Docket # 50-275
Control # 8706010217
Date 5/12/87 ~~of Document~~ Memo
REGULATORY DOCKET FILE

ORIGINAL

Attachment 3

UNITED STATES NUCLEAR REGULATORY COMMISSION

IN THE MATTER OF: DOCKET NO: 50-275
50-323

MEETING RE: DIABLO CANYON
SPENT FUEL POOL RERACKING

LOCATION: BETHESDA, MARYLAND PAGES: 1 - 121

DATE: WEDNESDAY, MAY 6, 1987

ACE-FEDERAL REPORTERS, INC.

Official Reporters
444 North Capitol Street
Washington, D.C. 20001
(202) 347-3700

NATIONWIDE COVERAGE

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UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

MEETING RE:

DIABLO CANYON SPENT FUEL POOL RERACKING

DOCKET NUMBERS 50-275 AND 50-323

Nuclear Regulatory Commission
Room P-110
Phillips Building
7920 Norfolk Avenue
Bethesda, Maryland

Wednesday, May 6, 1987

The meeting convened at 9:15 a.m., Mr. C. Trammell presiding.

PRESENT:

C. TRAMMELL, NRC, Project Manager
B. VOGLER, NRC/OGC
M. TRESLER, PG&E
S. BHATTACHARYA, PG&E
J. MARTORE, PG&E
B. PAUL, Holtec
C. COFFER, PG&E
D. JENG, NRC
K. SINGH, Holtec
G. BAGCHI, NRC
H. FISHMAN, FRC
G. DeGRASSI, Brookhaven
H. ASCHAR, NRC
A. SOLER, Holtec

PROCEEDINGS

MR. TRAMMELL: I am Charlie Trammell, newly assigned NRC Project Manager for Diablo Canyon.

This is a meeting, a final meeting, regarding the spent fuel pool and some matters that were attached to a meeting announcement the 29th of April. The agenda for this meeting was as attached to that meeting announcement.

Before we begin, I would like to have everybody go around the table and announce who they are and what their position is so that everyone knows everyone here.

Can we start with you?

MR. VOGLER: I am Ben Vogler, Office of the General Counsel.

MR. TRESLER: I am Mike Tresler, Project Engineer on Diablo Canyon for PG&E.

MR. BHATTACHARYA: I am Shan Bhattacharya, Senior Civil Engineer, NRR.

MR. MATORE: J. Matore, Consultant to PG&E.

MR. PAUL: B. Paul. I am consultant to Holtec.

MR. COFFER: Charles Coffer, Supervisor, Nuclear Regulatory Affairs of PG&E.

MR. JENG: I am David Jeng, Section Chief, NRR.

MR. FISHMAN: I am Howard Fishman, Consultant for the NRC.

MR. DE GRASSI: G. DeGrassi. I am consultant to

the NRC, Brookhaven National Lab.

MR. ASCHAR: H. Aschar, Structural Uses Branch, NRR.

MR. SOLER: A. Soler, Vice President - Engineering, Holtec.

MR. TRAMMELL: I am passing around a piece of paper. Would you put your name and affiliation on that, and I will give a copy to the reporter, and I will also give you a copy at the first break.

This meeting is being transcribed, as you can see, and the reason it is being transcribed is that the Intervenor and its attorney could not be present, and so we are recording this meeting.

If any portions of this -- if there are any portions of this meeting which are proprietary to PG&E, I would ask that they please identify when we reach those portions so that we may have two editions of this transcript, one proprietary and one nonproprietary.

With that introduction, I think I am ready to turn the meeting over to PG&E, who stands ready to respond to our questions.

MR. COFFER: I thank you, Charlie.

As you mentioned earlier, the meeting is to respond to questions asked by the NRC Staff after PG&E provided a submittal with respect to reracking.

After we received the additional information request, we spoke to NRC Staff and reviewers to make sure we understood the questions, and at that time, at the end of, that particular telephone call, we were told that there were no additional items that were needed to be discussed in this meeting.

Accordingly, we have prepared a presentation to answer those particular questions. Since this is being transcribed, we will try to format the meeting such that each question will be answered as a read statement.

We would like to have it so that we finish the statement before there is any discussion, and then after the statement we can be open to questions, doing away with each question and then going to the next statement.

Alan Soler will be presenting the major portion of this. We have some overheads, and I will supply the recorder with copies of those overheads at the end of the meeting.

Mike, do you have anything you want to add to that?

MR. TRESLER: No. Simply, I would like to reinforce Chuck's comments on these questions.

We do understand that this is the last meeting, as you stated, Charlie, and we certainly want to provide all information so that it indeed can be the last meeting.

And even though in our phone call there were no additional questions, if any arise today, we certainly encourage people to get them on the table, and we would intend to be fully responsive today so that we can all complete the work necessary to support the hearing date that we have.

MR. TRAMMELL: Mr. Chandler advises me this is not the last meeting. This is the final meeting. As a lawyer, he made a distinction between those two terms, which I did not really fully appreciate until he explained the difference.

This is the final meeting.

MR. TRESLER: I will accept final.

MR. COFFER: Alan.

MR. SOLER: What I will do is read the question first and then read the response, and when I put an overhead up, I will have a slight pause while you attempt to absorb it.

The first question:

Provide a clarification of usages of various spring constants for conservative and realistic assumptions as related to the calculated values.

In this respect the reference is made to page 229A of Reference 2, Tables 3-1 and 3-2 of Reference 4, and page 2 of Reference 5.

Also, provide the method used to calculate spring constants at girdle bars and baseplates.

Firstly, Table 1 provides a comparison of the calculated spring constants and those used in the realistic and conservative rack models. The original design basis calculations intentionally use spring rates in different areas of the rack models which were higher than the actual values. This was done to conservatively amplify the peak values of the forces obtained in the various springs.

For example, stiff springs simulating the support feet were conservatively calculated based on the classical solution for a point load on a semi-infinite half-space.

Similarly, fuel assembly to rack cell wall impact spring constants were calculated using an analytical procedure plate theory and then increased by a factor of 10 to provide for conservatism.

Rack-to-rack girdle bar and baseplate spring constants were set at a high value, which simulated the effect of impact of two rigid bodies.

The realistic model in subsequent design studies on single and multi-rack configurations in 2-D and 3-D models, the spring constants were revised to values reflecting the actual calculated results or reflecting additional calculations.

For example, the support foot spring rate was

based on a solution for the load at the corner of an elastic plate resting on a classical elastic foundation. The elastic foundation constant used was based on the behavior of the grid work when subjected to a uniform loading.

This calculation resulted in a support foot spring constant lower than the spring constant used in the design basis calculations. This more realistic spring constant was used in the subsequent studies as a lower bound value to maximize the rack vertical and rocking behavior.

The fuel assembly to rack constants used in the realistic analysis reflected the actual calculated values increased by a factor of 1.5 instead of 10.

Finally, rack-to-rack or rack-to-wall impact spring rates were based on calculation models rather than usually using a very large value.

The girdle bar stiffness was calculated using the theory of a beam on an elastic foundation.

The flexibility of the gap channels connecting two rows of cells were used to calculate the elastic foundation modulus. The baseplate stiffness was set at twice the stiffness of the girdle bar to account for in-plane rigidity.

That ends Question 1. Do you wish to comment on that, or should I proceed?

MR. ASCHAR: After each question, I think we

should discuss them.

MR. TRAMMELL: That is a good idea because, as Chuck suggested, the record would be tied together a lot better. So let's take them one at a time, and then we will ask questions and then go on to the next one.

(Pause.)

MR. TRAMMELL: If you would like to confer, go ahead and take your time.

MR. ASCHAR: Do you have calculations somewhere in the record that you will show us later on and how you calculated the spring constants for the girdle bars and the baseplates, the numerical values?

MR. SOLER: The numerical values that we ended up with are right there. The actual method has been documented, and you will have to tell me, Mike.

MR. SINGH: The seismic report which was audited by the NRC last year in March of '86 --

MR. SOLER: That is for the conservative model.

MR. SINGH: That has the spring calculations for the girdle bar.

MR. SOLER: He is talking about the new one.

MR. SINGH: The new one.

MR. TRESLER: What we have is a description of the methodology used to calculate the stiffness of the girdle bar and then the stiffness we used for the baseplate

driven off that calculation for the girdle bar.

The actual calculations we did not bring with us. We did not understand this to be an audit of calculations like we have had in the past.

I guess I am having a little bit of difficulty understanding exactly what you would like. Would you like more discussion of the theory, or would you like us to somehow try and get copies of calculations?

MR. DE GRASSI: To our knowledge, the calculations are not documented in the seismic report.

MR. TRESLER: For the realistic model the calculations are not in the seismic report. The methodology used to calculate the spring constants for the realistic model analysis is as described by Alan just now.

Now, if we need more discussion of that methodology, I think you are prepared to do that.

MR. ASCHAR: Let me rephrase the whole question. The way we understood, there are calculated values for girdle bar and baseplates, and what we were thinking, the way it had been presented before, that for conservative assumptions you multiply by 10 times. You make them stiff springs.

The realistic ones, they are 1.5 times the calculated value.

MR. SOLER: You are a little lost.

MR. TRESLER: Let him finish.

MR. ASCHAR: I want the explanation of why the separate realistic calculations are needed. What we are looking for, what are the calculated values and how they are calculated?

Alan did tell us the method used, and it is in the report, too, the method used, but we did not see any numerical calculations in the seismic report, as Dr. Singh pointed out just now.

MR. TRESLER: Okay. Alan, why don't you explain how we arrived at the conservative -- now, we are talking girdle bar and baseplate, right?

MR. ASCHAR: That is right.

MR. SOLER: That is what I wanted to clarify.

The girdle bar spring constants in the conservative model, which is the one discussed in the original seismic report, they were set at a high value, sufficient to simulate behavior of two rigid bodies hitting together.

In the realistic model those values were calculated by looking at the structural details of the actual grid work behind the girdle bar, applying a load to that analytically, and calculating the deflection under that load, and because the actual grid work model is, let's say, for lack of a better word, a jagged beam, you can calculate

a flexibility by matching two different beams with an offset, applying a load and using classical beam theory to get the load deformation relationship.

That formula that evolved was then used to get the numbers for the girdle bar spring constant. That particular value that came out of that formula was then increased by a small fraction to bring it up to the .5 times 10 to the 5th value, which is shown in that realistic value. Calculated value was .36 E to the 5th.

The baseplate, which is simply a flat plate, roughly 1 by 100 inches, 5/8th of an inch deep, there was an estimate of EA over L type of calculation for that, but in the realistic analysis we doubled the calculated girdle bar stiffness to get a value that was used in the various numerical calculations.

Now, the fuel-to-rack impact springs, in the conservative analysis they were calculated by using a beam on an elastic foundation type of calculation, which is outlined in our seismic report, the original seismic report, and then for conservatism the values that we attained from that calculation were increased by a factor of 10.

In the realistic model the same calculation values were used, but an increase of only 1.5 was applied instead of 10.

In the support feet calculation, a method of

determining the flexibility of the grid work was based on a model of a point load on a semi-infinite half-space. That resulted in the .515 times E to the 7th model.

In the realistic model, a method based on a plate on an elastic foundation where the elastic foundation was calculated based on applying a uniform load to the foundation resulted in a calculated value of .061 times 10 to the 6th, and that value was used without any amplification.

The friction spring value is set at a very large number, sufficient to simulate the stick slip condition but not sufficiently high to cause any numerical problem.

I think I have covered all of the springs there on the table.

MR. FISHMAN: These realistic and conservative values, how are they incorporated in the 2-D and the 3-D models?

MR. SOLER: Can you elaborate on your question as to what do you mean by how are they incorporated? Where are they located?

MR. FISHMAN: You defined a realistic column and a conservative column.

MR. SOLER: Yes.

MR. FISHMAN: You have made three sets of runs, essentially -- the original design basis and then the 2-D

set and then the 3-D, the latest three.

MR. SOLER: All right.

The original design basis, which was a 3-D model, used springs in the last column marked "Conservative." The 2-D runs used the springs in the "Realistic" column, with appropriate doubling of values, since in a 2-D model one spring really represents the effect of two.

The final set of runs, which was the 3-D realistic model, was simply the original 3-D model with simply the values replaced spring by spring where appropriate. But the model itself was the same.

MR. ASCHAR: We understand what you have done with the calculated values better than what we understood before.

However, what I would request from PG&E is to provide us with calculations for those two springs only, the girdle bar spring and the baseplate.

MR. TRESLER: Okay.

MR. ASCHAR: If it has been provided before in any fashion to the NRC, please tell us where.

MR. TRESLER: Your request is that we provide you with copies of the calculations for the realistic girdle bar and baseplate springs?

MR. ASCHAR: The way I understand it, once you give us the calculated values we should be able to figure

out what they are used for, the figures that are right here in the realistic.

MR. TRESLER: The question is for realistic?

MR. FISHMAN: No, calculated.

MR. TRESLER: Calculated realistic -- calculated, I am sorry.

MR. ASCHAR: Calculated is what we are looking for.

MR. TRESLER: Calculated, okay.

MR. TRAMMELL: The clarity of the transcript would be enhanced if you would try to speak only one at a time. It is difficult, and if you would take special care to try to do that on the transcript it would enhance what we have for a record.

Thank you.

MR. JENG: I have a few questions.

Number one, the conservative spring constants used in the multiple analysis are identical to those used in the single 3-D analysis?

MR. SOLER: The values in the last column are those from the original design basis report.

MR. JENG: So they are identical?

MR. SOLER: Yes.

MR. JENG: Second question.

In the case of a fuel-to-rack interaction spring,

you earlier mentioned that the realistic one was multiplied by 1.5.

MR. SOLER: Yes.

MR. JENG: Can you tell me what is the basis?

And the second part is: if one were not to multiply by 1.5, what would the consequence show? Would that reduce the loading, increase the deformation? What are the implications?

MR. SOLER: Let me answer the last question first. That is the simplest answer.

Our studies have shown when you look at our conservative, which has a high value of fuel-to-rack impact springs, and realistic, which has a lower value of fuel-to-rack impact springs, is that the forces between the fuel in the cell due to the impacts are directly proportional to the values that we use for those spring constants.

So the answer to your last question, if I did not multiply by 1.5, those forces would be even lower between rack and fuel.

The springs that affect the deformation of the structure, the rocking behavior in the translation, are mostly the floor springs and to some extent the rack-to-rack springs.

Now, I guess you better repeat some of your question.

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MR. JENG: The question is if we are naming this particular number the realistic spring, and why we are multiplying 1.5, if you feel that your way of calculating the number is the most pertinent, unless you have some other reason to cover another situation?

MR. SOLER: The 1.5 calculation was simply used to cover uncertainties and knowing the material properties, not the calculation method.

So since anything that would make stiffer will lead us to higher forces, and therefore that was a multiplication of 1.5 to increase the value that was calculated rather than, say, let's reduce it even further because the material properties are not enough.

MR. TRESLER: I think one of the reasons is that we feel that we still want to leave conservatism in what we represent as being the realistic model, and by using 1.5 we have allowed for some conservatism to remain.

MR. JENG: That is pertinent only if you can tell me if one wants to account for the over-protection of the material properties and to introduce the lower number, and that would not lead to any significant increase in the deformation aspect and therefore you are containing the conservative approach overall if you use the 1.0 computed value because of the uncertainty in the property.

You told me that you want to cover the

uncertainty.

MR. SOLER: If I use a lower value than the computed ones, 10 to the 5th or 1.1, if I use the lower value, what would happen, there would be essentially no changes in the rack behavior, deformation racking. The change that you would see is in the predicted maximum force between the rack and the fuel assembly, which would decrease even further.

MR. JENG: Your statement that there would be no changes in the deformation, is that based on your judgment, or is that based upon some --

MR. SOLER: Two sets of calculations using high and using low values.

MR. JENG: The next question.

In regard to the stiffness, you explained earlier that you did different approaches for the general one and the corner one.

Could you restate what did you do in these two cases?

MR. SOLER: And the initial design basis calculation -- just let me back off a little bit -- the calculation of that foot spring constant is really obtained by looking at three springs in series. The three springs being the spring representing the grid work, the spring representing the support foot itself and a spring, if any, representing this flexibility of the liner and the concrete floor.

Because of the way springs in series add up, the one that has the smallest value, if it is considerably smaller than the other two, essentially governs the results of the calculations.

In the design basis calculation, you find that the value that governs the final result that you get is essentially the flexibility of the grid work above the support links. The method of calculating that in the design basis report was to use the -- to start from the solution of a foot point load on an elastic half space, adjusting the half space to reflect that it is not a continuous media, but it is really grid work. That was done by using an effective Young's modulus.

That calculation led to the value .515 x 10 to the 7th, which was not adjusted in any way.

In the analysis that we have done subsequently, in response to NRC's questions over various periods of time,

we have taken a solution for a patch of loading on the corner of a plate resting on an elastic foundation. Based on our previous experience, we don't even worry about the EA over L of the support foot, because that is so large that just does not play a role when you add the springs up.

So in our more realistic analysis, we have concentrated simply on the calculation of the spring constant due to the grid work. And we look at the solution of the plate on an elastic foundation, which is loaded by a pressure patch on the corner, circular pressure patch on the corner.

To get that solution, you need to get a foundation modulus for that plate. And the method used to get that foundation modulus was to look at the -- analytically, to look at the grid work supported by a uniform loading P zero and calculating what the deflection of that grid work should be, taking into account the actual area that is used, as opposed to the area of the continuous block, and that value, for the foundation modulus was then put into the solution for the corner loaded plate. P versus delta value was obtained and that value was .06 x 10 to the 6th, which again was not adjusted when it was put into the model. And we deliberately attempted to get a soft value to maximize the displacements that one would get.

MR. JENG: So in the second method, if you were

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to use the first method, meaning a single load on the elastic approach for the second corner consideration, would that have changed your outcome appreciably or not?

MR. SOLER: You get back to a very stiff spring constant.

MR. JENG: Which means larger forces?

MR. SOLER: It goes back to the design basis.

MR. JENG: So the idea is to account for more softer situation in the hope that you would bound the worst of boundary?

MR. SOLER: The runs we have done have shown that the softer the spring values at the feet, the higher the displacements, both vertically, translational and rocking that evolve when you run through the mathematics.

MR. JENG: Thank you.

MR. DE GRASSI: The latest study performed seems to indicate a trend of higher rack-to-rack and rack-to-wall impacts, when you went through the realistic parameter model. Can you provide a rationale for that?

MR. COFFER: Before you do that, I am not really sure that applies to this particular question, and if there are some general questions of that nature, we certainly are prepared to answer them. But could we focus on the particular questions, the six questions, first and then go into the general aspect afterwards?

MR. ASCHAR: What happens is that this is our presumptions that the basic values are the same constants. That is why this question is related to the spring constant more than any other thing that we can think of. There could be some other reasons we don't know. It is relevant, but if you want to save it till later, I have no problem.

MR. TRESLER: That would be our preference, to deal with it later.

MR. TRAMMELL: Write it down.

MR. TRESLER: We will not forget it.

MR. SOLER: Are you ready for the second question?

"Provide a discussion of rack-to-wall gap size used in the direction of motion and the calculation of hydrodynamic coupling for all cases in Reference 4. Include in the discussion the results of 2-D multirack analysis, which we understand have been performed using real wall-to-rack gap 4 to 5 inches."

PG&E submittal, reference 4 results for a single rack model, 10 x 10 module and for two different rack arrays identified as sections AA and BB, as shown in Figure 3 of Reference 4, the nominal wall-to-rack gap for a 10 x 10 module is 9 inches. Rack No A-2. The corresponding gaps for the peripheral racks studied in the multirack analysis are shown in Table 2.

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(Slide.)

In the analyses reported in Refernce 4, the rack-to-wall gap was conservatively set equal to 3 inches to reflect the worst case condition. The hydrodynamic coupling calculation used 3-inch gaps, and the girdle bar, and base place impact springs used gaps of 2 inches, which allowed for the thickness of the girdle bar or the projection of the base plate.

The multirack analysis previously performed by PG&E, DC87-022, Reference 6, used the actual rack-to-wall gap. The model utilized 2 degrees of freedom per module and studied multirack behavior for the coefficient of friction of .2. The results showed the racks did not impact the wall. These results were further reviewed when the boundary condtions for the 4 degree of freedom model were established.

It was PG&E's judgment that use of the worst case rack-to-wall gap of 3 inches rather than the actual gaps will increase the likelihood of wall impact and provide conservative wall impact loads.

While it is true that the reduction in the gap increases the hydrodynamic mass and thereby, the coupling coefficient, such an increase is not significant as explained below. The calculation of hydrodynamic coupling effects in the 2-D analyses can be shown to be governed by

an effective hydrodynamic gap, the formula for which is 1 divided by G effective is equal to 1 divided by G subleft plus 1 divided by G subright. Where G subleft and G subright are the gaps to the left and the right, respectively, of the rack under study in the 2-D analysis of multiple racks, DCL 87-022, the assumed motion of adjacent racks leads to a value of G right equal 1.125 inches and G left equals 5.125 inches or 9 inches, depending on the particular rack under study.

This leads to the following calculated effective gap values for hydrodynamic computations. G effective equals .923 inches for a 5.125 inch space into the wall and G effective equals 1.0 inch for a 9-inch spacing into the wall. In the NRC requested 2-D parametric studies, Reference 4. The value of G subleft was taken as 3.0 inches, in order to insure that the simulation would result in a wall impact. Earlier results commented on above indicated no wall impacts occur, if a larger spacing is used. Using a 3-inch gap in the calculation of G effective leads to G effective equals .818 inches.

The effect of using a wall gap of 3 inch in lieu of 9 inch leads to minimal changes and the fluid coupling mass terms; however, should the more realistic but still conservative lateral gap turn, H sub 0 of 3.6 inches be used in conjunction with a 9-inch wall spacing, the hydrodynamic

mass terms will actually increase when compared to the values calculated using a 3-inch wall gap with an H zero of 7.5 inches.

It should be noted that any E increase in the values of the hydrodynamic mass terms results in a decrease in rank displacements, rotations and forces. These relationships clearly demonstrate the use of conservative gaps to maximize rack wall impacts, do not significantly affect hydrodynamic coupling terms.

MR. ASCHAR: Your explanation gives me some insight into what happens when the wall gaps are increased from what has been used. What I can see as a problem is when have the realistic springs that we have used before in our analysis and higher wall gaps, and if you do not decrease the hydrodynamic coupling effect from 7.5 inches that you have used to 3.5 inches or something, our feeling is that the impact loads that you will see might be higher, as you go through from 3.5 to 7.5 inches. There would be a point versus the wall gap that you assume in the direction of motion, where both would have the similar effect.

We have been thinking about your use of 3.5 inches as a 1.3 times, considering that consideration; however, it -- we do not know the validity of it experimentally or any other way, so we are trying to keep it as conservative as possible. But still giving it a

realistic flavor. And we think that the 7.5 inches is a good number to stick to for most of the realistic calculations.

Now my question is, if you use that wall gap size and the 7.5 inches for the spacing, the hydrodynamic coupling effect, how would this evolve?

MR. SOLER: Let me answer that as follows:

In the 2-D multirack analysis, which considered four racks with four degrees of freedom per rack, so we are talking about a 16 x 16 mass matrix. If you use a 3-inch gap at the two walls instead of a 9-inch gap at one wall and a 5.125 inch gap at the other wall, the total number of terms affected in that whole 16 x 16 mass matrix are two terms that correspond to what arises from the leftmost rack, the 9-inch wall, and two terms, the lower right-hand end of the mass matrix. Those terms do decrease by a small amoutn.

The effect will be, if you use the actual gaps, some slight increase, possibly in a rack-to-rack impact force, the next one over, but you will not, because of the nine-inch spacing, get a wall impact in that case, and we deliberately set out in those models, since we wanted to get a wall intact, with a low wall spacing, and it was our judgment that the slight decrease in those coefficients, four of them, out of 16 x 16, would not significantly affect the rack-to-rack impact between the first and second rack,

which, if my memory serves me correctly, was not the governing rack to rack impact in that multirack model. I believe it was between the second and third that you actually got the highest impact loads over the course of the time history.

The statements that I have made concerning the change of 7-1/2 to 3-1/2, no calculations of rack analyses were actually done and reported using that. It was simply a calculation of what the coefficients would be if, in fact, you did decrease that 7-1/2. And all we are trying to see there is that the effect of that side gap value far overshadows the effect of the 9 inches or 5-1/2 inches, simply because of the way that you calculate the effective gap.

MR. DE GRASSI: I would like to ask a clarification on that formula that you mentioned. 1 over GEX = 1 over GL 1 over GR.

Can you clarify what the GL and GR gaps are? Are they the gaps that are opening and closing while the rack is moving in one direction, or are they the racks -- the gaps to the side?

MR. SOLER: They are the nominal gaps from the rack side up. They are the nominal gaps in the -- perpendicular to the direction of motion. In other words, the 7-1/2 inch gap, which I have called H zero, is the gap

between adjacent lines of racks. The 9 inches or the 5-1/4 or the 3, the numbers that have been used here represent the gaps at either end of the one line of four racks that we are studying. And they represent the distance from the side of the rack to the wall.

MR. SINGH: Slip of tongue. The transcript should be corrected. If the gap in the direction of motion at the two ends, not perpendicular. The end gaps.

MR. SOLER: The gap -- the channel is perpendicular to the direction of motion.

MR. DE GRASSI: Correct.

MR. ASCHAR: You did indicate that if you increase the gap, and if you keep the 7.5 inches, you might see some increase in the impact loads.

Do you have any estimate as to what analysis you perform on those things as to what the slight increase amounts to? How much?

MR. SOLER: Based on the analyses, taken as a whole, that have been done on this particular project, and looking at the amount that those few coefficients change in the mass matrix, my estimate would be that the slight increase in rack to rack impact that you would get in that one location would not exceed the values that we have predicted for that entire model over the course of the time history. The maximum value being predicted in another

location.

MR. JENG: To follow up that point. You said that your estimate indicates they would not exceed the maximum basis predicted for that model analysis.

Can you give us some ideas. Is that 5 percent increase, possibly, or 8 percent or 30 percent, some sense?

MR. SOLER: In the coefficient or the value?

MR. JENG: For the increasing forces.

MR. SOLER: At the top of my head, I can you a number.

MR. JENG: An estimate.

MR. TRESLER: Is it clear what the question is?

MR. SOLER: Let me ask the question that I think you are asking, and you tell me if I am asking the right question.

MR. JENG: Okay.

MR. SOLER: You would like to know that with a 9-inch spacing, I get a certain value for a coefficient, and if I use a three-inch spacing, I get a different value for the coefficient. And what is the magnitude of those two values?

MR. JENG: Let me clarify. The Staff has concern that if you were to use the realistic gaps at both ends, there may be occasions that the forces of impact among the racks may be increased. Your earlier statement seems to

confirm that such a concern is valid, to some extent, because you, yourself estimate there could be occasions when differences may be increased to some extent, because of the numbers out of the 16 x 16 matrix, would show some changes. In your judgment that would increase the forces to some extent. And you mentioned, in answering his question, that these forces, in any case, will not increase beyond the maximum calculations.

Can you give an estimate as to what percentage of increase from what you did?

MR. SOLER: The only true estimate I can give to that is to say that the percentage change in those coefficients was in the order of 18 to 20 percent in the worst case of the four numbers we are looking at.

Now I would like to clarify what you said that I have said, in the sense that I believe that those changes could lead to an increase in the impact load between the first rack and the second rack. It is my recollection of the results of those various computer studies that the difference between the impact load at that location and the impact load at the most severe location in that model was significantly more than simply taking 18 to 20 percent of that value would lead to.

Actually, I changed something even further. If you look at the numbers of the gaps here, the effective gap

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value is about a 20 percent change. The coefficient that you calculate from that, when you take into account the contribution of the 7-1/2 inch side gap, the change is more like 8 to 10 percent. In other words, it is not a direct 20 percent in a gap that leads to a 20 percent change in the coefficient, because the coefficient, the predominant term in that is the side gap term, which is fixed at 7-1/2. So there is really like an 8 to 10 percent change in the coefficients. And it is my judgment that one could say in the worst case that an 8 to 10 percent change in certain of the coefficients might lead to an 8 to 10 percent increase in that particular force.

But the dominant force in that model which occurred between the second and the third rack was higher, significantly higher than the value one would tend to increase by 10 percent.

MR. JENG: So the bottom line in any case that the value you came out with would encompass an 8 to 10 percent increase in one location?

MR. SOLER: The values we reported would envelope the values that we might get.

MR. JENG: Second question. Could you give me some experimental verification of the validity of the G effective that you call G left and G right? Any experimental data that has been applied to provide some

basis for such an approach?

MR. SOLER: That comes out of a very precise theoretical model. I am not sure, and you can correct me, whether there is any experimental verification of large objects moving in a pool, as far as the formulas that you arrive at for hydrodynamic coupling.

MR. JENG: Is this the best we have in the state of the art?

MR. SOLER: I believe it is the best that is available in the state of the art now.

MR. SINGH: I guess the point to be made is that 1 over G term is the classical solution for plate to plate and the variable in whole expression. So to make comparisons between various gaps, we took the 1 over G, which is the way it appears in the hydrodynamic math term, and we compared it for different gaps. That is what we are saying here. We have not changed the basis, the original formulation for the hydrodynamic mass.

If you look at the equation, it has one overgap in there, and we are simply taking that one over gap term and comparing for various gap values, to give you an idea how the terms in the mass matrix would change.

Am I right, Al?

MR. SOLER: Yes. To answer this particular question.

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MR. JENG: My point is, besides saying this is the best in the state of the art, is different from saying this is the best available, but in our best judgment, this does an adequate job in representing the situation we are facing, providing good engineering solutions. It could be best available in the state of the art, and it can still be lousy, miserable, with regard to reality.

And I want the second part of the statement, if you can make it. And if you make it, what is the basis for such a statement?

MR. SINGH: We can make the statement that the statements are based on classic fluid mechanics, and they are unimpeachable. They are not based on any flimsy basis. It is really the classical solutions for deriving hydrodynamic mass, which draws only upon continuity, based on that, so that we cannot be wrong.

It is taking that and applying to appropriate to a plate moving in a large fluid medium. Use that solution, and we have, as a matter of fact, going back, we have considered cases where the gap changes during the earthquake, the nonlinear coupling effects. And it shows that the reactions go way down. So these are not only realistic, they are conservative. It is a conservative way to treat the problem. It is based on classical hydrodynamics formulations. It is not any new-fangled

formula that has not been tried before, that has not been used before in any way.

MR. JENG: Thank you.

MR. ASCHAR: Another question.

The questions and responses are leading to the third question, so why don't we take the two right now. I think you explained a lot of things relating to the third question anyway.

MR. SOLER: Question 3: Provide a Russian offer values for hydrodynamic coupling used on page 3 of Reference 5. The hydrodynamic coupling coefficients requested 3-D studies -- DCL-87-082 -- were calculated using the design gap for the rack module M, as shown in figure 32 of DCL-87-070.

These actual gaps are as shown in Table 3.

(Slide.)

Using the methodology previously discussed in response to question 2 for the determination of effective gap values results in effective gaps in the East-West direction, g, sub-B of one inch, and g, sub-B of 884 inches in the north/south direction.

These effective gaps were used in calculating the hydrodynamic coupling coefficents. And the lateral direction, as indicated in page 4 of DCL-87-082, the equivalent lateral gap, H-zero of seven and a half inches was used to calculate the height or dynamic coupling coefficients. for translational motion.

(Pause._

MR. TRAMMELL: The staff would like to confer for a few minutes with itself. And this might be a good time for everyone to take a 10-minute break.

(Recess.)

MR. TRAMMELL: We're back. And, Hans, you had

something you wanted to say.

MR. ASCHAR: The staff finds responses to questions 2 and 3 adequate. However, we might revisit them in the general line of questioning at the end of the six questions. So we can go to 4.

MR. SOLER: Describe how hydrodynamic coupling effects induced by rocking and torsional motion of the raps, as mentioned on page 3 of reference 5 were incorporated in the realistic cases analyzed in references 4 and 5.

The effect of rocking and torsion had been included in the analysis by considering the actual channel configuration constrained by real or hypothetical boundaries (which you see will simulate motion of adjacent racks). And by tracking the fluid movement in the channels consistent with the postulated rack motion. Integration of the fluid kinetic energy over the height and width of the rack yields rocking and torsional coefficients.

The step by step procedure used to develop these coefficients is based on the models shown in figure 1.

(Slide.)

Step one, the fluid velocity across the flow channel is assumed to be linear and is zero at the boundary and is equal to that of the rack at the face of the rack.

Step two, flow velocities along the channel are then calculated for each flow channel using the principle of

continuity.

For example, for Region I, the equations for the two fluid velocities, V-1 and U-1, are shown also in figure 1.

Step three, calculate the kinetic energy of the fluid in the flow channels surrounding the rack. The resulting equations are shown in figure 2.

(Slide.)

Integrating the expression for kinetic energy both in the horizontal and vertical plane yields the final expression.

The q dots are generalized velocities for the degrees of freedom described in the reracking report, and the b's are fluid coupling coefficients whose values are based on the geometry of the racks on the gaps in the flow channels, and on the fluid densities.

It should be noted that this equation is very similar to the equation described in Section 5.1 of Reference 4, DCL-87-070. That is the response.

MR. ASCHAR: Alan, what we are not clear about is the analysis in references 4 and 5. These effects have been used in which analysis has not been used.

MR. SOLER: In reference 5, which is the 3-D realistic studies, these analyses have been used in all of the rems made. In other words, incorporating torsion,

rocking, and, of course, translation.

MR. DEGRASSI: Also in the conservative models?

MR. SOLER: Indeed, conservative models. No torsion fluid coupling was used. And in the conservative model, while some rocking effect showed up, it showed up simply because of the assumption as to the location of the hydrodynamic forces being placed at the middle of the rack, rather than an integration procedure.

So the effect was there. The coefficient was slightly different because now integration was used. I'll leave it at that.

MR. FISHMAN: Let's state this again in a little more orderly fashion. We have 3-D and 2-D, conservative and realistic analysis. I want to make sure I completely understand one by one which effects were used.

MR. SOLER: Okay. 3-D conservative.

MR. FISHMAN: Yes.

MR. SOLER: The original licensee document. Hydrodynamic forces were computed based on translation of the rack at its centroid. The forces that resulted from that calculation were placed at the level of the rack centroid -- 3-D realistic.

The kinetic energy was calculated on a per unit depth basis based on the fluid flow in the channel, and then integration was performed over the rack height and around

all of the channels.

And that automatically led to that result after the integration. And since the fluid velocity incorporated both translation and rotation about the vertical of the rack, you incorporated torsional, as stated here, effects in calculation.

And the rocking contribution comes naturally out of the integration over the height of the rack of all of the forces.

Now, 2-D. No torsion in any situation. The 2-D multi-rack analysis using realistic spring constants incorporated what we have called translation and rocking.

The 2-D conservative model incorporated translation and rocking but the rocking simply was a result of the placement of the hydrodynamic forces. That was the same placement as in the original 3-D model. Namely, at a certain height of the rack so that there was no integration process.

You calculated a force and multiplied it and that gave you the contribution.

MR. DEGRASSI: Wouldn't that be applied to the centroid?

MR. SOLER: In the 2-D conservative model that force was applied to the centroid, yes.

MR. FISHMAN: In an earlier response, you

indicated that making changes to the gap sizes changed some of the mass coefficients by maybe 20 percent.

Now, here you are adding some terms on the diagonal, I guess would be added to moments of inertia.

What order of magnitude change do those coefficients, if you know that?

MR. SOLER: I cannot give you off the top of my head numbers. In general, I can make the statement that adding these terms increased terms on the diagonal. Under certain circumstances, where you have different gaps, and you can get off diagonal terms, too, and they simply, wherever they appear, as you can see, you will get off diagonal terms.

When they appear, they were taken into account. And of course you cannot correlate any of that to previous solutions. But the general statement can be made that the incorporation of this more accurate calculation led to an increase in the terms on the diagonals.

I cannot really give you a number.

MR. FISHMAN: Perhaps more importantly, what is the effect on the impact loads? Can you give us an order of magnitude number on that, including these effects or not including these effects?

MR. SINGH: We really have not done a term by term study like that to give you numbers. Any numbers would

be purely conjecture and should not be given.

We really have not done a parametric study with each term.

MR. ASCHAR: What question -- this parameter, it appears to me, appears to reduce the impact forces in some areas. It is realistic assumption. But, would have thought at least that there had been some calculation made by PG&E and their consultants without these effects.

These effects were meant to reduce the impact forces. So that is the reason we want to find out what kind of increases or decreases of the impact forces are what percentage. If you have any idea, one of you, we would like to know about it.

MR. SINGH: I would like to comment on that. The sequence of events was not precisely the way you said. The sequence of events was that when we did the multi-rack analysis -- that is, when we used the cross-coupling solutions that you folks have listened to us in February, and seen in San Franciso -- after that, when we were asked to do a 3-D analysis, we simply continued with the cross-coupling solution.

So, naturally, when you have rotational motion, in all directions you're going to have additional coupling terms. And then, naturally, we arrive at the same integrations as we had before.

And then the object was to take the spring constants, get values, more realistic values in some cases, where uncertainties at the lower bound values, to again exaggerate the response. Keep the conservatism, some conservatism in the model, and just proceed and do now an analysis including our fluid effects, just as we had done in the multi-rack analysis, and do it with a single rack analysis.

So we have not done a term by term evaluation. If we took away the rotational coupling term, what would happen, we cannot give you an answer.

MR. DEGRASSI: I have another question.

Is this method considered an extension of Fritz' work? Or is it based on the same first theory?

MR. SOLER: It is based on the same first theory except, since the rack has a different motion, that affects the fluid velocity in the channel.

You must calculate based on continuity of the fluid, the fluid velocity and the channel, based on whatever you say the rack is doing.

In this case, we said not only is the rack translating, but it is also doing this and this then directly affects the fluid velocity.

As soon as I locate this other figure, you can just see the presence of the dot terms, the fluid velocity

in that particular channel.

But that is a direct result of continuity. If you say Fritz' method is based on continuity and calculation of the energy or the momentum equation, and this leads to the contributions then, if that is what you say is Fritz' method, then I say it is exactly the same thing.

MR. DEGRASSI: I guess I would see it as an extension of the method. You would not find this in his paper.

MR. SOLER: Because he did not treat torsion. He did not treat this problem.

MR. DEGRASSI: -- rocking or torsion. As I recall, the equation you presented before, with the equivalent gap, is based on vibration of vibrating near a wall. Then we are always talking about pure translation of motion.

When you were getting into torsion and rocking now, you're going back to basic principles and developing basically what I call an extension of his method.

MR. SOLER: I guess it is then a matter of terminology what one considers an extension.

MR. DEGRASSI: It is based on continuity principles and basic fluid equations.

MR. SINGH: It's the same thing Fritz did.

MR. DEGRASSI: One other thing. Before you were

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running down the different cases. I believe you did not mention what you did on the 2-D single rack, realistic.

MR. SOLER: 2-D single rack, realistic, which was--I'm trying to get the chronology in my own mind--which was done after, I believe, the 2-D multi-rack.

I would have to honestly look at the report to say whether the rocking term was accounted for by doing the integration or by placing the force at the centroidal level.

MR. COFFER: Maybe Chris can help.

MR. SINGH: I would put it on the record. Instead of calling it an extension of Fritz' --

MR. TRESLER: We're on to a new question. You were busy.

MR. SOLER: He wants to know for the 2-D single rack analysis whether, in effect, the rocking was accounted for by integration, or was it accounted for by calculating the force based on translation and then putting that force into the centroidal location?

MR. SINGH: 2-D single --

MR. SOLER: The 2-D model of the single rack -- with realistic spring constants.

MR. TRESLER: The study that we submitted as a part of our 2-D -- multi-rack.

MR. FISHMAN: A single rack portion of the analysis. You did single rack conservative. You did single rack realistic. You did multi-rack realistic.

MR. SINGH: Right.

MR. FISHMAN: The question was single rack -- expanding it to conservative and realistic.

MR. SOLER: My inclination, the way I reconstruct the order of calculation, not the way the table showed up, is that that report dealt primarily with a multi-rack model.

And in the multi-rack model, we used the integration procedure. Now, subsequent to that and in writing the report, we made comparisons with conservative springs and realistic springs with a single rack model.

Therefore, since those analyses were done after in time the multi-rack, I wouldsay that they were done using the same procedures with the multi-rack as far as getting the coupling coefficients.

MR. SINGH: The rotational terms were not in there. The rotational effect.

MR. SOLER: They were in there. It was just whether they were calculated by integration or simply taking a moment arm of the force. They were always in there.

MR. DEGRASSI: Let me just back up to the 3-D conservative design basis models. You say that the fluid coupling was based only on the translation of the rack.

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Does that imply that the additional translation at the centroid due to rotation was ignored?

MR. SOLER: No. It does not imply that. Maybe to clarify in your own mind the value of the B coefficient and a certain term depends on the integration procedure. The hydrodynamic forces that arise from doing the calculation are located at different heights on the wall.

In the original design basis report, rather than calculating the energy per unit depth and integrating the moment over the whole height, we can get a 10D total energy for the rack and assume the force that resulted from it was concentrated at the centroid.

Then, when you use LeGrangian's equations, the moment or the contribution to the rocking degree of freedom was felt directly by that force times H over 2.

In the 3-D analysis that we have most recently done, we simply calculated the kinetic energy integrated over the height of the rack to get the final expression; and then the LeGrangian procedure puts the right terms in the right places.

And the same coupling, the net result of the two slight differences in approach, the coupling terms say q1 dot, q5 dot are still there.

But there is simply an insignant change in the value of that particular B coefficient, simply because of

the different assumption that was used in the design basis as opposed to simply going back to basics here and doing the integration.

MR. JENG: This is a clarification to understand where we're going in regard to the question just raised by Hans Aschar and supplemented by Howard Fishman.

It was raised that because of your sophisticated integration procedure you added some more terms, which may cause some concerns of increasing forces.

When this question was asked of you, you said we did not look at that, we do not know.

And my concern is can you somehow give us an indication of even if there were to be some increase -- number one, whether there would be an increase or not. If not, say so.

And then -- there may be some increases in load. Then could you give us some feeling as to what extent this increase would be? Would it be bounded by the other conservative parameters?

We would like some indication of that nature.

MR. SOLER: First of all, let me clarify your question, or correct your question.

The integration procedure is not what I would classify as sophisticated. It is just standard. The addition of extra terms comes about due to the torsion, not

due to the integration procedure.

If I neglect torsion of the rack, the kinetic energy expression would look exactly the same in the realistic analysis as opposed to the design basis analysis, except one coefficient would be different.

In fact, that coefficient would be slightly smaller in the integration procedure, simply because you get a one-fourth instead of a one-third -- the other way around. You get a one-third instead of a one-fourth.

So the actual numbers in the realistic analysis that you calculate in hydrodynamics are slightly smaller. And I say inisignificantly smaller because the number that is slightly different is a very small contribution to the whole term.

MR. JENG: So your answer to our earlier question is there would be no increase.

MR. SOLER: The earlier question really addressed what if you did not include torsion? That is how I interpreted it. And the answer to that question still stands the way we give it.

We did not do, or we did not interpret the questions as originally raised to do a study of including or not including that a fact. It was a simple, natural progression that, once you go from a 2-D to a 3-D and you are using a realistic model, that, therefore, we should now

say that since the rack is permitted to rotate about the vertical, that we can add the fluid coupling effects that would directly come out of the derivation due to the additional terms on those fluid velocities.

MR. JENG: But my question is, given the torsional factor was not accounted before, do we have a basis to indicate -- because the changes could be significant, or in your judgment, the changes would be minimal. Therefore, in your opinion, there is no need for further -- to develop more information to answer our question, which you are not able to answer right now?

MR. SOLER: I simply made the judgment that, by the definition of what we were classifying as a realistic model, these effects should be included naturally.

MR. JENG: The effect has included -- have you included the effect? The answer is no. Right?

MR. SOLER: I have included the torsional effect in the 3-D realistic model. That is the only place where it has been included.

MR. JENG: Have you included the effect, your result? You don't know how to answer that question.

MR. SOLER: Because the question was -- in other words, it was never a question in my mind at the beginning of the realistic 3-D analysis. By definition of a realistic 3-D analysis, we interpret it as not only putting in more

realistic spring constants, but also correctly accounting for the hydrodynamic coupling consistent with the basic assumptions of looking at a single rack.

So, therefore, not to include torsion would defeat the purpose of a realistic model.

MR. JENG: Having included that one to meet the specification of being realistic, is that account in your mind encompassing all of the considerations? Or is there something that you need to do in addition to answer all of the concerns?

MR. MARTORE: The earlier question that was not answered specifically is what is this specific contribution of each of the many terms, diagonal and not diagonal? And there was no study evaluating those.

I think what Dr. Soler is saying is that when -- you recall originally, we did a very conservative modeling and analysis. In the process of answering the followup questions, we said let's develop the most realistic and yet still conservative modeling representation of behavior.

And in doing that, torsion was considered in this way. So it is not that it was not considered.

MR. SINGH: I am prepared to make a statement that if you were to not include just the coupling terms from the rotational effect, the differences in the results would be marginal. It is not going to make a major difference in

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the calculated loads and so forth.

MR. JENG: I am with you on that statement. Are you concurring to his statement?

MR. SOLER: Yes.

MR. JENG: Okay. That's all I wanted.

MR. FISHMAN: On the rocking effect, it seems to me there would not be too much difference between including the rocking terms, as you are, or just including the translation at the centroid.

Is that a correct statement?

MR. SOLER: That is why I used the word "insignifant".

MR. FISHMAN: Is that at the one-fourth versus one-third?

MR. SOLER: Yes, that's right. It strictly comes in in calculating the effect of moment of inertia of this fluid that you calculate.

MR. ASCHAR: I think, for the time being, this question has been responded to. And we might revisit them in the general line of questioning at the end, but we should go forward.

MR. SOLER: Okay. Shan, you're going to present the fifth question.

MR. BHATTACHARYA: Question No. 5. In response to item 15-G of our request for additional information,

reference 3, the licensee provided tables one and four, indicating the design loads and capacities of the fuel vaults.

We understand that, at that time, the rack-to-wall impact was not postulated. In view of the analysis and references 2, 4 and 5, indicating significant rack-to-wall impact forces under certain conditions, provide calculations showing how postulated impact loads would be resisted by the walls.

Our response is table one of item 15-G of reference 3, PG&E letter DCL-86-019 addresses the in plant effects of the sheer walls, since the rack-to-wall impact essentially affects the outer plane loads on the pool walls, table one is not affected by this impact load.

Table 4 of item 15-G of reference 4 provides the results for the outer plane loads on the pool walls for Hosgri event. This included a rack-to-wall impact force of 80 kips. The revised rack-to-wall impact force is evaluated considering the rack-to-wall impact force of 215 kips per rack.

This force is the maximum force stated in references 2, 4 and 5.

The additional demands on the wall are calculated by analyzing the wall to impact forces applied as a line load. Total demands in the wall are obtained by combining

the demands due to impact with those due to other out of plane loads considered in table 4.

In determining capacities, conservatively, no period is taken for increasing allowable stresses due to rapid strain rate effects associated with the impact.

The critical wall A-6 with the stress ratio of 1.4 as shown in table 4 is analyzed for this additional impact force.

The resulting stress ratio for the critical wall A-D changes from 1.4 to 1.3 to accommodate the impact, revised impact, forces.

Furthermore, local effects -- that is, sheer on the concrete wall and the bearing on liner due to impact forces on the wall -- are checked and are found to meet the allowable with sufficient margin.

Stress ratios are greater than 3. Thus, all walls can accommodate the revised rack-to-wall impact forces.

MR. ASCHAR: Can you explain how you arrive at the 215 kips impact?

MR. BHATTACHARYA: In the 2-D multi-rack analysis we have reported the case where -- I guess it is the Section BB analysis -- where the maximum load was 107 kips per spring. Since we have two springs, it is 214 kips. We rounded it to 215, and we applied that load as a line load over the width of the rack and obtained the bending moment and the shear for the wall and added this moment of shear to the other global load that the wall has been originally designed to and then checked the stresses and then compared that with the capacity and then obtained the ratio for the demand over -- capacity over demand, which is 1.3, reporting for the critical wall.

MR. ASCHAR: The stress issue came out to be 1.3?

MR. BHATTACHARYA: For the critical wall, which is AD.

MR. ASCHAR: This is considering the rebar as the limiting --

MR. BHATTACHARYA: This is the same criteria we used for the Hosgri evaluation.

MR. FISHMAN: Was this reported in any of the other documents that we have seen? The original wall criteria?

MR. BHATTACHARYA: Yes. This is on our reracking

report. Subsequent to issuing our reracking report, there were additional requests for information which is, I guess, 15-G.

Question No. 15 of your reference, Reference 3, had a set of questions, and one of those questions dealt with the criterion in allowables for the design of the walls. These are stated in there, the load combination and allowables.

MR. ASCHAR: You mentioned at that time what you had assumed was 80 kips load on the wall, and you made calculations later on of 215 kips load which you anticipated.

Are they documented?

MR. BHATTACHARYA: Yes. Documented in the sense when we performed multi-rack analysis, 2-D multi-rack analysis, we had noticed that there would be cases where the wall load, the impact load on the wall would be exceeding the 80 kips.

So in order to evaluate the adequacy of the wall for such an increased load, we went back and revisited the original calculations.

MR. TRESLER: I would like, if I could, to make a clarification, at least from PG&E's perspective, and that is that the 2-D analysis and the loads that resulted from that analysis we do not consider to be a part of the design basis

for the racks or the building. These were parametric studies that were requested by the NRC, and they were done at your request.

And just to deal with this issue in total and all the loads that have ever come out of all of the analysis that has been requested, we enveloped all of those loads for purposes of demonstrating wall qualification, but I do not consider those to be design basis loads.

MR. COFFER: And this position has been made known to you in the past. This is not the first time.

MR. ASCHAR: In the previous submittals, in Reference 4 I have seen the numbers like 80 kips and 200 kips, two numbers I have seen. We have been confused with those numbers as to what they represent.

MR. BHATTACHARYA: 200 -- what we said is approximately 200 kips, we did the design to 215 kips. The parametric study, we did that for 215 kips on the wall.

MR. FISHMAN: The design of the wall consisted of considering several loading conditions?

MR. BHATTACHARYA: Yes.

MR. FISHMAN: One being hydrodynamic -- hydrostatic loads.

What percentage is this impact load affecting on the allowables, approximately?

MR. TRESLER: Can we draw a parallel between the

stress ratio that we had with 80 versus 215?

MR. BHATTACHARYA: 1.4 included the 80 kip load. I was just thinking about the bending moment that we had, about 10 percent.

MR. FISHMAN: So you had a 1.4 allowable or a 1.4 stress ratio at the critical section based on the 80 kip impact load plus other loads?

MR. BHATTACHARYA: Yes.

MR. FISHMAN: And then when you made a computation going up to 215 kips, it reduced it to 1.3?

MR. BHATTACHARYA: Yes.

MR. ASCHAR: The predominant effect is still the hydrostatic loadings?

MR. BHATTACHARYA: Hydrostatic and the sloshing effect of the water in the pool.

MR. ASCHAR: Your loads are not all -- some areas are five feet?

MR. BHATTACHARYA: One wall is five feet thick. This is the wall that divides the fuel transfer canal from the pool, and that wall has cross-walls. If you look at the span of that wall, that is not the critical wall, and also the stress ratio that we reported in response to 15-G, that is not the critical wall.

MR. ASCHAR: I think the response to this question is adequate at this time. If something comes out

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in the general questioning, we might revisit it.

MR. SOLER: Question 6:

Provide clarification of the weld stress calculations shown on page 271 of Reference 2 for the upper portion of the adjustable support leg as shown in Figure 3.6(b) of Reference 1.

Be specific concerning the use of stronger parent material property in arriving at the allowable weld stress.

(Slide.)

Figure 3 shows the classification and geometry for the weld connecting the upper portion of the adjustable support feet to the baseplate. The baseplate material has the lowest allowable stresses. This allowable weld stress, as calculated by subsection NF of the ASME Section 3 code, was used in the calculations. These allowables are as follows:

For a Level A service limit, fillet welds, the direct tension stress limit is 18 ksi for 304L baseplate material.

Groove welds, the direct tension stress limit is 21 ksi for the 304L material baseplate.

Level D service limits. The allowable stresses are increased by a factor of 2 for Level D service Hosgri conditions.

A typical support foot weld calculation for Case

Acorn 10 is summarized in Table 4.

(Slide.)

The maximum weld stress due to compression and bending is 27 ksi. The allowable stress is 18 ksi, Level A, resulting in a stress ratio of 1.5, which is less than 2.0.

The maximum weld stress due to shear is 13 ksi. The allowable stress for shear is 10.5 ksi for Level A loading, resulting in a stress ratio of 1.26, which is less than 2.0.

MR. ASCHAR: I did not understand. What is the allowable stress value and the value used for the Hosgri?

MR. SOLER: For the fillet, 18,000.

MR. ASCHAR: Then you multiply by 2 for the Hosgri?

MR. SOLER: Correct.

MR. ASCHAR: That is where our question arises. What is the maximum that could be allowed by the ASME code?

MR. SINGH: The allowable stresses, here reported to be 18,000, 21,000 psi, they are taken from the NF weld tables where the table has the type of material tension strength -- is the category, and the weld types are the other categories.

Groove welds, the allowable stresses are higher than those for fillet welds.

304L has a tensile strength in the neighborhood

of 70,000 psi, so it falls in the table in the line where it is between 57 to 80, I think.

MR. FISHMAN: 70.

MR. SINGH: So taken at the design temperature, the numbers are 18 and 21.

Now, in there you will see there is a paragraph in NF that says for Level D conditions, and it refers to the appendix where it tells you to multiply by a factor of 2 for Level D conditions.

So that is all we did. We went to the appropriate appendix.

MR. ASCHAR: In the subsection NF, the factors would be applied to the numbers themselves but not to the welds; the weld stresses are limited by the provisions in the code -- and I don't know which section it is -- referred to back in some area where it is limited to 1.42 times what is in the table, 18 or --

MR. FISHMAN: The 18 and 21.

MR. ASCHAR: Correct.

MR. SINGH: What issue of the code are you looking at?

MR. ASCHAR: '83.

MR. SINGH: That is the one we consulted, too.

MR. ASCHAR: That is why we consulted 1983, and we find for the welds I don't think that it has been used, 2

times 18 or 21 K. Maybe you can look into it and let us know later.

MR. SINGH: We will quote the appropriate paragraph from where we have derived this information.

MR. ASCHAR: You can look through it later on maybe and show us.

MR. FISHMAN: Could you show the sketch again?

(Slide.)

MR. SINGH: The fillets are on the outer diagram, and the groove is on the inner diameter. They are all 5/8ths.

MR. TRESLER: Hans, are you talking about -- when you say you don't believe -- at least your interpretation is that you do not believe that a factor of 2 should be used.

Is that your interpretation for the weld material allowables, which would exclude the baseplate material, or are you interpreting that to apply to the baseplate material, also?

MR. FISHMAN: Our understanding is you must use the weld allowables based on the weakest parent material that is attached, and our interpretation of that table in Section NF for the weld allowables states that the groove weld, you can use 21 ksi.

We did see in the NF the factor of 2 for Level D, but it did not seem to us that -- or to myself -- that it is

applicable to the weld itself. Section NF is everything for support.

MR. TRESLER: I understand your point now.

(Staff conferring.)

MR. ASCHAR: The response to Question 6 seems to be adequate, providing that --

MR. FISHMAN: There were some other slight questions --

MR. ASCHAR: I'm sorry.

MR. FISHMAN: As part of this question, what we wanted to derive from this is, when you completed the weld stress, independent of what the allowable is, what precise numbers did you plug into, which formula for weld stress.

MR. SOLER: What precise numbers --

MR. FISHMAN: What areas, what widths, what leg thicknesses.

MR. SOLER: The throat area of the welds. If you look at that picture and look at it from the bottom, what you see are two circular weld areas. Those weld areas were used to calculate the area and inertia of the weld metal. The resulting numbers were then corrected by a factor of .707 to reflect a throat calculation in those areas. And moments of inertia of the weld metal circles were used, formulas to calculate the stress on the weld, N over A plus MC over I.

MR. FISHMAN: Can you be a little bit more specific? You have an inside weld at approximately 6-1/4 inch OD and that was 5/8 inch on a leg?

MR. SOLER: Right.

MR. FISHMAN: They have an outside one at approximately 9 inches, and what was your leg?

MR. SOLER: The leg there was used as 2 x 5/8, corrected by .707.

MR. FISHMAN: Both of them were multiplied by .707.

MR. SOLER: Yes. Because when you do the area calculation, you are just looking at a total. The final area was reduced by 71 percent -- by 29 percent.

MR. FISHMAN: And what forces and moments did you use?

MR. SOLER: The forces and moments came out of the reracking report which quotes stress ratios for the upper part of the support leg, which are most critical. So what we had to do was to take those stress ratios and convert them to forces and moments and then convert them back to stresses in the weld.

MR. FISHMAN: And so you were talking about the normal compressive force, plus the two components and the bending moment.

MR. SOLER: Right.

MR. FISHMAN: What about the shear?

MR. SOLER: The shear calculation -- the direct shear is done separately. That is what the 13,000 number was based upon.

MR. FISHMAN: I am confused. I don't remember what the 13,000 --

MR. SOLER: In other words, at the point of maximum bending and compression, you will have the zero shear stress. At the point of maximum shear -- of course, the code does not require you to combine bending and shear, but, as a matter of fact, you will have zero bending at the point on the weld of maximum shear. So the two calculations were done separately by the same technique.

Take the stresses, both bending, normal and sheer in the support leg, calculate from those values, meaning the stress factors and the allowables in the areas, calculate from those values, the shear force, the direct compressive force and the net bending moment, and then take those forces and calculate using the weld area and weld inertia, the stresses in the weld at two locations. One, the location of maximum direct stress, which would be due to compression plus bending, and the other what you would loosely call the metal surface. And that is simply the maximum shear.

There are two different allowable stresses, factors that you use that differ by a square root of 3.

MR. FISHMAN: Thank you.

MR. ASCHAR: In that case, we will leave the answer to this question till after we refer to the ASME code and the stress allowables.

MR. TRAMMELL: This would be an excellent time to break. It would give the Staff a chance to consider everything you have said, in total. Hans will provide for you a copy of the ASME code, which you can look at over lunch or over the break. In order to give the Staff some time to confer and eat lunch, we would like to resume at 1:00 o'clock.

MR. COFFER: Is there a possibility that we can convene earlier?

MR. TRAMMELL: We can start at 12:30.

(Whereupon, at 11:31 a.m., the hearing was recessed, to reconvene at 12:30 p.m., this same day.)

AFTERNOON SESSION

(12:38 p.m.)

MR. TRAMMELL: We are back on the record.

We have some general questions from the presentation this morning.

MR. COFFER: What we would like to do is bring up the last question that came out of Question 6, on interpretaton of the Code, and try to take care of that one.

MR. ASCHAR: Let's do that one first, I agree, and then we will get into some general line of questioning to get our lines clear as to what and how the rationale --

MR. SINGH: Holtec International. I am responding to the question on the increase in the weld allowable or the level decondition. I have in front of me ASME Section 3, subsection NF of the code book, and I am going to refer the NRC to the paragraphs that deal with allowable stresses in welds NF 24.5, which has the caption "Design Requirement for Welds" in here, under groove weld, and that is where the requirements are most clearly spelled out.

It refers to the table 3324.5, A-1, and then the stresses given in that table are for level A conditions for multipliers to level A conditions, the code refers to paragraph NF-3321.1, applied to allowable stresses in table NF 3324.5, A-1. It gets cumbersome.

Now if you look at NF 3321, it says, quote, and I will read, quote, "For levels B, C and D service limits, the allowable stresses may be increased by one-third over the factors shown in Table NF 33-3533, B-1." 3321.1.

MR. FISHMAN: Is that pertinent to weld?

MR. SINGH: I refer to the section for the multipliers. In other words, if you look at the section under welds, it says the multiplying factors shall be those in NF 3321.1, applied to the liable stress given in the table above. And that multiplier, if you go to 3321.1, it tells you to take the multiplier from a table, from yet another table and increase that by a third. From that table, under level D condition, gives a multiplier of 2 for linear type supports.

So if I go strictly by a legalistic interpretation of the code and multipy a third -- 1-1/3 x 2, which is 2.6, we have always used 2, because -- because this particular section of the code has undergone many revisions. As a matter of fact, as you pick up different addenda from the same code, you will see differences in the verbiage. Sometimes it goes up, sometimes some statements are left out. Statements have been added, and we find that 2 -- a multiplier of 2, just as on the base material, once the code collects itself, and it has been our standard practice, and as we know, it has been the practice in the industry.

MR. FISHMAN: Note 5 on that table concerning shear stresses --

MR. SINGH: Note 5 on which table

MR. FISHMAN: 3523.2-1.

MR. TRAMMELL: Would you sit up here. The references are really rough.

MR. FISHMAN: The multiplier table that had the 2.

MR. SINGH: Here we go. The note 5 says that for service levels --

MR. FISHMAN: You are looking at this one.

MR. SINGH: Component supports.

MR. TRAMMELL: Off the record for just a moment.

(Discussion off the record.)

MR. ASCHAR: Howard, would you frame your question, and he answers the same thing he answered.

MR. FISHMAN: My understanding of your discussion is that using Table NF 3523.(b)-1 -- (b) is lower case (b) in parentheses -- that you are permitted to use 2 for your normal stress computation and a maximum of four 235 for your shear stress computation. Now S sub U should be, in your opinion, the weld material.

MR. SINGH: That is correct. For the reason that in tension, when there is a weldment between, say, a ball and a plate, the weak direction of the plate is in the true

thickness direction. The plates are rolled stock material and typically the strength in point is greater, the ultimate strength in point is greater than the transfer strength, and that is why the code, in looking at tension limits ties the allowable weld stress to the weaker of the two materials.. That is the reason behind it.

In shear, of course, the load is parallel to the surface of the plate, and therefore, the strong direction of the plate is affected. And therefore, in this case, for sheer stresses, the ultimate strength should be that of the weld itself and the code specifically does not say anything. It just says S sub U.

One has to draw one's own conclusions.

MR. ASCHAR: How is S sub U defined in the code?

MR. SINGH: The ultimate strength of the material, whether it is the weld wire plate, bar or whatever. Incidentally, even if the definition that S sub U would be that of the weaker material to be observed, we still need the stress limits here. We still need it.

MR. FISHMAN: So far what you have discussed refers to the paragraph 3323.5, complete or partial penetration groove welds.

What about the fillet welds?

MR. SINGH: This particular edition of the code has left out any multipliers. This does not talk about the

multipliers. So the general inference to be drawn in good design practice would be that the multipliers would be the same. The fact that the fillet welds are weaker than groove welds, is accounted for in this table here, the table being 3324.5. The fillet welds have 18 ksi, groove welds have 21. There is no reason why the multiplier should be any different.

MR. FISHMAN: It seems that each of the other joints best spell out the multipliers, and this did not.

MR. SINGH: I am quite sure that other addenda to this have verbiage on the fillet welds. It just -- the NF of the code, just about every single addendum has some minor corrections, things that were left out. Things that are being added, stress limits that have been increased, and so forth. As a matter of fact, in the 1977 issue of the code, the allowable stress in direct tension was only about 8000 psi. It was an error, but it was in the code, in the 1977 issue. Later it was corrected. There is a picture right here on the page.

MR. ASCHAR: In the seismic report what we saw was something like 80 percent of the ultimate strength of the stronger material being used is a base for justifying that you are within the allowable.

MR. SINGH: We apologize for the calculation. 80 percent is our internal design calculations that we do for

sizing members, and that some how got in the report that we have thrown you off. The governing code is this, and the allowable stresses are from here. Our requirements are more restrictive, but they should not have been in the seismic report per se.

MR. FISHMAN: Based on this, what is your margin of safety for the worst weld in the design report?

MR. SINGH: If I take a literal interpretation of the code, as you see, the verbiage permits me to multiply by a factor of 2 and then increase it by 1-1/3, which, again, I believe has been in later addendums changed. It is now only a factor of two. If I go strictly by this, the factor would be even greater, but the presentation that Dr. Soler made, that shows the factor to be 1.5 versus the allowable of 2. So the ratio of the two is the margin. 2 divided by 1.5 is 1.33; isn't it? Yes. 1.33 is the ratio allowable to the actual.

MR. FISHMAN: Are we talking about one of the Rs or a separate calculation that happened to be derived from the R number?

MR. SINGH: Just to get the moment in the shear, the juncture --

MR. SOLER: We have the calculations.

MR. SINGH: The R numbers were used to calculate the gross moment and shear at the weld location. After

that, the strength of materials calculations were done to compute the direct tension on the throat, the maximum value of the tensile stress on the throat, I should say, and the shear stress.,

MR. FISHMAN: When you computed the R values, you computed the R values for R 6, I guess, was compression plus the two bending moment effects. The section module, modulus of the support -- well, you had an axial term, a moment about X term and a moment about Y term.

Did you compute each of those individually and add them up, absolutely?

MR. SOLER: No. We computed each of them independently at the current time instant and added them up according to their -- the moment term. We take the resulting moments, the sum of the squares of the two independent moments, and you add that with a minus sign to the compressive load.

MR. FISHMAN: You did add --

MR. SOLER: Sure. There is at least one point where they add on the surface.

MR. FISHMAN: But in computing your R-6 for the leg, forgetting the weld computation for a second, you did take the vector sum of the moment multiply, divide it by a single section modulus?

MR. SOLER: Yes. Circular section.

MR. SINGH: That is right.

MR. ASCHAR: You have two welds on the perimeters. One is inside and one is outside.

MR. SINGH: Groove and a fillet.

MR. ASCHAR: Both are grove.

MR. SINGH: One is fillet, the other is groove.

MR. ASCHAR: Would you tell me what stress you used in case of the groove, after you told us about the effect of multiplying a factor of two. You used 42 ksi?

MR. SOLER: No. The factors that are quoted on that slide were based on using 18.

MR. ASCHAR: 36 ksi.

MR. SOLER: Right.

MR. SINGH: The added strength of the groove weld.

MR. SOLER: That was for every check we made.

MR. ASCHAR: For the fillet weld? On the outside is what you consider as the fillet weld? Outside. Okay.

MR. SOLER: 18 ksi was used for every weld check, whether it was groove or fillet.

MR. ASCHAR: For level A. I am talking about the HOSGRI.

MR. SINGH: We took 36.

MR. SOLER: We took 36 as the basis.

MR. SINGH: We did not take credit for the extra

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3000 psi that the groove weld permits you. We did not take credit of 21,000. We used 18 throughout.

MR. ASCHAR: 36 K is still conservative. Is that what you were telling us?

MR. SOLER: Yes.

MR. SINGH: 36 is the minimum allowable stress from that table in tension, be it fillet or groove. Groove allow higher stress. We took the lower of the two values and used it throughout in computing these ratios.

MR. ASCHAR: On these two answers on the whole aspect of stress, what we will do is talk to our weld experts in house. Right now I do not have an approach to tell, and we will come back to you as to the responses, whether the responses are satisfactory or not.

MR. COFFER: Hans, you said that your weld people are not available now?

MR. ASCHAR: I did not have a chance to call them up here, because I thought whatever you said, there would be something that I understood completely. And I don't.

MR. COFFER: Could we give them a call and get them down here to make their interpretation so that at least if it isn't consistent with ours, we will know now?

MR. ASCHAR: What happens is that after the reorganization, I even don't know where they are located.

(Staff conferred.)

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MR. COFFER: We will table this.

MR. ASCHAR: Until we can get to it.

MR. COFFER: We will move on to the more general questions you had in mind earlier?

MR. ASCHAR: Okay.

On general questions, Guiliano will address general questioning on the last two analyses.

MR. DE GRASSI: I would like to get back to the question raised earlier, that I raised earlier, considering the fact that in the last analysis, the realistic model showed higher rack-to-rack and wall-to-rack impacts. Number one, how can you assure us that if we had used the factor of 1.0 rather than 1.5 on the calculated values, you would have values still -- you would have still higher values, and number two, can you give us your explanation of why you believe that realistic parameter model gave the higher impact loads rack to rack.

MR. SINGH: First, I do not mean to be a wise guy, but I want to correct your statement. The realistic variable based answers, the loads are less than the design basis loads. In some cases, for instance, in the case of local efficient friction, the loads went above the corresponding load from what we called a conservative model, but yet these loads are less than the design, what we clal the design basis loads.

Am I correct in making that statement?

MR. TRESLER: Yes.

MR. SINGH: So the loads did not go up. Now when the realistic -- what we call realistic -- I don't like the world realistic myself. I like to use the word that these are still conservative analyses, and wherever there is room for doubt as to what the particular variable should be, we go the conservative way. For instance, we know from our prior analyses, that the fuel assembly to sell impact loads monotonically, as we increase the local spring constant. They always go. So even here we increase them by 50 percent from the calculated value to again maintain the conservatism.

In the vertical direction, vertical springs, the support springs, as we call them, they can be explained that if those springs are taken less than the actual spring for lattice type that this rack is, then you again amplify response. So we took spring constants in the vertical direction, as Alan was saying earlier, taking linear elastic foundation spring constant as an upper bound. We know it is not the linear elastic foundation. The boxes are connected, so it is more like a portion of a half space. It is not a half space, because they are loading into the corner. It is one-fourth of a half space.

Now Paul did some analysis, and he showed that

the spring constant is more in the order of one-fourth of what we originally used in our conservative model than the realistic model that we are calling the realistic spring constant.

Now if you take for the case, a .2 coefficient of friction, there's one additional piece of information that we have to remember. The rack typically tends to slide. It is more a fraction of the duration of the earthquake. It is on four legs, as opposed to a high coefficient of friction which tends to stick, and it may be a one leg or two legs for a later fraction of the term.

The other piece of information to bear in mind is that in this model, the model that we have here, we have absolutely -- we have given the rack absolutely no natural adjustments for motion in the vertical direction. There is no coupling, there is no form drag, even though NRC has allowed form drag in prior licensing. Imagine a 10 foot by 10 foot plate trying to lift up with absolutely no force opposing it.

So as a result, in the vertical direction, if we change the spring constant, such that, if you look at the response spectrum, the vertical response spectrum, if we climp up to the peak of the response spectrum that amplifies the response, now if you take the coefficient of friction were it equals .2 case, if you look at that, and if you make

the supposition that four legs, the rack is on four legs, and it is now a spring mass system in the vertical direction, you will see, on the response spectrum, it is sitting high upon the curve, when four legs have four springs in parallel.

And for that case, because of the vertical implication, the rack rocks more and the loads go up. If you were to use the -- a higher spring constant which took it off that peak, or if we were to add some realistic phenomenon, such as form drag in the vertical direction, that effect would be suppressed and the loads will come down. But since we did not want to add any additional wrinkle to the model, at this point, any additional sophistication to it, we keep it as it was, we used a value of vertical spring constant which obviously exaggerates the vertical response. And in the case of load coefficient of frictiion, gives a higher impact load, contrary to what one would expect with a -- if one wer not to give much thought to it.

So that is how we, from a dynamic standpoint, explain this, and it is very plausible, and we are all in unanimous agreement that that is the mechanism that causes it.

If you wanted to get a realistic response, meaning what would really happen, you would hae to add, in a

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vertical direction, a little more sophistication to the model or use a higher spring constant, more appropriate spring constant for the springs. And that would take it off the peak of the curve, and then you would not see the peak. You would not see the increase in the load. But being that, with these springs the reactions, the impact loads are still less than design value, there is no motivation to do any further study.

What we have done is essentially made the rack as rocky as possible in the vertical direction, and even then showed that the impact loads are less than the design basis loads.

MR. TRESLER: And the increase only again occurred within the .2 coefficient friction case. We did not have an increase of .8 coefficient.

MR. SINGH: Because there --

MR. TRESLER: You should check that. That is in our perspective.

MR. DEGRASSI: I'm looking at table 4-2 in your April 23, 1987 submittal. The increase I'm referring to is rack to rack between 8, 12, 3-D through 2.

MR. SINGH: Coefficient of fraction .2. This is the only case where it goes up. It does not go up in the other table that deals with .2.

MR. DEGRASSI: 7-6-C-85.

MR. SINGH: Yes. The loads are comparable for 4.8. They go down substantially. They go down substantially on the support. And then they go up slightly.

MR. DEGRASSI: They go from no impact to a 48 chip impact. I would say that goes up.

MR. TRESLER: I think what we intended, when we were saying forces, what we intended to be speaking to was the stress ratios, that the stress ratios were lower for the realistic model. In all cases in the .8 coefficient of friction case. The .2 coefficient of friction case, we did have some increase in the realistic model over the conservative model.

MR. DEGRASSI: Considering the uncertainties related to calculating this spring coefficients, particularly of the support feet. I would like to have a warm feeling that if the actual spring rates were lower, you would not have higher still impact loads between racks or rack to wall, saying if you look at the single model in the

vertical direction, you would see that the spring constants we have used, realistic spring constants, they put us smack on the peak of the curve. There is no way to go but down.

We have as best one can using analytical models tried to maximize the calculated response. We are not coming in here with numbers.

If you look at the people -- people projecting response characteristics of the structure by looking at the response spectrum in the vertical direction, it is most meaningful because that is where we have no coupling effects, nothing to complicate matters.

We have put it smack on the peak of the response curve, the realistic spring constant.

MR. DEGRASSI: Of the vertical response curve.

MR. SINGH: Yes. It is sitting right on the speak. So I would say with good confidence that reducing the spring constant any further or increasing it would reduce the response.

MR. DEGRASSI: Isn't it primarily the rocking response that would increase when you go to the more realistic or lower spring values?

MR. SINGH: The vertical translation of movement and rocking. And they of course interact with -- the rack tends to lift off and the rack can also rock more. Any increase or boost in the vertical direction, any

synchronization in the vertical direction, tends to aggravate things all the way around.

And being that there is nothing in the model to mitigate the effect of vertical seismic input, it shows up very rapidly.

That is why even though the model is realistic in other aspects, just by putting the vertical support springs on the top of the response spectrum, we are getting very conservative answers again.

These are realistic conservative solutions.

MR. ASCHAR: What we were thinking of before you provided this explanation at this time was the bias for the impact forces given in the licensing report, or in the seismic report even. In some cases, when you started doing so-called realistic spring, constants, it prompted Juliano's questions that if you start with softening the springs more than what you have done before in the realistic calculations, would you see higher -- you were looking at the response spectrum and you're saying you're putting the vertical value at the peak.

Now, you have combined horizontal and vertical response. The peak of the horizontal response is not in the same place as the peak of the vertical response.

MR. SINGH: Please understand it is in the vertical direction that this spring system, it has four

support springs. In the horizontal direction, it is not a spring massive system until it impacts something, until it impacts another rack. In the vertical direction, it is constantly behaving like a spring mass system.

So there looking at the response spectrum, you can draw meaningful conclusions.

MR. ASCHAR: But, displacements are going on under the horizontal response effects. Displacements.

MR. SINGH: By looking at the response spectrum, you cannot draw any conclusions of that sort; whereas, in the vertical direction, you can't directly relate to it until it lifts off the ground; it is indeed a massive spring system.

MR. DEGRASSI: But it is also a rocking system which results in translation of motion in the horizontal direction.

Can you relate that frequency, the rocking mode, to the horizontal response spectrum?

MR. SINGH: In my own experience, you cannot because of the coupling effects that are present, and so forth. You cannot draw any conclusions from horizontal the spectrum, the vertical spectrum, unless you add additional sophistication to the model, drawing upon the -- bringing the model closer to reality --

MR. TRESLER: But you are the expert in this

area. But isn't the friction coefficient limits, doesn't that limit the load because of that limiting the input from the horizontal?

MR. SINGH: What he is saying is that the horizontal -- we are explaining why the realistic model gave higher values on the basis of the response spectrum in the vertical direction.

You can draw that conclusion in the vertical direction for a spring mass system because it does not have any coupling, it has no form drag, it has nothing. It is really a spring mass system in the vertical direction.

In the horizontal direction, it has fuse coupling terms and, therefore, it does not have a spring mass collective, as it is in the vertical direction.

If it weren't connected to the wall with a spring, then it would be more like a system.

MR. TRESLER: The only connection is at the foot and even that is limited.

MR. SINGH: Friction is the highly nonlinear spring. The horizontal response spectrum, we are unable to draw any conclusions. But, the vertical spectrum, the conclusion is direct in. It is the only possible explanation.

MR. BHATTACHARYA: Since we are picking out the high value acceleration due to the peak of the spectrum,

you're picking up here, sure, because the downward load has exceeded significantly by being in the peak of the spectra. So the higher the foot force, the higher the share getting transferred to the body of the rack times the foot force, times the coefficient of friction.

So you do see, even though we're at the lower friction value, we do see combined increased vertical force in the foot as well as sheer and moment because of the increased vertical force.

MR. SINGH: Explaining the further effect of what happens when you put it on the peak of the curve, you have a higher vertical load in the support foot, being that the lateral share horizontal -- the coefficient of friction times the vertical force, typically the limiting condition before sliding occurs, is the horizontal share.

That becomes the moment in the body of the rack, so the stress factors go up. In other words, all of these terms go up, can be explained on that basis.

I firmly believe that that is the explanation. If we were to take a spring constant, a higher spring constant, the actions go down.

MR. ASCHAR: The values that have been shown in the report, it shows when the impact values, the rack to rack impact values, are higher, the stress -- compared to stress -- is going down instead of up.

And that we could not understand why that is happening.

MR. SINGH: The stress in the rack depends on should I say three or four major forces? It depends on the horizontal sheer force. Equally, you have horizontal sheer force on whatever support legs are in content. You have inertia force. You have the racking force, the fuel assembly on the rack. And then you have the impact forces.

All of these contribute to no small moment on a section. And you could well have a high impact force which is opposing the moment from other forces. And the net results in moment is less.

When the impact force is high, that can well happen. So you do not -- that is not the sole force contributing to the section moment. That is one of the forces.

MR. DEGRASSI: What is the predominant effect from what you have seen? Which force dominates? Is it the fuel to rack impact? Is it the rack to rack impact? Is it something else?

MR. SINGH: Rack to rack impact is relatively small compared to all of the forces reacting on the system. The inertia force would be the largest.

MR. SOLER: I don't know from a time history analysis that you can draw any kind of a conclusion, that

you could back up. What is happening in this case is that when you set on the peak of the vertical response spectrum and you don't -- I'm leading up to this because you mentioned the word "rocking" -- when you have this thing going up and down and you have no restraint on it, because you have chosen not to realistically model the vertical motion in terms of form, drag, things like that, for the .2 coefficient of friction, when this thing is driven down to get a higher support load, that means that this share load when it is in contact is going to be higher, which can induce the motion of rocking that .2 that you normally would associate at .8.

But it all is traced back to being on the vertical motion. In other words, it is not rocking at lower coefficients of friction because there is something about the model per se, the values of the spring constants used in the horizontal direction, the time at which a particular force hits and the direction that it is in.

The tendency to rock at lower coefficients of friction is solely traceable back to the fact that you have higher normal forces and can then get higher sheer forces.

So that if you are off that peak and these forces go down, then you do not ever tend to rock at lower coefficients of friction.

But I don't think that one can answer your

question as to which force is the most important when you can generate all of the time history curves, as you saw at the last meeting, of various forces impacting, and you cannot draw conclusions to a specific question that says:

This is the force that is more important than this force.

MR. TRESLER: One of the things that we cannot lose perspective on is even though we did have a slight increase in stress ratios for the more realistic springs and hypodynamic coupling at .2, we still have tremendous margin to allowables.

There was a small change at a very low level and still all of those values were significantly below the ratios that we had for the realistic loads and stress ratios out at the .8 coefficient of friction case.

To put it in perspective.

MR. DEGRASSI: That is a significant point.

MR. CONGEL: The delta in terms of the spring constants going from what we had is conservative to realistic. It is certainly a bigger change than what you are asking about from one and a half to one; in that big change, we did not have very much increase.

MR. TRESLER: But I think, as to what happened, I think Chris has done a good job explaining it. To be honest with you, we were somewhat bewildered, too. And there was a

fair amount of work done as a result of this misunderstanding which caused a fall in the peak. And that is why we have this slight increase.

But, even though that happened, it is a slight increase.

MR. DEGRASSI: My impression, and you can correct me, I suppose, if you do not agree with this, is that you basically have a rigid system with a mass on top on two -- this is equivalent to having a rotational spring at the bottom of a stiff system, with a mass up on top.

MR. SINGH: That is one. But what I am referring to is the -- you have an earthquake imposed on this. This is a vertical ton history which has a response spectrum like this.

MR. TRAMMELL: If you're going to use sketches, we have to get copies of them for the record --

MR. TRESLER: Can we go off the record for a second?

MR. TRAMMELL: If it is an explanation we want, we are losing it. So we will see if we cannot orchestrate this thing.

MR. SINGH: We have it on the record, the explanation. We are just now informally making some cartoons.

Assuming the spring is one massive spring and we

have a mass here and we have the vertical, we actually have --

MR. TRAMMELL: Just a minute. I want to get clear on this. Can you wait just a minute, Chris?

Off the record.

(Discussion off the record.)

MR. TRAMMELL: Back on the record.

MR. DEGRASSI: Chris, can we look at the horizontal modes of the model which result from the rocking effect, and relate that to the horizontal response spectra?

If we can look at it that way, can we see where we fall in the spectra; specifically, are we at or near the peak?

MR. SINGH: We could do that if we did not have-- if we had the rock in air and there were no fluid effects involved. Then it is a clean proposition. You would look at the rack in motion and get some information, glean some information from the response spectra.

However, because the fluid effect, the fuel mass rattling effect, such high nonlinear terms and of course fluid coupling terms off diagonal components. So the direct deduction from response spectrum is obscure.

The reductions cannot be made. Whereas, in the vertical direction, the plain vertical translation of motion, there, we don't have any coupling. We have a large

mass which is connected to the base, which is attached to the base of the rack. That can vibrate lineariallý, linear spring mass damper.

And as a consequence, in that direction, the seismic input -- that direction being the vertical direction -- the seismic input, you can draw inferences as to how the support spring stiffness would affect the vertical motion of the rack.

And we find that the acceleration, the peak acceleration jumps up to 1.6 g. The peak acceleration jumps up to 1.6 g, whereas the typical value would be in the neighborhood of -- for a stiff spring model -- in the neighborhood of .75 g.

And if one were to assume that the rack is primarily on one leg, say, for instance, which means not only one spring, is active in the vertical translation of motion, then it drops down to 1 g.

So, as I said earlier, that with four springs in parallel with the values we have used, we have put this mass, this spring mass system in the vertical direction smack at the peak of the response spectrum curve. And that is why you see the increases.

This is a -- way for us to make a conservative--

MR. ASCHAR: As we see, the impact loads on the liner plate with the reaction of one foot on the liner

played is increasing in some of the cases in this recent analysis.

Does it affect any of the liner -- how are we as far as the strains in the liner?

MR. BHATTACHARYA: It has not exceeded the design value.

MR. ASCHAR: I thought, in two cases, it did. Not by a very high amount.

MR. BHATTACHARYA: In the seismic report, we have 295, I believe, value impact levels.

MR. SINGH: Close to 300 kips is the design basis. The other loads are much smaller.

MR. ASCHAR: At those loads, the strain levels are within the ASME Section 3, et cetera, et cetera?

MR. SINGH: Yes, it was checked for that.

MR. DEGRASSI: Can you tell me what kind of safety margin you have on the -- liner due to impact of a single foot?

MR. BHATTACHARYA: We send in an additional submittal that we made for the reracking report. I happen to have a copy of that.

This is PG&E's letter DCL-86-19. And we have provided a response to the same question. I believe it is in response to 15E and 15F. Now, this response deals with if a lower load is less than the loads that we finally end

up with as reported in the seismic report.

But, again, our revised calculation shows that there is ample factor of safety in the strain calculations. Our original report showed that the factor of safety for liner plate was on the order of 4.5. And the liner anchor displacement design, the factor of safety was on the order of -- for a liner plate is a factor of safety of 3.2 per liner anchor displacement. On the floor was 4.5.

MR. FISHMAN: What do you mean by liner anchor displacement? Bed plate?

MR. BHATTACHARYA: Division Two has a specific criteria for the liner as well as the anchor.

MR. TRESLER: And that was with a load of --

MR. BHATTACHARYA: Those factors were developed based on a load of -- these factors were based on the maximum value of accumulated reaction on four feet. That means an interior rack 10 by 11. You know, we have only one 10 by 11 rack, but the assumption was that the interior 10 by 11 rack on all four feet are sitting on one bridge plate. And a total load that we got, the reaction was 677 chip.

And that value corresponds to a factor of safety that I just mentioned here.

MR. DEGRASSI: Have you looked at potential punching of single feet?

MR. BHATTACHARYA: Yes.

MR. TRESLER: Are there leak chase channels underneath the floor?

MR. BHATTACHARYA: In calculating the punching area, we deducted the area of the leak chase.

MR. TRESLER: And are the feet of these racks arranged such that you stay away from the leak chase channels?

MR. BHATTACHARYA: There are some places where the leak chase is underneath the bearing plate. The feet of the rack do not sit on the liner itself. There is a bridge plate and the feet sit on the bridge plate.

MR. ASCHAR: Is this shown on the sketch? I have not seen that bridge plate. Is it shown on the sketches in the report, in the licensing report?

MR. BHATTACHARYA: It is in this submittal.

MR. ASCHAR: Which submittal? The January '86 submittal.

MR. BHATTACHARYA: Yes.

MR. TRESLER: So you bridge across these leak plate channels. The feet sit on a bridge, so it makes no difference where these go. You are offering protection from punch -- by that method?

MR. BHATTACHARYA: Yes. Charlie, we have some on the welds. There are some crowns on the liner. We do not want to grind those crowns and increase the potential for

any leakage. So we cut through underside of the bridge plate over it.

MR. FISHMAN: One question goes back to question six, where we wanted to fully understand the weld calculation at the top of the support, the adjustable support.

We've referred to the seismic report, page 2-71. In that report, there was a calculation of the weld stress based on taking the largest R6, which whatever was computed at that time as being the largest R6. And you essentially ratioed it by an area at the weld to the area of the parent cylindrical material.

And then did some other computational -- this is not quite in accordance with what Chris earlier discussed of the proper way to do this weld, treating the groove weld as one portion of it and the fillet weld as another portion of it, separating the areas and the moments of inertia.

What is the calculation that you did for this?

MR. SOLER: The values we have just reported on in response to your question, which lead to the factors-- what are the numbers there? 1.5 and 1. --

MR. FISHMAN: Are these numbers available to us?

MR. SOLER: Are these available to them like the other ones?

MR. BHATTACHARYA: Sure.

MR. SINGH: We can make that available to you.

MR. FISHMAN: In this calculation -- you do not need to read it out to me -- it starts with an R6 or some other component --

MR. SOLER: It first starts with the calculation of the areas of inertia, and eventually it reads the R1 to R3, the R4.

MR. FISHMAN: And extracts the moments.

MR. SOLER: Extracts the moments and forces from that, and then makes the calculation.

MR. FISHMAN: And you end up with a stress?

MR. SOLER: In the welds.

MR. FISHMAN: Of how much?

MR. SOLER: A stress in the weld -- this is compression plus bending, both at that location of 27,066.

MR. FISHMAN: Fine. That answers that question. As long as I will have that piece of information to look at.

Another question concerning allowable loads, 1D-- not associated with the welds. One of the important allowable loads we have been using as a guideline is the impact load on the girdle bar. And we have been using 175 chips. Looking at the seismic report on page 267, it seems to me that was computed by taking some yield stress and multiplying it by one of the areas, cross-sectional areas of the girdle bars -- not necessarily the very smallest of the

two typical areas, but one of the areas.

You used a yield stress at that time of 55,000 psi. So that gave us a total of 175,312 as the limit.

It seems to me that there is no real margin on this. Assuming that 55,000 is the proper yield stress, which I think you later degraded that to 40 -- 44.9 or 49.4, don't remember.

MR. SOLER: It is the upper support leg material.

MR. FISHMAN: Part of the question is what is the appropriate yield stress for the girdle bar?

MR. SINGH: We will look it up and we will give it to you. But that load of 175,000 pounds has a lot of fat in it because we should be looking at the frame, not just the end of the bar. We should be looking at the frame.

And any kind of an impact that takes place, that initiates -- that takes place on a corner, proficates to the rest of it. We have calculated a load of 2,100 pounds per cell, I believe, in the calculations following the end calculations right there, following on those pages (indicating).

And if you take the entire frame, the load carrying ability is substantially greater than that the limit load on the end of the bar itself.

The second thing is the load carrying -- the limit load is not a requirement. The requirement is large

deformations near the active fuel. That is a requirement from the NRC position papers.

The bar itself, the actual load limit on the bar, should be the ultimate strength of the bar. In other words, when that fails, it no longer provides any support and the stretcher collapses and you have large deformations near the active view or region, that is the true limit specified by the NRC.

So this number right here, the number here should not be the ultimate. It should be the ultimate strength -- yield strength. It should be the ultimate strength for the load carrying capability.

So the 175,000 pounds that we have used before is by a long shot a very conservative number. We just did not bother to go back and -- explain it.

MR. ASCHAR: I do not know that the conservatism is as much as you are trying to emphasize at this point. The first reason, the vertical bars that you are considering, compression.

MR. SINGH: That is correct.

MR. ASCHAR: And you are taking the full use strength of the material to compute the girdle bar.

MR. SINGH: The key point is that your organization has put in the OT position paper that the subcriticality of the rack should not be violated at the

top; where the girdle bar is, we are about one foot away from the active fuel region.

The frame, the girdle bar frame, its load-carrying capability, there is nothing magical about the elastic; the load carrying capability is not a long, unsupported beam interconnected to the cells, it is welded continuously.

MR. ASCHAR: I understand what you are explaining. Let me rephrase my question. In compression there is no code or anybody allows you to go to the ultimate strength of the material with the dynamic impact loading, I think it is. Not a right way to emphasize the point that because you did not go to ultimate strength, you have a lot of margin of safety because you are not supposed to go to compression at that much of a level.

And the second thing, your point that your whole frame resist impact loads, as you had indicated before, the wide amount of impact load comes from three feet on one foot and hitting somewhere in the edge.

In those cases, it could be only one part of a girdle bar might be contributing to the distance rather than both the parts or some distribution inbetween. It is not as much as you emphasize.

MR. SINGH: I would take what you said, and that still provides another blanket of safety on the numbers provided here, and I will leave it at that.

MR. TRESLER: Is the NRC comfortable that they understand that there is a significant margin beyond the design value that we used of 175 kips? Is that clearly understood and accepted?

If it isn't, I think we need to talk about it.

MR. FISHMAN: The number as stated, with the appropriate yield stress, which Chris thinks may not be the appropriate thing -- feels that it may not be the appropriate thing to use -- the yield stress is lower than the 175 kips.

MR. TRESLER: You did not answer my question.

MR. FISHMAN: The computation was made in the seismic report, and this value was inserted in the licensing report and every other document. We have an allowable impact force of 175 kips. The only computation of 175 kips that I see is on this page 2-67 of the seismic report, and it should be degraded somewhat based on the argument in the seismic report. Based on his further arguments, then it could perhaps be increased somewhat or significantly.

MR. TRESLER: Okay. I understand and agree with what you have said.

Do we need to do anything to allow you to be

comfortable with this allowable value?

MR. ASCHAR: Our general line of questioning is over. We would caucus ourselves and comment on what we want you to do.

MR. TRESLER: Fine.

(Staff conferring.)

MR. FISHMAN: I have nothing else.

MR. TRAMMELL: The Staff has no further questions.

At this point we would like to have a Staff caucus of approximately 15 minutes, at which point we would resume and give you -- conclude this meeting.

MR. TRESLER: Charlie, we would like to interrupt your caucus, if we could, when we get a copy of the weld calculation you have requested so that you can review that during your caucus, and if that raises any questions, we can address it again in the meeting.

MR. TRAMMELL: Very good.

(Recess.)

MR. TRAMMELL: Back on the record.

I think we are ready to conclude this session.

First, Howard Fishman has reviewed a calculation which we -- which I would like to have him describe and report on.

MR. FISHMAN: You provided me with the response

to NRC Question 6, and I checked the numbers, and I think you did for the most part an appropriate job in computing the stresses, especially in the -- well, in the outer weld. You did not take advantage of the smaller radius in computing the stresses in the inner weld, which might help you somewhat.

But for the record, I would like to say that the stress computations are appropriate. The allowables are still somewhat questionable.

MR. TRESLER: Can we understand some sort of schedule for resolving the issue of appropriate allowables?

MR. TRAMMELL: We will get to that in a minute. We can get to it right now since you brought it up. NRC is not able to answer that question immediately. There are some people that we need to consult with who are on the code groups involved, and I imagine we can do that in a matter of a few days.

If there is something that comes of this and we need action on your part, we will contact you. As it stands now, it is our ball, we need to do some checking. If we need to contact you, we will.

I was about to say that we will attach that calculation that Dr. Fishman has described to the record.

The second area involves the calculation involving the girdle box, which Dr. DeGrassi will go over.

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MR. DE GRASSI: I have reviewed the calculation that was provided entitled "Calculation of Impact Spring Top Grid Work." This provides the basis for the impact springs at the girdle bar locations, and I conclude that the calculation provides a reasonable basis for the values used in the model.

MR. TRAMMELL: I also will attach that calculation to the transcript.

MR. ASCHAR: There is one question that I would like you to answer in whatever manner you can at this time. You have provided us with baseplate stiffness, not calculations but what you use in your model analysis.

What we would like to make sure is when the baseplates -- when the racks move around and if the baseplate hits the wall it is not going to penetrate or damage the liner of the wall. That could happen when the protrusion of the baseplate is more than the girdle bar thickness in some cases. So we would like to have some confirmation as to the -- the first thing, the baseplates will not touch the walls even, and the second thing, if it does, then what kind of stresses would it produce?

MR. FISHMAN: May I amplify that question?

MR. ASCHAR: Sure.

MR. FISHMAN: The background of this question, which seems like it is coming out of nowhere, is really in

response -- is related to the calculation of the impact springs, which was calculated based on the cells and channels, and in fact it does not even use the girdle bar at all in the calculation, and it is a relatively small number, 36,000 pounds per inch.

And you said, well, let's up it to 50,000 as being the realistic number. And then you said, well, let's make the baseplate spring double that as the realistic number for the baseplate spring. And then you add multiple factors of 10 on the realistic numbers to get your conservative numbers.

Now, it is our feeling that a realistic number for the baseplate may be larger than your conservative number, and if that were to be the case, then perhaps the impact loads at the baseplate might be understated, under-computed, and that is the basis of this question, that we feel that all of your spring constants, as called conservative, are conservative except perhaps this one, and you may be able to show us otherwise.

MR. TRAMMELL: Would you like to consult?

MR. TRESLER: Let's consult for a second.

(Discussion off the record.)

MR. SINGH: Looking at like the EA over L calculation for the baseplate stiffness, it is like a beam in-plane load at the end of the plane. The impact load

would be the in-plane load, and from the centroid to the edge of the plate being the distance.

We do the calculation. We find the stiffness to be 374 kips per inch, this in comparison to 100 kips per inch used in the realistic model and 1000 kips used in the conservative model. That is the value of the stiffness as they stack up.

Now, we also went back and looked at the information we have given you earlier, how the impact load at the baseplate location compares to that at the girdle bar location, and for records in Acorn 10, which was the coefficient of friction .8 conservative model, the impact load at the top is 76 kips at the girdle bar location, and the corresponding maximum load during the duration of the earthquake at the baseplate location is 36 kips. This is assuming identical gaps top and bottom.

So you see that assuming equal stiffnesses at the bottom and top, which was the case in the conservative model, the impact load at the bottom is much less than that at the top, as one would expect, because that tends to do some rocking. So the impact initiates at the top and much of the impact energy is taken at the top, and the impact loads at the bottom are lower.

Now, subsequently, in the realistic model runs we made the baseplate stiffer than -- the baseplate impact

stiffer than the top by a factor of 2. We find the same pattern, a coefficient of friction .2. The maximum impact load is 35 kips, recorded in the PG&E's most recent submittal, that load at the top. The corresponding maximum load for that case at the baseplate location is 17 kips.

To further buttress the argument, we look at the next case with .8 coefficient of friction. The submittal shows the girdle bar location to be 85 kips. That is the maximum load again. The corresponding load for that case is 44 kips at the baseplate.

Now, in these two cases the baseplate is much different, twice the value at the top. So being that we noticed that the baseplate loads are always smaller, less than half in most cases, close to half at the most, and the spring constants are on the order of what we calculate, the conclusion can be drawn that the impact loads at the baseplate locations are much smaller than the maximum value given, as represented in rack-to-rack loads in our submittals to you.

MR. FISHMAN: In your EA over L calculation, what was your dimension? What were your dimensions?

MR. SINGH: The half ones would be 54 inches. E is 29 million psi, or 29,000 ksi, and the thickness of the plate is 5/8ths of an inch.

MR. FISHMAN: 5/8ths?

MR. SINGH: 5/8ths, yes.

MR. ASCHAR: The impact load for the two coefficients of friction, are they between the plate and the plate or between the plate and the wall?

MR. SINGH: I need to check that before I give you an answer.

MR. TRESLER: The relationship would be the same. The issue of the thickness of the spring, that issue remains the same whether it is rack-to-rack or wall-to-wall, or how does a stiffer spring affect the relationship, right?

MR. FISHMAN: We were not too concerned about impact on the baseplate from rack-to-rack. The baseplate itself is pretty sturdy.

MR. TRESLER: My question was how did it affect loads?

MR. FISHMAN: What was the concern was the pool wall, not loads in general.

MR. TRESLER: These relationships of these loads is a maximum of that and may apply to the wall. That is one thing we know.

MR. DE GRASSI: The final question is how does that affect the integrity of the wall?

MR. SINGH: The rack-to-rack loads, baseplate to baseplate. The rack-to-wall loads were zero in these runs.

MR. SOLER: It will never hit the wall?

MR. ASCHAR: The baseplate will be protruding more than the girdle box. The baseplate may be hitting. Even though it is protruding, it may not hit.

MR. SINGH: The baseplate protrude. For manufacturing tolerance the loads go up. The loads, you see, are a fraction of the maximum load at the top. They may become more comparable.

MR. ASCHAR: We are not concerned about the load between baseplate and baseplate.

Is it possible that the baseplate can hit the wall liner and can puncture it because it is only 5/8ths inch thick instead of 3-1/2 inch. That is our concern.

MR. SOLER: The situation with Acorn 10 with the stiff springs is that it never even comes close to the wall. The maximum displacements are on the order of half an inch. That is maximum in any direction.

MR. ASCHAR: For the plate?

MR. SOLER: For the worst point on the rack. You want to add an inch or something for something protruding, you still do not come close to the wall.

MR. COFFER: You say puncture the liner. Why do you care about that? We have indicated before that loss of the liner is not a concern.

MR. SINGH: A puncture of the liner, a local puncture of the liner is not a governing condition.

MR. COFFER: We said that in the FSAR.

MR. TRESLER: Although we have continually demonstrated that that would not occur.

MR. COFFER: That may not be, but it is not a safety criteria. There is no concern with puncture of the liner.

MR. SINGH: And if the material of the liner in the 304 standards -- elongation of that material before rupture is typically between 45 to 51 percent. You have a concrete backing. You will leave an enormous amount of local crushing of concrete to create a rupture. You get deformation, but that is about it.

MR. ASCHAR: The response is such that the baseplate will not hit the wall. Then we do not need to think about how to consider that load on the liner. If there is a possibility can hit the wall in some particular conditions, then we go into the detail as to how -- or into the possibility, or if there is safety concerning it.

MR. SINGH: Being that we have designed the racks to impact the wall, the design intent had been that the baseplate can hit the wall. There is no compunction making the statement here as a designer that the baseplate can hit the wall.

The loads can develop to the levels of load that we have used as a design basis and that, as Chuck said

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earlier, as a standard design for Level B conditions for SSE or Hosgris conditions, is that you could have local puncture. Even that situation would be impossible with the kinds of loads, the level of loads, the kind of direct impact loads we are talking against the liner, which is backed by the concrete. You need large deformations there, real large deformations to create any kind of a puncture in the 304 standards.

So it is a very remote -- the question has -- I do not want to say here -- tell you here that the actual design only impacts at the girdle bar location. That is not the case. They are designed to impact at the baseplate locations, also.

Whether a particular set of runs show no impact, that is fine, good and dandy, but we do not want to tell you that that is how a design basis is. It impacts the baseplate, also. That is perfectly legitimate, and there is no problem meeting any of the stress limits or safety conditions considerations.

MR. ASCHAR: Are there any conditions in the licensing report or the seismic report showing that there are no strains on the load?

MR. SINGH: The prior licensing submittals that I was involved in where the question of the liner came up, it was clearly accepted by the NRC that you could -- the

rupture of the liner per se is not a concern under SSE conditions, which correspond to the Hosri conditions here, and therefore such analysis need not be reported. That has been the NRC's licensing position in other submittals.

This liner is considerably thicker than many liners in many pools.

MR. TRAMMELL: What is the thickness?

MR. BHATTACHARYA: The walls are 11-gauge, which is very close to an eighth of an inch thickness.

MR. TRAMMELL: Not particularly thick. Most walls are quarter of an inch.

MR. TRESLER: The floor is quarter inch?

MR. BHATTACHARYA: The floor is quarter inch.

MR. FISHMAN: I just want to double-check the numbers you told me. You said that the simple-minded calculation, EA over L, yielded 374 kips per inch?

MR. SOLER: Based on a half-length.

MR. FISHMAN: That is what I am trying to understand. The length, you are taking half the length, okay, and the thickness is .625, and what was the width?

MR. SINGH: 30 inches.

MR. FISHMAN: 30 inches, okay. My calculations, I don't get that.

Do you have your calculations there?

MR. TRESLER: Where is the calculation for this?

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MR. SINGH: What are you getting, Howard?

MR. FISHMAN: I worked backwards. I took your number, and I got a width of -- and I may be entirely wrong. It is late in the day -- of only 1.15 inches, an area of .696 inches in that calculation.

As I say, I may have slipped a decimal point. It is late in the day.

MR. BHATTACHARYA: The number in the notes says it assumes sigma, which is the stress, is 30 ksi.

MR. FISHMAN: You are not doing EA over L. You are doing some other calculation.

MR. BHATTACHARYA: The elasticity is 29 million. The corresponding strain is .001 for a 54-inch length, which is half the length of the baseplate. This corresponds to a delta, which is the deflection, the actual shortening of two times .108, .216. And then force, that is corresponding to 30 ksi -- the delta -- let me go back again and correct the delta value. The delta value is .001 times 54, which is .054 inch, and then the force is .625, which is a thickness of the plate times 30 ksi, which corresponds to 18.75 kips.

MR. FISHMAN: You are missing a width.

MR. BHATTACHARYA: One inch. And that corresponds to K value of 18.75 divided by .108 -- by .054.

MR. FISHMAN: But now that is per inch. But we are interested in the -- so your spring constant was 374

kips per inch per inch, right?

So now we agree on our calculations. Now, you have to multiply by an appropriate width.

MR. SINGH: We have springs at each of the two corners.

MR. FISHMAN: Multiply it by the width and then we will talk about it. I am not saying that you should use a width of 54 inches either. If you want to say, well, when it hits at the corner, only five or 10 inches are in contact, that is for you to say.

MR. SINGH: Right.

MR. FISHMAN: Whether we agree with it or not, that is another story.

MR. SINGH: I guess the key position is if the spring constant of the baseplate was assumed even larger, the numbers that we gave you show that the impact loads at the bottom tend to be smaller than those at the top because of the behavior of the rack. The rack does not just slide over. It tends to hit at the top first. Most of the kinetic energy is absorbed at the top no matter how soft the spring, and then the bottom impacts and some loads develop at the bottom.

Even the cases where we have the higher spring constant at the base, we find that the loads tend to be 50 percent at the top. If one were to go further and take a

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larger spring constant, even larger, you would find the same general trend. You are not going to have a 36 kip load going out to 200 kips.

Our generic sense tells us that 36 kips may become 56 kips, 60 kips, but the load on the wall can be as high as 200 and -- how much is that, Shan, 215? The impact load, allowable impact?

MR. BHATTACHARYA: At the lower elevation the load is transferred to shear so the wall can carry; depending on the high load, the wall has to --

MR. FISHMAN: Fine, and earlier on we spoke about the allowable load on the pool wall being based on hydrostatic and sloshing and impact, impact at the girdle bar level.

MR. BHATTACHARYA: As well as the impact obtained from this calculation for the baseplate elevation.

MR. FISHMAN: And a few moments ago Chris said you were not responsible to compute any puncturing effect in the liner?

MR. SINGH: That is right.

MR. FISHMAN: That is correct. So consequently, you were saying that even if the impact load at the baseplate were significantly increased, it would not significantly -- because it is so low down to the ground, it would not greatly affect the stresses in the reinforcing

bars and concrete.

MR. MARTORE: The baseplate is significantly below the girdle bar load to start with, and the second is that the wall evaluation showed significant margins of safety.

MR. SINGH: You are right, Howard. That is correct.

MR. FISHMAN: Our concern was with those racks that were fabricated where the baseplate extended beyond the girdle bar, and if any of those happened to be adjacent to the wall, that is where our concern came from.

MR. COFFER: Have we addressed that?

MR. FISHMAN: You addressed that problem, and your answer helps fill in this missing blank space on this answer to the question.

MR. TRESLER: If we truly need to fill in that missing blank space, we may wish to do some calculations other than what we did over a five-minute break.

MR. FISHMAN: A half-space calculation may be more appropriate.

MR. TRESLER: If it is an issue, then it needs to be filled in.

The other thing Chris said is that we did look at the calculation, the 2-D multi-rack calculations, where we had presumed that the bottom protruded beyond the face of

the girdle bar, and we still found that the baseplate loads were lower than those on the girdle bar.

MR. FISHMAN: That was based on the "conservative" distribution of springs.

MR. TRESLER: Realistic, I recall.

MR. FISHMAN: Which might be off by a factor of 100 or so in the baseplate, maybe.

MR. TRESLER: Maybe.

MR. FISHMAN: I am satisfied.

MR. ASCHAR: I think it is worthwhile for us to have some calculations documented on the baseplate spring constant as you calculated just now, maybe later more refined, and show that when you include girdle bar impact loads, the baseplate load on the wall, the walls have similar safety margin, as you indicated earlier.

MR. TRESLER: Was that already provided?

MR. ASCHAR: You say you included 215 k loads from the girdle bar.

Did you include anything from the baseplate load?

MR. BHATTACHARYA: Yes, we have. We have included -- 125 kip load for the design of the wall, wall evaluation, 125 kip load at the elevation of the girdle bar.

MR. FISHMAN: Which document are you talking about?

MR. BHATTACHARYA: These are our calculations for

the wall, the parametric studies.

MR. TRESLER: We previously provided this information to them.

MR. BHATTACHARYA: The girdle bar loads, no -- girdle bar, yes, but the baseplate we have not.

The factor of safety that we had provided earlier in response to Question No. 5 included 215 kip load at the girdle bar and 125 kip load at the elevation of the baseplate simultaneously. 125 kip load is primarily transferred through shear, and it does not have any influence on the bending moment.

MR. ASCHAR: You are considering a cantilevered wall?

MR. BHATTACHARYA: This particular wall indeed does not have support at Elevation 140, the deck for the fuel handling building, so we have treated it like a wall supported on three sides.

MR. ASCHAR: Then it would not only be shear load; it might have bending as a plate?

MR. BHATTACHARYA: The baseplate is located about eight inches above the bottom of the fuel pool.

MR. TRESLER: Is that enough information to address this issue?

We considered 125 simultaneously with 215. We understand that the baseplate acts just eight inches off the

Wbur

floor.

MR. FISHMAN: Can you make a comment if that 125 at the baseplate was doubled? How would that affect the margin?

MR. SINGH: The spring house --

MR. FISHMAN: No. Shan described the situation which may have come directly from the calculations and may not have, where they had doubled the load at the girdle bar at the baseplate, which seems to follow the trend in a lot of your calculations.

MR. TRESLER: I think the wall safety factor, as I recall, was 1.4 with 80 kips at the girdle bar and 0 kips at the baseplate. With 125 kips at the baseplate and 215 kips at the girdle bar, the factor of safety only went down to 1.3.

So that has got to give us a reasonable perspective.

MR. BHATTACHARYA: My judgment is that if the load goes up to twice the value, if you postulate that the load is twice the value at the baseplate, we still are going to be within the allowable. The primary shear load is coming from the hydrostatic shear, and then it is coming from the sloshing effect, and the third thing is the vibration for the six-foot thick wall. It is the out of plane vibration of the six-foot thick wall.

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(Discussion off the record.)

MR. TRAMMELL: Back on the record.

We don't have any further questions, except we would like to have PG&E get back to me on the fact that the liner is not safety related.

I do not think that is terribly unusual, but if it is in the FSAR, would you point me to it so I can see that?

I expect that that is the case. I would like to see it for myself.

And, finally, we hope to get the transcripts and meeting summary out very quickly, today, maybe tomorrow, maybe Friday, very quickly, and we would ask that PG&E look at the transcript when you get it, and if there is anything in there that is incorrect or you wish you had not said or you did not say but wish you had said, please write us a letter and tell us about that because it is possible there are some errors in it and he knows. If it is not correct, we want to hear from you.

MR. TRESLER: I, Charlie, would like to say that we have thought some more about the baseplate stiffness that we calculated on that five-minute break and feel that it is really not an appropriate approximation of the stiffness of that plate.

So if this is an issue, I guess we need to know

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that, and we would provide you with a more appropriate stiffness.

MR. ASCHAR: That would be appreciated.

MR. TRESLER: Can we do that by a phone call so that your review is not delayed, your preparation of testimony for the hearings is not delayed?

MR. FISHMAN: A calculation sheet could be faxed to us.

MR. TRAMMELL: We are happy to talk to you on the telephone, but to the extent that you are going to rely on this information, we would like to have it in letter form to all the parties, and so forth.

MR. TRESLER: Fine.

MR. TRAMMELL: I think that would be preferable.

MR. TRESLER: Okay.

MR. TRAMMELL: Do you have anything further, Hans?

MR. ASCHAR: No.

MR. DE GRASSI: No.

MR. FISHMAN: No.

MR. VOGLER: No.

MR. TRAMMELL: Thank you very much. I appreciate it.

MR. TRESLER: The final meeting is over.

(Laughter.)

(Whereupon, at 3:16 p.m., the meeting was adjourned.)

CERTIFICATE OF OFFICIAL REPORTER

This is to certify that the attached proceedings before the UNITED STATES NUCLEAR REGULATORY COMMISSION in the matter of:

NAME OF PROCEEDING: MEETING RE: DIABLO CANYON SPENT FUEL POOL RERACKING

DOCKET NO.: 50-275, 50-323

PLACE: BETHESDA, MARYLAND

DATE: WEDNESDAY, MAY 6, 1987

were held as herein appears, and that this is the original transcript thereof for the file of the United States Nuclear Regulatory Commission.

(sigt) Barbara L Whitlock

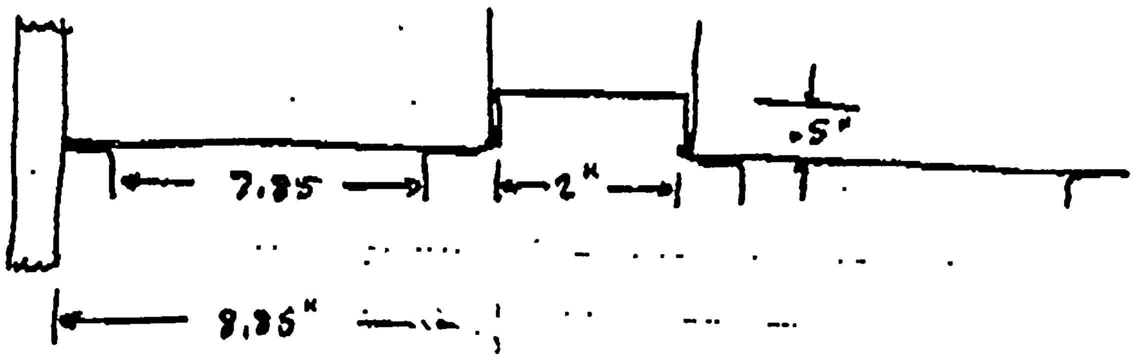
(TYPED)
BARBARA L. WHITLOCK

Official Reporter
ACE-FEDERAL REPORTERS, INC.
Reporter's Affiliation

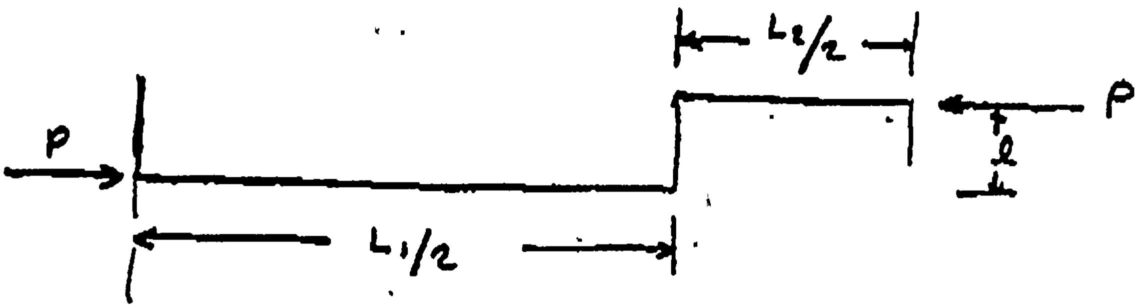
Attachment 1.4

TOP GRIDWORK
CALCULATION OF IMPACT SPRING

GRIDWORK



Consider the following analysis for flexibility



$$M_1 + M_2 = P\ell$$

$$\theta = \frac{M_1 L_1}{2EI} = \frac{M_2 L_2}{2EI} \Longrightarrow M_1 L_1 = M_2 L_2$$

$$\therefore \theta = \frac{M_1 L_1}{2EI} \quad ; \quad M_1\left(1 + \frac{L_1}{L_2}\right) = P\ell$$

$$\delta = \ell\theta = \frac{P\ell^2 L_1}{2EI(1+L_1/L_2)}$$

$$\Delta = 2\delta = PL_1\ell^2/EI(1+L_1/L_2)$$

Hence if $P = K\delta$, then the spring constant for one grid wall is

$$K = EI(1+L_1/L_2)/\ell^2 L_1$$

where we assume that all flexibility occurs in the first cell plus connecting channel. $I = wt^3/12$ where w is the "depth" of skin assumed active in supporting load from the girdle bar. We consider that all flexibility is absorbed above the active fuel area (i.e. $w \le 20''$)

For calculation purposes; assume $L_1 = 7.85''$ $L_2 = 2''$, $\ell = .5''$, $t = .08''$, and take $w \approx 2 \times$ depth of girdle bar. Then for one cell wall, and assuming only the first cell, we obtain

$$K = \frac{27.9\times10^6 \times 6 \times (.08)^3 \times (1+\frac{7.85}{2})}{12\times.25\times7.85} = 17924\ \#/in$$

Assuming two cells deep provide flexibility to load on girdle bar, then yields

K (per each .08" wall) = 8962 #/in

Now if we assume that the impact load is spread over 2 cells (along the girdle bar), then the final value for K to be used in the simulation is approximately equal to

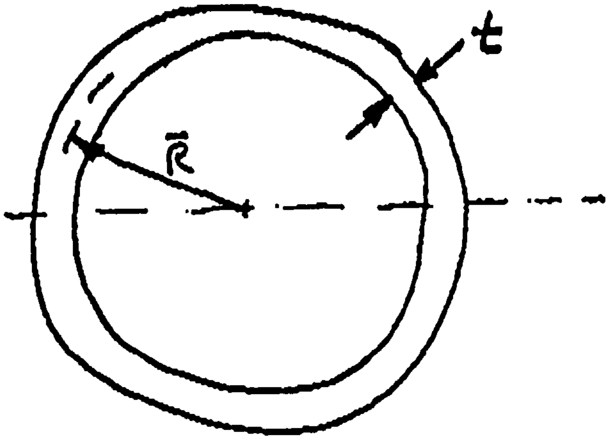
$$K_{FINAL} = 4K \approx 36000 \ \#/in$$

We use $.1 \times 10^6$ (2 impact springs) at the top (rack/wall) and $.05 \times 10^6$ for rack/rack in the simulation.

RESPONSE TO NRC QUESTION 6

6. The following are calculations, based on the actual configuration J.O. Drawing D-7708, Rev. 5, to determine the safety margin that exists in the weld between the base plate and the upper part of the support leg.

Consider a weld circle of mean radius R and width t. The area and the moment of inertia are given as



$$A \cong 2\pi \bar{R} t$$
$$I \cong \pi \bar{R}^3 t$$

The weld between the upper support leg and the base plate consists of two such circular elements:

$\bar{R}_1 = 4.5''$ $\quad$ $t_1 = 1.25''$

$\bar{R}_2 = 3.4375$ $\quad$ $t_2 = .625''$

Therefore, the total weld metal available to support direct load (compression only), bending moment, and shear loads is

$$A_T = 6.28\,(4.5 \times 1.25 + 3.44 \times .625) = 48.83 \text{ in}^2$$
$$I_W = 3.14\,(4.5^3 \times 1.25 + 3.44^3 \times .625) = 437.78$$

The weld stress (max. absolute value) due to support compression and bending is given as

$$\sigma = \frac{Mc}{I_W} + \frac{N}{A_W} \qquad c = \bar{R}_1 \qquad (1)$$

Note we have made the conservative assumption that only the weld supports compression; that is, we neglect any contact between the base plate and the support metal top surface.

From the design basis seismic calculations, we compute the stress in the upper part of the support based on the following considerations. Let A_f, I_f be the upper support metal area and metal inertia. Using the as installed dimensions $A_f = 25.34 \text{ in}^2$, $I_f = 258 \text{ in}^4$ (p II-56 of the design basis seismic report).

If σ_A is the allowable stress for the upper portion of the support material in compression and bending, then the design basis seismic results permits back calculation of the normal force N and bending moment M from the following equations.

$$N = (R_1 \sigma_A) A_f \qquad (2)$$

$$M = (R_3^2 + R_4^2)^{1/2} \sigma_A \frac{I_f}{\overline{R}_1} \qquad (3)$$

where R_1, R_3, R_4 are the maximum values for the stress ratios R_i defined in the seismic report and obtained from design case simulations at the most critical section of the support.

Using (2), (3) in (1) yields the weld stress as

$$\sigma = \sigma_A \left[(R_3^2 + R_4^2)^{1/2} \frac{I_f}{I_w} + \frac{R_1 A_f}{A_w} \right] (1.414) \qquad (4)$$

where we have included the factor 1.414 to correct I_w, A_w for the minimum throat dimension of the weld.

From the seismic results for ACORN 10, corrected for actual area and inertia, and using $\sigma_A = 33000$ psi. at section 6

$R_1 = .81 \times .306 = .248$

$R_3 = .63 \times .958 = .604$

$R_4 < .63 \times .509 = .321$

Therefore, using equation (4) yields

$$\sigma = 33000 \times 1.414 \left[.68 \times \frac{258}{438} + .248 \times \frac{35.34}{48.83} \right]$$

$= 27066$ psi (maximum compression)

NF of the ASME Code provides the following stress limits for Level A condition.

Fillet Weld: Direct tension stress limit is 18 ksi for 304L material base plate.

Groove Weld: Direct tension stress limit is 21 ksi for 304L material base plate.

These allowable stresses are increased by a factor of 2 for Level D (Hosgri) condition.

Based on 18000 psi allowable stress for a fillet weld and multiplying by a factor of 2 (HOSGRI) yields the safety margin for the weld at the point of maximum bending as

$$S.M.)_{BENDING} = \frac{2 \times 18000}{27066} = 1.33$$

At the location of maximum shear stress, the weld stress in shear should be such that

$$\sigma/\sqrt{3} \geq \tau$$

where the maximum weld stress τ is related to the average shear stress on the gross section of the upper support by the following formula

... $\tau = 2\,\tau_{AV}\frac{A_f}{A_W}$ where τ_{AV} is the average shear stress on the gross section.

From the seismic report, the maximum value of the stress factor for maximum shear in either the x or y directions is $R_2 = .81 \times .349 = .283$ after the correction is applied for the as installed section area. The information available is for shear in either x or y direction. The combined shear is calculated as

$$R_2^* = \sqrt{R_{2x}^2 + R_{2y}^2}$$

We estimate R_2^* by using the information from the bending stress factors R_3, R_4 to apportion the shear factors in the respective directions. Therefore an estimate of the correct R_2^*, including both x and y components, is

$$R_2^* = \left[.283^2\left(1+\left(\frac{.321}{.604}\right)^2\right)\right]^{1/2} = .320$$

Hence, the maximum shear stress τ is

$$\tau = 2\,(R_2^*\,\sigma_{AS})\,\frac{A_f}{A_W}$$

where σ_{AS} = 20000 psi is the allowable upper support stress used in the ACORN10 analysis. Amplifying the response by 1.414 to account for throat area yields for the maximum weld stress τ the value

$$\tau = 2 \times 1.414 \times 20000 \times .32 \times \frac{35.34}{48.83} = 13099 \text{ psi}$$

The safety margin on shear stress is, using 18,000 psi for σ

$$S.M.)_{SHEAR} = \frac{2 \times 18000}{\sqrt{3} \times 13099} = 1.587$$

Note that the above margins will be further increased based on stress factors obtained from DCL-87-082. Finally, we note again that the effect of direct contact between base plate and upper support leg surface to aid in the transfer of compressive stress is neglected