC Staff is trying to ascertain -- where you were looking at and what its configuration may have been at that time because that will have an impact on the stresses you calculated as compared to what we might calculate today.

MR. LIPINSKI: Were you an employee of Gibbs and Hill or --

MR. WALSH: Neither. I was working for PDS under the direction of TUSI.

DR. HOFMAYER: At this point, it might be difficult to find out, but we can analyze what's there. Okay? I can't guarantee we'll find it, but we haven't seen it. The design could have changed. It presumably could be traced back to see if it was there and it was changed.

In terms of the final process, if we did go back and reanalyze, you know, indirectly, there's certainly nothing wrong with that. I'm not sure where the decision goes.

MR. LIPINSKI: I would like to tell you that we were there twice, and we looked all over the place for the plate and we couldn't find it.

MR. WALSH: Is there a "V"? For some reason I think it's VX, VS system.

MR. LIPINSKI: VS system?

MR. WALSH: CASE accidentally got a drawing with it on there at the time we were doing so much. I don't recall what the system is, but I did see it on one of the drawings that we received.

MR. LIPINSKI: And that was on that drawing?

MR. WALSH: The system and the plan; it

was not the particular support. I recognize something like that.

MR. LIPINSKI: As was said before, we'll try our best to find it. If we find the records of this, at least we'll know where it went. That's it.

MR. POSLUSNY: Ms. Ellis, do you have anything else to add?

MS. ELLIS: We will be getting any information that you receive -- I assume we have been getting it and will keep getting any information that you provided the Staff?

MR. BECK: Ms. Ellis, perhaps this is a good time for me to be responsive to your thrice-repeated request for information regarding organization, as I understand it. As we've said, we're developing a comprehensive response program. Integral to that response program is an organization. The organization plan that we would give you last week isn't necessarily the same that will be coming forth when we finished our plan development.

It would be probably, certainly not in our best interest or in yours, to give you an incomplete piece. I have thought very carefully about a preliminary submittal, and I just don't think it would be in either one of our interests for you to go down the

road that would ultimately end up not being the one that we would advocate traveling a few weeks later, for example.

I just want to assure you that I appreciate your concerns about organization, about people, about their qualifications, and that our response plan, when it's submitted, will be comprehensive in that regard in outlining who is doing what, what their responsibilities are, what their qualifications are, what their previous involvement has been.

Our tact is to clearly involve in any instance where a third party, what we characterize third party people, that they not be previously involved in any way in the areas that they're charged with investigation or examining, and that will stand the test of examination.

So I don't want you to think that we're not being responsive to your earnest desire for information. It will all be there. It's just that I think it will be best for you to see the whole thing at once rather than dribble it in.

MS. ELLIS: Okay. Great! I appreciate that, John.

And let's see. I have one other question. The information we had received from a meeting with Cygna -- I guess it was on the 14th -- was that they

were going to be sending a letter on the cinched U-bolts. Is that incorrect?

MR. BECK: That was my understanding. I haven't seen anything yet. I'm sure when it comes, we'll all see it at once.

MS. ELLIS: Okay.

There is one other thing about the Cygna information that I think would be helpful to us as far as getting information, especially as we're getting into winding down on some of these issues; that is, a lot of times we've received information from Cygna where you have sent them information and then they turn around later and send it to us.

I think it would be helpful and save a lot of time if y'all could send us the information direct, as you do when you send things to the Staff, start sending us the same things that you send Cygna at the same time. If that would be possible, I think that would help.

I guess that's about it for right now. I want to say again that I think this was a very productive sort of get-together, and we appreciate the opportunity. And I want to say again that I certainly don't envy Howard.

And I also appreciate all of the efforts that the Staff's people have been putting into all of this,

and I know it's a very difficult thing to try to go back through transcripts and through all the documents that you need to, to find out all the details that you need. We realize it is a mammoth effort, and we appreciate that effort. And while we may not always agree on the final results of some of these things, we do feel that the efforts of the Staff are much, much improved over what they were before, and we appreciate that.

MR. POSLUSNY: Appreciate the comments.

Mr. Walsh and Mr. Doyle, thank you again for the meeting. I think it was very productive.

Mr. Beck?

MR. BECK: I want to thank Mr. Walsh, Mr. Doyle, and Ms. Ellis, particularly you, Jack, for having traveled as far as you did. And we certainly will look at everything in the detail that you expect.

MR. DOYLE: Appreciate that.

MR. POSLUSNY: Thank you very much.

(The meeting was concluded at 6:50 p.m.)

(1) STABILITY

(a) BOX FRAME STR/SNUBB

(b) 'U' BOLT STR/SNUBB

5 (c) 'U' BOLT (2) STRUT AXIAL (TTAF)

(d) CLAMP ... ... TO ACTION

(e) STR FRAMES ... HGR STRUT SPIN (CC-...-716 A63)

FIXES

(a) BRACKETS TO STOP STRUT ... PRIOR TO BUMPERS

(b) CLIP ANGLE TO HOLD U BOLTS (BOX FR)

(c) LUGS (STOP AXIAL BUT NOT OTHERWISE)

7 (d) DOUBLE STRUT FOR BOX FR

(e) BUMPERS TO REPLACE STRUT STOPS

(f) SHIMS TO 0 GAP (THERMAL CONST)

(g) CINCHING 'U' BOLTS

(2) LOADS ON SPT NOT ACCTED FOR

(a) SWE HARDWARE

(b) SWING ∠ OF STRUTS & SNUBBERS

(c) FRICTION (1/10")

(d) LOAD Δ FOR SNUBBERS/STRUTS (CC-008-019)

7 (e) ACTUAL SECTION PROP (1 1/2 ... HOLE FOR RICH)

(f) HANGING SUPPORTS OFF RICH BOLTS

~~(g) SHEAR DISTRIBUTION ... RICH IN BRG~~

(g) (h) SWE STRUCT. ANY DIRECT.

~~(i) ... MOMENT ... ...~~

~~(j) INCORRECT TORSIONAL COUPLING~~

(3) HARDWARE

(a) DIFF LOCKUP OF SNUBBERS (... PART ... ...)

9 (b) CROSS BARS (NOT TUBES) FOR 'U' BOLTS (CC-008-006) 75% SPAN 19 3/4 X 3" OVER...

(c) DOUBLE AXIAL RESTR (STR/SNUBBERS) LOAD DIST REL TO STIFF.

(d) THERMAL ROT Ay (RAD) LOAD REDISTRIBUTION

(4) RICHMONDS

(a) EXCESS Δ (ALTERS STIFF SPT)

(b) BENDING IN BOLT

6 (c) BRG JOINT NOT ACCEPT FOR DYNAMICS

(d) SHEAR LOADS NOT BASED ON PROPER DIST/BRG ST

(e) M_e MOM PRYING ACTION NOT CONSID

(f) INCORRECT PROCEED FOR COUPLING TORSION TO BOLT

(5) LOCAL EFFECTS NOT CONSID. (HYATT REGENCY K.C. 114)
(a) 16"Ø DIAPH FAILURE - CC-02-709 FAIL OF CMPX
(b) TUBE WALL EXEX14 FAILURE CC-116-03A " OF COMPOSITE
(c) FAIL W6X12 CC-028-039 S33R
(d) FAIL PLATE/W UBOLT (SEE ABOVE) 3X3/4 14"± PAN 7600#/LD OVER T
11 (e) TORSIONAL FAILURE SPT/ON COL FOR CC-197-00E

ADDITIONAL AREAS

(f) CALC OF RIGID H/D INSIGNIFICANT TO SHOW SPT BY FLEX FORM.
(g) CLOSED ∠ < 60° GROOVE WELDS RATIO F/D 1→1.7 INTERN CRACKING
(h) CALC FOR FAILS INTEG WITH PIPE
(i) CALC OF WRAPPING MODE NOT CONSID - FLEX FORM. (FAILURE IS IN SIDE WALL
(j) PUNCHING SHEAR NOT ALWAYS USED (APPL SAY WELD COMP)
(k) SHORT TUBES ON BASE & LOAD PATH BENDING OF SIDE WALLS

(6) 'U' BOLT AS 2 & 3 WAY
(a) CAN SEE 2 WAY LOADS
3 (b) WITH 2 WAY LOADS FRICTION NEED TO BE ACCOUNTED FOR
(c) WITH 2 WAY " " & BINDING "

(7) CINCHED DOWN 'U' BOLTS
(a) LOAD EXCEEDS MFG LDS ALLOW.
3 (b) PULL THRU ON T'BES NOT CALC (I.C. HYATT)
(c) ~~REVISES~~

8 LOADS ON PIPE EFFECTING LOCAL STRESS & STRAIN OUTPUT
(a) ½ HARDWARE SNUB STRUT - CLAMP ETC
(b) STRUCT - U' BOLT & STRUCT, BOX RHS, VF ETC
(c) KICK LOADS ANGULAR STRUTS
(d) CINCHED UP 'U' BOLT EFFECTS
(e) BOX FRAME CONSTRAINTS
(f) ANCHORS LOCKING THERMAL GROWTH
(g) EFFECTS OF TRUNNIONS /PAD OR NO

(9) BUILDING LOADS ON SPT MEMBERS
2 (a) WALL TO WALL, / WALL TO FLOOR/LINE ETC D.L. (BECOME BASE COL)
(b) SEISMIC RESTRAINT

(10) STIFFNESS
1 (a) ACTUAL STIFFNESS V.S. GENERIC OR LOWER GENERIC

11 UNDERSIZE WELDS
(I) SINCE PREV. OF CRACKING IS PREMISE (VOL EXAM REQ'D)

(12) CRACK PROPAGATION

(a) WELDS THAT VIOL. BOTH REQ ARE SUBJECT TO CRACK PROP. WITHOUT VOL. EXAM NEGLECTING AREA OUTSIDE B IS NOT ANS.

(13) HIGH LOADS ON PIPE LINE WITH SPT

(a) BOTHLINE PROBLEM FOR PIPE .99 $R_i^{1/4} b^{-1/2} \delta^{-1/2}$ = CIRC ST.

(b) COLLAPSE CHECK OF WEB WALL DUE TO COMPARATIVE LOAD

(14) APPENDIX XI

(a) THIS WAS POINTED OUT IN DEFOE BUT NEVER ANSWERED APPL.

(15) CALC.

(a) CYGNA HAS FOUND ~78% OF CALCS DONE NOT STAND ALONE

(b) I HAVE FOUND MOST OF APPL CALCS ARE INCORRECT IN THERE FAVOR - V.I.P. — HIGH REST k FACTORS, DIAPH, TORSIONAL ETC.

ENCLOSURE 3

MEETING ATTENDANCE

March 23, 1985

MEETING BETWEEN CASE, THE COMANCHE PEAK RESPONSE TEAM AND THE NRC STAFF RELATING TO THE CONCERNS OF MESSRS. WALSH AND DOYLE

NRC

Robert J. Bosnak
David Terao
Chester Poslusny
John R. Fair
Romuald E. Lipinski
W. Paul Chen, ETEC
Donald F, Landers, Teledyne
Charles Hofmayer, Brookhaven

CASE

Juanita Ellis
Mark Walsh
Jack Doyle
Jerry Ellis
Barbara Boltz

Texas Utilities

John W. Beck, TUGCO
Howard Levin, TERA
Douglas M. Witt, TERA