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June 13, 1984

Dennis M. Crutchfield, Chief
Operating Reactors Branch #5
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Dear Mr. Crutchfield:

Subject: Oyster Creek Nuclear Generating Station
Docket No. 50-219
Spent Fuel Pool Expansion - Additional Information

Enclosed are responses to questions forwarded to me by your letter of June 1, 1984 concerning GPU Nuclear's request to expand the capacity of the spent fuel pool.

Very truly yours,

Peter B. Fiedler
Vice President & Director
Oyster Creek

PBF:SD:dam
Enclosure

cc: Dr. Thomas E. Murley, Administrator
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NRC Resident Inspector
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ENCLOSURE

OYSTER CREEK NUCLEAR GENERATING STATION

Additional information regarding the following is requested.

A. High-Density Spent Fuel Rack Dynamic Structural Analysis

Based on the review of the recently submitted responses by Joseph Oat Corporation (consultant to Oyster Creek) [1, 2] and the information presented in a meeting with Joseph Oat at Franklin Research Center on May 7, 1984 [3], the following questions are prepared for additional information and/or clarification.

1. In the referenced meeting [3] and responses submitted on May 10 [2], Joseph Oat indicated that the equivalent gap was developed to take into account the hydrodynamic effects on all four sides of the rack. The following reasons are given for this concept of equivalent gap:

   a. The seismic loading should be equal to zero when taking the average of the complete seismic input time history.

   b. The hydrodynamic mass, according to Fritz [4], is related to the gap (g) by the following expression:

$$\text{Hydrodynamic mass} = M_H = \frac{\text{Constant}}{g}$$

   Please respond to the following questions:

   a. Provide the technical basis as to why the seismic loading will be zero; even if this is true, it is not clear how this would be used to justify the proposed approach.

   b. Provide a discussion of why, at the instant the motion starts, the rack is assumed to be at an artificial center position instead of its actual position.

   <u>Response to Question #1</u>

   Since the seismic motion is essentially vibratory, the excitation imparted to the structures is one characterized by a large number of harmonic inputs. Harmonic inputs, by definition, have a zero net integral. A statement to this effect was made at the above referenced meeting to allay the reviewer's concern that the rack may move towards the most proximate rack and impact it before the mathematical model, predicated on g_{eq}. would predict the incidence of impact. In our response, we point out that the mathematical model for the rack seeks to replace a highly coupled system of vibrating bodies, to one wherein the rack vibrates inside a fictitious container attached to the ground. The coupling and hydrodynamic masses

are estimated by centering this rack in that container. This assumption is merely consistent with the rest; not in itself necessarily conservative. The conservatism, perhaps an excessive amount, is introduced at earlier steps in the model wherein a complete coherence of motion of all fuel assemblies at all times during the earthquake is enforced, all fluid drag effects are neglected, etc.

Recent research on the non-linear hydraulic coupling effect between vibrating bodies subject to harmonic excitations confirms the intuitive result that the smaller gap provides more of an inertial resistance to its closing than a larger gap. Consequently, an eccentrically placed body will tend to center itself if subjected to harmonic excitation of sufficient magnitude.

These concepts have provided the rationale for the model used by Oat in analyzing high density racks.

2. In the referenced meeting [3], Joseph Oat Corporation indicated that the seismic loading in three directions was applied simultaneously to the model. Please indicate whether these three input time histories are statistically independent as specified by Regulatory Guide 1.92 [5].

Response to Question #2

This rack was designed on the basis of one horizontal seismic motion and one vertical seismic motion. The time histories associated with these two motions are statistically independent as specified by Regulatory Guide 1.92. In order to study what we feel is the most severe condition (i.e., a 3-D seismic input), the one specified horizontal component was broken into two components acting along the rack x ands y directions, respectively.

3. With respect to the influence of coefficient of friction to the rack displacement, the following table is prepared based on the outputs given in Joseph Oat's submittal [6]:

Case	Rack	Loading Condition	Coefficient of Friction	Maximum X-Displacement
i	E	Full load	0.8	0.125
	F	Full load	0.8	1.298
ii	E	Full load	0.8	0.125
	E	Full load	0.2	0.655
iii	F	Full load	0.8	1.298
	F	Full load	0.2	0.535

With reference to the above table, please respond to the following questions:

a. For case i, racks E and F are very similar with the exception that E is higher that F. Explain possible reasons why F has higher maximum displacement.

b. For cases ii and iii, please provide possible reasons why a high coefficient of friction in case ii produces smaller maximum displacement and high coefficient of friction in case iii produces higher maximum displacement.

c. Please provide disolacement and base support reaction time histories for case i (both racks E and F) with the following coefficients of friction: 0.2, 0.4, 0.6, and 0.8. If this information is not available, it is strongly recommended, as a minimum, that outputs for coefficients of friction 0.2 and 0.8 should be generated for review.

Response to Question #3

While it is true that racks E and F are very similar in total mass and shape, there are some differences, which, when coupled with the highly non-linear nature of the problem, can account for the differences in maximum displacement.

a. Rack F has somewhat less stiffness for bending in the x direction.

b. Rack F has shorter legs which leads to higher vertical stiffnesses in these members.

c. The gaps, simulating the spacing between adjacent structure used to calculate the virtual fluid masses, are somewhat different.

d. The location of the support feet (in the x-y plane) is somewhat different for each rack type.

Graphs showing the reaction force history in each of the legs for rack E (run 10) and for rack F (run 51) have been provided to FRC. The results are for full racks, COF=.8, and are listed as case i in the FRC table. We can see that there is a substantial difference in behavior. Note that a zero value for any foot load implies that the foot has lifted off. The feet labelled "1,2", are in the positive x half of the base, while the feet labelled "3,4" are in the negative x half of the base plane.

Up to about 4.5 seconds into the event, both racks show essentially the same foot behavior. That is, only one foot loses contact. Subsequent to this point, the behavior differs markedly. For rack F, we see considerable rocking, where 2 feet are off the ground. We

cannot ascertain whether sliding is occurring from this data, but certainly at about 13.5 seconds into the event, a third foot has a relatively low compressive load. Rack E shows most of its rocking motion between 2 seconds and 8 seconds, while the rocking of rack F is carried through the entire event. Note from the graphical results that the peak compressive load on a single foot generally correlates well in time with the occurrence of the maximum displacement. We conclude that the differences in maximum displacement are to be expected given the differences in the rack rocking behavior under the seismic event.

4. With respect to Response No. 3 of Reference 1, please confirm whether the following information is true:

   a. For a coefficient of friction of 0.8, maximum displacement always occurs at the instant three support legs are lifted off the pool floor and the rack never gets into the sliding mode.

   b. For a coefficient of friction of 0.2, the maximum displacement always occurs in the sliding mode and the rack never lifts off the pool floor.

   Also, please respond to the following question:

   For the case where three support legs are lifted off the pool floor, please indicate a typical number of time steps during which they are off the floor. It is noted that this is a completely unstable configuration in which the rack itself loses all of its resisting capacity against the motion along the horizontal directions, and the chance that the two horizontal components of seismic input motion would form a stable balance (no rotation of the rack) is remote. Please address this concern.

   <u>Response to Question #4</u>

   We cannot confirm the generality of the FRC statement. From the detailed data presented here, what we are willing to say is that the maximum displacement under a specified 3-D earthquake probably occurs when either three feet are off the ground and the rack is pivoting about the fourth foot, or when two feet are off the ground and a third foot is sliding.

   We are not clear on the meaning of the FRC statement concerning unstable configurations. It is precisely because we have recognized the possibility of pivoting about a single foot that we have gone to the detail of a 3-D model. Our model admits the possibility of gross rotation about the z axis; when and if this occurs equilibrium is still satisfied since the moment due to the seismic input is balanced by the moment due to the inertia forces.

5. With respect to Response 2 of Reference 1, Joseph Oat Corporation indicated that the fluid coupling term represents the hydrodynamic mass contribution due to motion of the plane of symmetry in antisymmetric motion. Please respond to the following questions:

   ° Since the analysis was carried out for one rack at a time, indicate whether the model has the capability to identify symmetric or antisymmetric motion. For symmetric motion, please confirm whether this plane of symmetry exists and how the gap is treated.

   ° According to the Joseph Oat Corporation approach, the planes of symmetry around a rack, in effect, will form four rigid walls around the rack and have the motion of the pool floor. Provide a technical basis to validate this approach. In reality, it is most likely that the fluid will cross these planes of symmetry, and there should be free interaction between racks.

   Response to Question #5

   The response to this question is best answered by reference to Figure 1 attached. FRC is correct in their ascertion that in general all racks will move independently. However, the extremes are as shown on the figure. In the antisymmetric mode, we can define hypothetical planes of symmetry and use Fritz's relation for virtual fluid mass based on the nominal gap. In this case, inter-rack impact potential is most severe. In the symmetric mode, no such symmetry plane can be defined, except at infinity. The fluid virtual mass expression used here is the value for an isolated rack. Note that in the symmetric mode, postulated, inter-rack impact is precluded and rack stress levels are the only consideration. Because of the infinite number of possibilities, we have chosen to study only the two extreme cases; these extremes are applied to both horizontal directions in any specific run.

B. Spent Fuel Pool Analysis

1. The Licensee stated that different finite element models were used for static and dynamic (seismic) analysis of the fuel pool slab, and that the results of the two analyses are compared to determine the dynamic load factors. The resulting small value of 0.005 (Page 8-7 [7]) of the seismic multiplying factor does not seem to confirm the conservative nature of this approach.

   ° A clarification of this comparison and the unusually small value of dynamic amplification factor is requested.

   ° Please provide information on how the effect of a 40-ft column of water was included, and on the lumping of the distributed mass to 9 master degree of freedom.

° Information is also requested describing how the effects of the wall hydrostatic and hydrodynamic pressures on the slab were considered.

Response to Question #1

a. The small value of dynamic amplification factor of .005 for the response of the pool floor itself (and the column of water) reflects the fact that the vertical seismic acceleration itself is relatively low and that the structural 9 DOF model for the floor uses 7% damping reflecting the predominate concrete structure.

b. Wall hydrostatic and hydrodynamic effects are not included in the model. To permit disregard of these effects, we have assumed simply supported boundary conditions to exist on the three edges not abutting the reactor walls. The reactor wall was assumed to be completely fixed against rotation. The weight of the walls was included, however, so that we could approximate the correct column reactions.

c. The effect of the 40 ft. head is accounted for by defining an effective density for the concrete slab. Thus, the water weight is accounted for in the dead weight ANSYS analysis, and its mass is also included in the ANSYS eigenvalue analysis. The effect of the racks and the fuel assemblies are accounted for in the dynamic foot loadings.

d. A clarification of the determination of amplification factors is given below:

   (1) The ANSYS model of the floor is used to determine the behavior of the floor under the dead weight of the floor plus the 40' of water.

   (2) The ANSYS model is also used to determine the behavior of the floor under the dead weight of the racks plus assemblies. This case is analyzed under the assumption that the loading is a concentrated loading applied at 9 interior points and a series of points around the edge of the slab. The location of the concentrated load points approximates the location of the rack feet groups (from 2,3, or 4 adjacent racks).

   (3) The output from the above static runs are floor moments and floor displacements. Of importance to the derivation of amplification factors are the displacements at the 9 chosen interior points.

(4) We next use the ANSYS model for an eigenvalue-eigenvector analysis. We choose the lateral deformation of the 9 locations mentioned above as the master degrees of freedom. We need not make any judgment on mass lumping since ANSYS does the lumping based on our specification of the degrees of freedom. With reference to p. 8-3 of the Licensing Document, the output from the ANSYS analysis, the actual seismic time history in the vertical direction, and the output of the floor load time history from our individual rack analysis, provides us with all of the information to write the 9 equations:

$$Z_n(t) + w_n^2 Z_n(t) = G_n(t) \qquad n = 1,2..9$$

where w_n are the natural frequencies of the reduced system (obtained from ANSYS) and $G_n(t)$ are known functions. Thus, by using the ANSYS model, we are able to derive the information necessary to develop a 9 DOF floor model which can be used to generate dynamic displacements of the floor slab.

(5) The Joseph Oat dynamic program DYNAHIS is used for the above 9 DOF model to study the slab dynamics first under the vertical seismic event alone, and then under the rack support loads (applied at each of the nine locations simultaneously). The output from the dynamic analysis (maximum displacement at each location) is compared with the static displacement obtained from ANSYS and an average amplification factor is obtained. These factors are then used to amplify the static floor moments which are then compared to the ACI allowables.

2. The Licensee has stated (Section 8.2.2) that the stiffness and strength of concrete are based on complete cracking of concrete. Please provide information whether the section capacities listed in Tables 8.2 and 8.3 are also based on the same assumption.

Please provide information on whether properties of the slab were calculated on an orthotropic or isotropic basis, and how the variation of reinforcement on the two faces of slab and in different directions was accounted for.

Response to Question #2

The section capacities listed in Tables 8.2, 8.3 are based on complete cracking of the concrete. The slab properties were calculated on an orthotropic basis for use in the ANSYS model. That is, the actual reinforcement in the two orthogonal directions was used to compute effective properties. The effective properties of the beam

elements were computed using the actual locations of the reinforcements as ascertained for the GPU drawings. For the ANSYS analysis, the properties based on the water side being in tension were used (as opposed to the rack side of the slab in tension).

3. The thermal loading has been based on a 21°F temperature differential across the slab depth. The thermal moment due to a temperature gradient is calculated by a formula given on page 8-6 [7] which uses the effective Young's modulus, E*, for a homogeneous slab. Please provide full details on the calculation of E* and the conservatism of using cracked sections in this calculation, if it was based on this assumption.

Response to Question #3

The calculation of the effective E* is based on a standard strength of materials approach which calculates a new neutral axis reflecting the number and location of reinforcing rods, acting in tension. A balance of forces evolves the location of the new neutral axis; a calculation of the moment curvature relation about the new neutral axis then gives

$$\frac{M}{K} = E^{*}I$$

which defines E* based on $I = H^3/12\ (1-v^2)$, H being the slab thickness.

Since the thermal moment gives compressive stresses in the rack side, the E* used is based on tension in the water side concrete. An E* effective is used to be consistent with the remainder of our analysis. Since we compute an ultimate carrying capacity based on cracked concrete, it is consistent to use the same assumption to compute the thermal moment. Use of the actual E in the thermal moment equation simply predicts that the concrete must crack, thus relieving the moment.

4. The floor slab moment capacity from Table 8.2 [7] (M_u = 48,350 lb-in/in) seems quite low in comparison to the other values. Please confirm the correct value.

Response to Question #4

The floor slab moment capacity of 48,350 in.#/in. is in fact low in this area because the imbedment length of the reinforcement in this area (water side of the slab in compression) is less than called for by ACI. Therefore, in this area, a reinforcement maximum stress of 6000 psi was used, per ACI, to account for the less than fully effective reinforcement rods.

REFERENCES

1. Joseph Oat Corporation, Preliminary Responses to FRC's List of Questions, May 7, 1984

2. Joseph Oat Corporation, Additional Responses to FRC's List of Questions, May 11, 1984

3. Technical Meeting with Joseph Oat at FRC on May 7, 1984

4. R. J. Fritz, "The Effects of Liquids on the Dynamic Motions of Immersed Solids," Journal of Engineering for Industry, Trans. ASME, February 1972, pp. 167-172

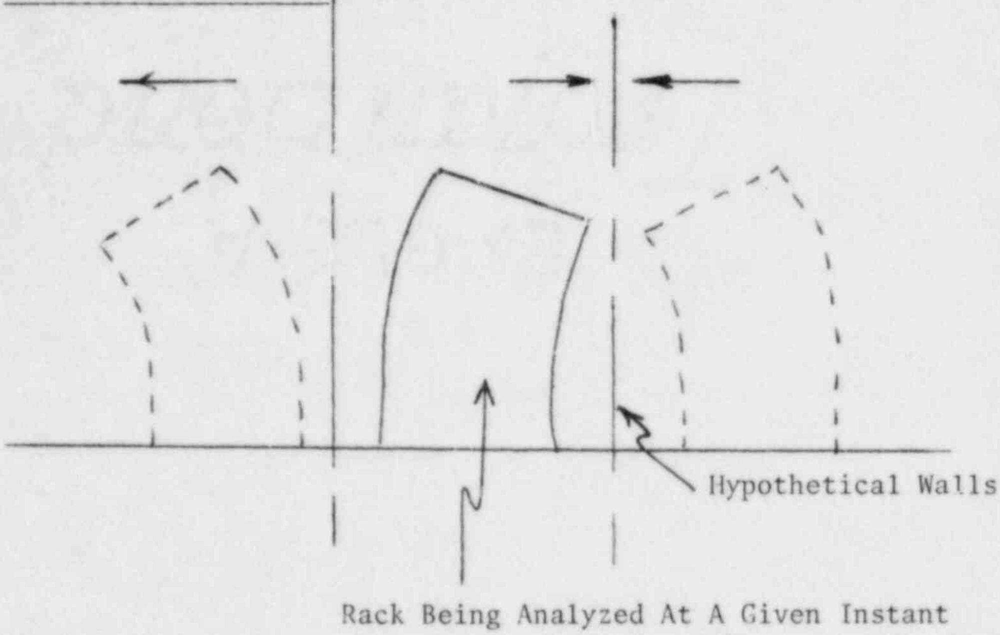
5. U.S. Nuclear Regulatory Commission, Regulatory Guide 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," February 1976

6. A. I. Soler, "Seismic Analysis of High-Density Fuel Racks for GPU Oyster Creek Nuclear Station," TM Report No. 678, April 24, 1984

7. GPU Nuclear, "Licensing Report on High-Density Spent Fuel Racks for Oyster Creek Nuclear Generating Station," August 1983

FIG. 1 RACK MODELLING

ANTI-SYMMETRIC MODEL



SYMMETRIC MODEL