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Docket No.: 50-275/323

LICENSEE: Pacific Gas and Electric Company (PG&E)

FACILITY: Diablo Canyon Nuclear Power Plant, Units 1 and 2

SUBJECT: TRANSCRIPT CORRECTIONS TO TRANSCRIPT OF MEETING HELD ON MAY 6, 1987, SPENT FUEL POOL RERACKING AT DIABLO CANYON

On May 6, 1987, the NRC staff met with PG&E and its consultants to discuss the seismic analysis of the proposed high-density spent fuel storage racks. The meeting was transcribed. PG&E was requested to provide any needed transcript corrections.

The NRC staff has briefly reviewed the corrections provided by PG&E and found none that it did not agree with. The transcript corrections are enclosed.

Original signed by

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Enclosure:
Transcript corrections

cc: See next page

DRSP:PDV
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6/ /87

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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

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ORIGINAL

MASTER
Includes
SB, AS, KS, JM, DWO

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

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P R O C E E D I N G S

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~~ PG&E.

MR. ~~MATORE~~ MARTORE: J. ~~Matore~~ Martore, 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

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 II-29A ~~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 cell wall spring 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

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~~ as the realistic value. The calculated value was .36 ~~E~~ times 10 to the 5th.

The baseplate, which is simply a flat plate, roughly ~~1~~ 108 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~~ 10 to the 7th ~~model.~~ value

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

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~~ in 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 used a factor of 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?

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MR. SOLER: ~~And~~ In 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.~~ legs 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

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 gives 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.

(Slide.)

In the analyses reported in Reference 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 plate ~~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, DCL-87-022 ~~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 conditions 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

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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~~ spacing to into the wall and G effective equals 1.0 inch for a 9-inch spacing ~~into~~ to 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~~ term 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 sub 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 also two terms, at the lower right-hand end of the mass matrix. Those terms do decrease by a small amount. ~~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 G effective GEXℓ= 1 over GL plus 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~~ It's 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

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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~~ off the top of my head, I can not give 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

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 over gap 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.

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~~ as appropriate to a plate moving in a large fluid medium, and 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

MR. SOLER: Question 3: Provide a ~~Russian offer~~ rationale for 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~~ sub e of one inch, and g, ~~sub B~~ sub e of .884 inches in the north/south direction.

These effective gaps were used in calculating the hydrodynamic coupling coefficents. In ~~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~~ hydrodynamic 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

1 something you wanted to say.

2 MR. ASCHAR: The staff finds responses to

3 questions 2 and 3 adequate. However, we might revisit them

4 in the general line of questioning at the end of the six

5 questions. So we can go to 4.

6 MR. SOLER: Describe how hydrodynamic coupling

7 effects induced by rocking and torsional motion of the ~~raps~~ racks

8 as mentioned on page 3 of reference 5 were incorporated in

9 the realistic cases analyzed in references 4 and 5.

10 The effect of rack rocking and torsion had been

11 included in the analysis by considering the actual channel

12 configuration constrained by real or hypothetical boundaries

13 (which you see will simulate motion of adjacent racks). And

14 by tracking the fluid movement in the channels consistent

15 with the postulated rack motion. Integration of the fluid

16 kinetic energy over the height and width of the rack yields

17 rocking and torsional coefficients.

18 The step by step procedure used to develop these

19 coefficients is based on the models shown in figure 1.

20 (Slide.)

21 Step one, the fluid velocity across the flow

22 channel is assumed to be linear and is zero at the boundary

23 and is equal to that of the rack at the face of the rack.

24 Step two, flow velocities along the channel are

25 then calculated for each flow channel using the principle of

1 continuity.

2 For example, for Region I, the equations for the
3 two fluid velocities, V-1 and U-1, are shown also in figure
4 1.

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

8 (Slide.)

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

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

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

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

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

rocking, and, of course, translation.

MR. DEGRASSI: Also in the conservative models?

MR. SOLER: ~~Indeed,~~ In the 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

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~~ in each 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

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~~ insignificant change in the value of that particular B coefficient, simply because of

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

Table 4 of item 15-G of reference ~~4~~ 3 provides the results for the ~~outer~~ out-of-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~~ effect 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 credit ~~period~~ is taken for increas~~ing~~ed allowable stresses due to rapid strain rate effects associated with the impact.

The critical wall ~~A-6~~ AD (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, punching shear 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~~ and 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~~ ratio 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

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~~ walls are not all six feet -- 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 shows that is not the critical wall.

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

in the general questioning, we might revisit it.

MR. SOLER: Question 6:

Provide clarification of the weld stress calculations shown on page ~~271~~ II-71 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

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1 times 18 or 21 K. Maybe you can look into it and let us
2 know later.
3 MR. SINGH: We will quote the appropriate
4 paragraph from where we have derived this information.
5 MR. ASCHAR: You can look through it later on
6 maybe and show us.
7 MR. FISHMAN: Could you show the sketch again?
8 (Slide.)
9 MR. SINGH: The fillets are on the outer diameter ~~diagram,~~
10 and the groove is on the inner diameter. They are all
11 5/8ths.
12 MR. TRESLER: Hans, are you talking about -- when
13 you say you don't believe -- at least your interpretation is
14 that you do not believe that a factor of 2 should be used.
15 Is that your interpretation for the weld material
16 allowables, which would exclude the baseplate material, or
17 are you interpreting that to apply to the baseplate
18 material, also?
19 MR. FISHMAN: Our understanding is you must use
20 the weld allowables based on the weakest parent material
21 that is attached, and our interpretation of that table in
22 Section NF for the weld allowables states that the groove
23 weld, you can use 21 ksi.
24 We did see in the NF the factor of 2 for Level D,
25 but it did not seem to us that -- or to myself -- that it is

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~~ allowable 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 by one and 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~~ corrects 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~~ 0.42 S sub U 2 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~~ bar and a plate, the weak direction of the plate is in the ~~true~~ through

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thickness direction. The plates are rolled stock material and typically the strength in ~~point~~ the in plane direction is greater, the ultimate strength in ~~point~~ plane is greater than the ~~transfer~~ transverse 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~~ shear 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~~ meet the stress limits here. We still ~~need~~ meet 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

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1 sizing members, and that some how got in the report that ~~we~~ may
2 have thrown you off. The governing code is this, and the
3 allowable stresses are from here. Our requirements are more
4 restrictive, but they should not have been in the seismic
5 report per se.
6 MR. FISHMAN: Based on this, what is your margin
7 of safety for the worst weld in the design report?
8 MR. SINGH: If I take a literal interpretation of
9 the code, as you see, the verbiage permits me to multiply by
10 a factor of 2 and then increase it by 1-1/3, which, again, I
11 believe has been in later addendums changed. It is now only
12 a factor of two. If I go strictly by this, the factor would
13 be even greater, but the presentation that Dr. Soler made,
14 that shows the factor to be 1.5 versus the allowable of 2.
15 So the ratio of the two is the margin. 2 divided by 1.5 is
16 1.33; isn't it? Yes. 1.33 is the ratio allowable to the
17 actual.
18 MR. FISHMAN: Are we talking about one of the Rs
19 or a separate calculation that happened to be derived from
20 the R number?
21 MR. SINGH: Just to get the moment in the shear,
22 the juncture --
23 MR. SOLER: We have the calculations.
24 MR. SINGH: The R numbers were used to calculate
25 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 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 square root of 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.

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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~~ groove.

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

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~~ low coefficient of 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~~ call 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

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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 on four legs for more than a fraction of the duration of the earthquake. It is on four legs, as opposed to a high coefficient of friction case ~~which~~ when it tends to stick, and it may be ~~a~~ one leg or two legs for a ~~later~~ longer fraction of the ~~term.~~ time

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~~ impediments 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~~ climb up to the peak of the response spectrum ~~that amplifies~~ we amplify the response, now if you take the coefficient of friction ~~were~~ where 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 amplication ~~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 kept ~~keep~~ it as it was, we used a value of vertical spring constant which obviously exaggerates the vertical response. And in the case of low ~~load~~ coefficient of frictiion, gives a higher impact load, contrary to what one would expect with a -- if one was ~~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 have ~~hae~~ to add, in a

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~~ for the .8 coefficient of friction case.

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~~ friction .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~~ kip 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~~ these 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~~ MR SINGH: if you look at the single model in the

1 vertical direction, you would see that the spring constants
2 we have used, realistic spring constants, they put us smack
3 on the peak of the curve. There is no way to go but down.
4 We have as best one can using analytical models
5 tried to maximize the calculated response. We are not
6 coming in here with numbers.
7 If you look at the people -- people projecting
8 response characteristics of the structure by looking at the
9 response spectrum in the vertical direction, it is most
10 meaningful because that is where we have no coupling
11 effects, nothing to complicate matters.
12 We have put it smack on the peak of the response
13 curve, the realistic spring constant.
14 MR. DEGRASSI: Of the vertical response curve.
15 MR. SINGH: Yes. It is sitting right on the
16 speak. So I would say with good confidence that reducing
17 the spring constant any further or increasing it would
18 reduce the response.
19 MR. DEGRASSI: Isn't it primarily the rocking
20 response that would increase when you go to the more
21 realistic or lower spring values?
22 MR. SINGH: The vertical translation of movement
23 and rocking. And they of course interact with -- the rack
24 tends to lift off and the rack can also rock more. Any
25 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, or 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

BWH/bc

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~~ fluid coupling terms and, therefore, it does not have a spring mass collective characteristic, 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,

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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~~ shear getting transferred to the body of the rack ~~times~~ which is 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~~ shear 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 shear -- the coefficient of friction times the vertical force, typically the limiting condition before sliding occurs, is the horizontal ~~share.~~ shear

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~~ reactions 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,~~ Actually, you have horizontal sheer force on whatever support legs are in ~~content.~~ contact. You have inertia force. You have the ~~racking~~ rocking 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 a moment that is less.

When the impact force is high, that can well happen. ~~So you do not~~ However, -- 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~~ support 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~~ shear 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

1 question as to which force is the most important when you
2 can generate all of the time history curves, as you saw at
3 the last meeting, of various forces impacting, and you
4 cannot draw conclusions to a specific question that says:
5 This is the force that is more important than
6 this force.
7 MR. TRESLER: One of the things that we cannot
8 lose perspective on is even though we did have a slight
9 increase in stress ratios for the more realistic springs and
10 ~~hypodynamic~~ hydrodynamic coupling at .2, we still have tremendous margin
11 to allowables.
12 There was a small change at a very low level and
13 still all of those values were significantly below the
14 ratios that we had for the realistic loads and stress ratios
15 out at the .8 coefficient of friction case.
16 To put it in perspective.
17 MR. DEGRASSI: That is a significant point.
18 MR. CONGEL: The delta in terms of the spring
19 constants going from what we had is conservative to
20 realistic. It is certainly a bigger change than what you
21 are asking about from one and a half to one; in that big
22 change, we did not have very much increase.
23 MR. TRESLER: But I think, as to what happened, I
24 think Chris has done a good job explaining it. To be honest
25 with you, we were somewhat bewildered, too. And there was a

fair amount of work done as a result of this increase which lead to misunderstanding ~~which caused a~~ rack response. fall ~~in~~ on 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~~ time 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~~ rack 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 in the off diagonal components. So the direct deduction from response spectrum is obscured.

The ~~reductions~~ deductions 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

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mass which is connected to the base, which is attached to the base of the rack. That can vibrate linear~~ially~~ly, 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 of impact ~~levels.~~ loads

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

1 up with as reported in the seismic report.

2 But, again, our revised calculation shows that

3 there is ample factor of safety in the strain calculations.

4 Our original report showed that the factor of safety for

5 liner plate was on the order of 4.5. And the liner anchor

6 displacement design, the factor of safety was on the order

7 of -- for a liner plate is a factor of safety of 3.2 per

8 liner anchor displacement. On the floor was 4.5.

9 MR. FISHMAN: What do you mean by liner anchor

10 displacement? Bed plate?

11 MR. BHATTACHARYA: Division Two has a specific

12 criteria for the liner as well as the anchor.

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

14 MR. BHATTACHARYA: Those factors were developed

15 based on a load of -- these factors were based on the

16 maximum value of accumulated reaction on four feet. That

17 means an interior rack 10 by 11. You know, we have only one

18 10 by 11 rack, but the assumption was that the interior 10

19 by 11 rack on all four feet are sitting on one bridge plate.

20 And a total load that we got, the reaction was 677 ~~chips~~ kips.

21 And that value corresponds to a factor of safety

22 that I just mentioned here.

23 MR. DEGRASSI: Have you looked at potential

24 punching of single feet?

25 MR. BHATTACHARYA: Yes.

MR. ~~TRESLER:~~ TRAMMELL: 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:~~ TRAMMELL: 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:~~ TRAMMELL: So you bridge across these leak ~~plate~~ chase 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

1 any leakage. So we ~~cut through~~ notch the underside of the bridge
2 plate ~~over it.~~ to bridge the weld crowns.
3 MR. FISHMAN: One question goes back to question
4 six, where we wanted to fully understand the weld
5 calculation at the top of the support, the adjustable
6 support.
7 We've referred to the seismic report, page 2-71.
8 In that report, there was a calculation of the weld stress
9 based on taking the largest R6, which whatever was computed
10 at that time as being the largest R6. And you essentially
11 ratioed it by an area at the weld to the area of the parent
12 cylindrical material.
13 And then did some other computational -- this is
14 not quite in accordance with what Chris earlier discussed of
15 the proper way to do this weld, treating the groove weld as
16 one portion of it and the fillet weld as another portion of
17 it, separating the areas and the moments of inertia.
18 What is the calculation that you did for this?
19 MR. SOLER: The values we have just reported on
20 in response to your question, which lead to the factors--
21 what are the numbers there? 1.5 and 1. --
22 MR. FISHMAN: Are these numbers available to us?
23 MR. SOLER: Are these available to them like the
24 other ones?
25 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.~~ kips. 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

1 comfortable with this allowable value?

2 MR. ASCHAR: Our general line of questioning is

3 over. We would caucus ourselves and comment on what we want

4 you to do.

5 MR. TRESLER: Fine.

6 (Staff conferring.)

7 MR. FISHMAN: I have nothing else.

8 MR. TRAMMELL: The Staff has no further

9 questions.

10 At this point we would like to have a Staff

11 caucus of approximately 15 minutes, at which point we would

12 resume and give you -- conclude this meeting.

13 MR. TRESLER: Charlie, we would like to interrupt

14 your caucus, if we could, when we get a copy of the weld

15 calculation you have requested we will provide it so that you can review ~~that~~ if

16 during your caucus, and if that raises any questions, we can

17 address it again in the meeting.

18 MR. TRAMMELL: Very good.

19 (Recess.)

20 MR. TRAMMELL: Back on the record.

21 I think we are ready to conclude this session.

22 First, Howard Fishman has reviewed a calculation

23 which we -- which I would like to have him describe and

24 report on.

25 MR. FISHMAN: You provided me with the response

1 to NRC Question 6, and I checked the numbers, and I think
2 you did for the most part an appropriate job in computing
3 the stresses, especially in the -- well, in the outer weld.
4 You did not take advantage of the smaller radius in
5 computing the stresses in the inner weld, which might help
6 you somewhat.
7 But for the record, I would like to say that the
8 stress computations are appropriate. The allowables are
9 still somewhat questionable.
10 MR. TRESLER: Can we understand some sort of
11 schedule for resolving the issue of appropriate allowables?
12 MR. TRAMMELL: We will get to that in a minute.
13 We can get to it right now since you brought it up. NRC is
14 not able to answer that question immediately. There are
15 some people that we need to consult with who are on the code
16 groups involved, and I imagine we can do that in a matter of
17 a few days.
18 If there is something that comes of this and we
19 need action on your part, we will contact you. As it stands
20 now, it is our ball, we need to do some checking. If we
21 need to contact you, we will.
22 I was about to say that we will attach that
23 calculation that Dr. Fishman has described to the record.
24 The second area involves the calculation
25 involving the girdle ~~box~~ barge, which Dr. DeGrassi will go over.

stiffer than the top by a factor of 2. We find the same pattern at a coefficient of friction .2. The maximum impact load is 35 kips, recorded in the PG&E's most recent submittal, that load is 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 stiffer ~~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~~ value 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~~ The relationships of these loads ~~is~~ indicates a maximum ~~of which and may apply~~ load from the upper spring will be applied 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.

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

Our general ~~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 through ~~to~~ shear so the wall can carry a higher 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

1 the wall, the parametric studies.

2 MR. TRESLER: We previously provided this

3 information to them.

4 MR. BHATTACHARYA: The girdle bar loads, no --

5 girdle bar, yes, but the baseplate we have not.

6 The factor of safety that we had provided earlier

7 in response to Question No. 5 included 215 kip load at the

8 girdle bar and 125 kip load at the elevation of the

9 baseplate simultaneously. 125 kip load is primarily

10 transferred through shear, and it does not have any

11 influence on the bending moment.

12 MR. ASCHAR: You are considering a cantilevered

13 wall?

14 MR. BHATTACHARYA: This particular wall indeed

15 does not have support at Elevation 140, the deck for the

16 fuel handling building, so we have treated it like a wall

17 supported on three sides.

18 MR. ASCHAR: Then it would not only be shear

19 load; it might have bending as a plate?

20 MR. BHATTACHARYA: The baseplate is located about

21 eight inches above the bottom of the fuel pool.

22 MR. TRESLER: Is that enough information to

23 address this issue?

24 We considered 125 simultaneously with 215. We

25 understand that the baseplate acts just eight inches off the

1 floor.

2 MR. FISHMAN: Can you make a comment if that 125

3 at the baseplate was doubled? How would that affect the

4 margin?

5 MR. SINGH: The spring house --

6 MR. FISHMAN: No. Shan described the situation

7 which may have come directly from the calculations and may

8 not have, where they had doubled the load at the girdle bar

9 at the baseplate, which seems to follow the trend in a lot

10 of your calculations.

11 MR. TRESLER: I think the wall safety factor, as

12 I recall, was 1.4 with 80 kips at the girdle bar and 0 kips

13 at the baseplate. With 125 kips at the baseplate and 215

14 kips at the girdle bar, the factor of safety only went down

15 to 1.3.

16 So that has got to give us a reasonable

17 perspective.

18 MR. BHATTACHARYA: My judgment is that if the

19 load goes up to twice the value, if you postulate that the

20 load is twice the value at the baseplate, we still are going

21 to be within the allowable. The primary shear load is

22 coming from the hydrostatic shear, and then it is coming

23 from the sloshing effect, and the third thing is the self weight

24 vibration for the six-foot thick wall. It is the out of

25 plane vibration of the six-foot thick wall.