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June 8, 1981

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

Subject: Dresden Station Units 2 and 3
Response to SEP Request for
Additional Information on Spent
Fuel Storage Racks
NRC Docket Nos. 50-237/249

Dear Mr. Crutchfield:

Enclosed is the Commonwealth Edison Company response to the Reference (a) request for additional information concerning Dresden Units 2 and 3 spent fuel pool storage racks.

Please address any questions you may have in this regard to this office.

One (1) signed original and thirty-nine (39) copies of this transmittal are provided for your use.

Very truly yours,

Thomas J. Rausch

Thomas J. Rausch
Nuclear Licensing Administrator
Boiling Water Reactors

Enclosure

cc: NRC Resident Inspector - Dresden

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June 5, 1981

RESPONSE TO NRC SYSTEMATIC
EVALUATION PROGRAM BRANCH'S
REQUEST FOR ADDITIONAL INFORMATION
ON DRESDEN UNITS 2 AND 3 SPENT FUEL
POOL STORAGE RACKS DATED MAY 13, 1981

QUESTION 1:

Verify that the horizontal velocity used in the rack-to-rack impact evaluation was that from the nonlinear analysis of the empty rack with the 0.2 coefficient of friction between the rack legs and pool floor. Revise Table 3.4-2 to reflect any increases in stresses in the overall rack structure under such a condition, as appropriate. Clearly state and justify all assumptions made in your analysis.

RESPONSE:

The horizontal velocity used in the rack-to-rack impact evaluation was 13.0 in/sec. This was the maximum sliding velocity of the rack structure obtained from the nonlinear analysis of the empty rack with the 0.2 coefficient of friction between the rack legs and pool floor and with the SSE impact motion in the short direction of the rack structure. The sliding of the empty rack in its short direction was judged to be most critical because of its largest ratio of hydrodynamic mass to real mass and larger aspect ratio of height over width.

The rack-to-rack and rack-to-pool wall impact during rocking would occur at the top edge of the racks (see Figure 1.1). Using an energy-balance method, and assuming elasto-plastic behavior of the tube walls, analysis of rack-to-rack and rack-to-pool wall impact was performed for a fully loaded 9 x 13 rack (which is the most critical) with an impact velocity of 13.0 in/sec. The maximum impact force (P_i) developed at the top edge of the rack during the impact was equal to 48.51 kips, and the maximum deceleration (a) of the rack was 0.4g. These values were obtained assuming that during the impacts, the change of the impact force and the deceleration were linear, and the rack would completely stop at the end of the impact (no rebound was assumed, hence conservative).

The reaction forces (R_v, R_h) induced at rack legs (shown in Figure 1.1) would vary with the actual coefficient of friction between the rack legs and pool floor. Using a friction coefficient of 0.75, these reaction forces were computed as shown in Table 1-1. As reported in the licensing report, NSC-COM-0219-R001, the stresses for various rack components given in Table 3.4-2 were obtained from analysis assuming that the rack cannot rock or slide. Based on this analysis, it was found that the elements which had the highest stresses were located at the lower part of the rack near the legs. The reaction forces at the rack legs obtained from this analysis are also listed in Table 1-1. A comparison of these two sets of values in Table 1-1 reveals that the reaction forces obtained by ignoring rocking and sliding are much higher than those if the rack is assumed a tilted position during rocking. Therefore, the maximum stresses at the critical lower part of the rack during rocking must also be less than the stress values given in Table 3.4-2. However, due to deceleration of the rack during the impact, the rack would also deform in a bending mode (shown as dotted line in Figure 1.1) which would cause additional maximum stress of 0.9 ksi in the tube wall and filler plate at mid-height of the rack. But, this additional stress is too small to be significant.

On the basis of preceding analysis, it is concluded that stresses in the overall rack structure under rack-to-rack and rack-to-pool wall impact conditions are not as critical as the stresses shown in Table 3.4-2 with the exception of tube walls near the top edge of the rack, where localized plastic deformation may be expected due to impact.

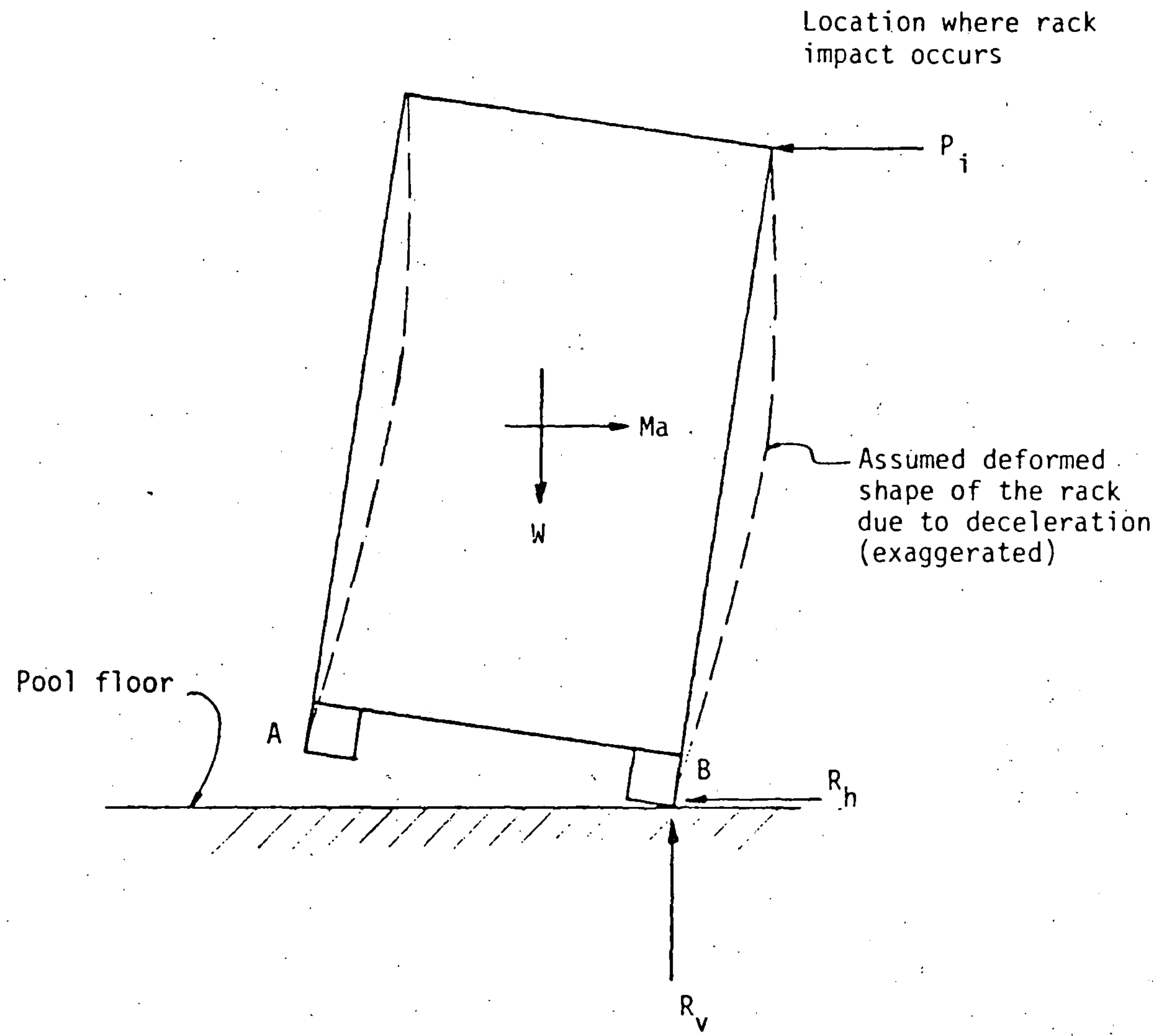

Figure 1.1 Rack in Impact Configuration

TABLE 1-1

Reaction Forces of Rack Legs

(Based on 9 x 13 rack)

LEG LOCATION	From Fixed Base Analysis of Loaded Rack		For Loaded Rack Impacted At Its Top Edge	
	R_v - vertical	R_h - Horiz.	R_v - vertical	R_h - Horiz.
CORNER LEG	179.8^k	37.1^k	29.7^k	22.4^k
MIDDLE LEG	208.9^k	51.0^k	37.2^k	28.0^k

QUESTION 2:

Provide the results of your analyses to demonstrate that the 4.625-inch minimum gap between the racks and pool wall is adequate (provides for the minimum factors of safety against impact stated in Standard Review Plan Section 3.8.5) considering both rocking and sliding of the racks under all postulated rack loading conditions and considering the variations in the coefficient of friction between the rack feet and pool floor. If such impact cannot be precluded, demonstrate that the local and gross effects on the racks and pool wall are acceptable. Revise Tables 3.4-2 and 3.5-2 to reflect any increase in loads and stresses as appropriate. Clearly state and justify all assumptions made in your analyses.

RESPONSE:

From the nonlinear analysis of an empty 9x13 rack (considering both rocking and sliding effect and assuming 0.2 coefficient of friction between the rack legs and pool floor) for a SSE motion in the short direction of the rack, the maximum lift-off at one side of the rack and relative horizontal displacement at top of the rack due to rocking (shown as D and A in Figure 2.1) were 0.56 in. and 2.01 in., respectively (See Table 2-1); the sliding distance of the rack was 1.012 in. (Refer to page 3-38, Licensing report NSC-COM-0219-R001). Thus, the total displacement at the top of the rack was computed to be 3.022 in. Hence, the factor of safety against the empty rack impacting the pool wall is (4.625 ÷ 3.022 =) 1.53. Energy-balance method was then used to compute D and A values of the rack due to rocking for various assumed loading conditions. In this approach, vertical displacement of the center of gravity (cg) of the rack was determined by equating the kinetic energy of the rack to the work required for raising the cg of the rack. The results obtained are summarized in Table 2-1. Study of the values given in Table 2-1 showed that: (a) Energy-balance method gave very conservative results when compared with results from nonlinear analysis. Thus, the factor of conservatism when energy-balance method is used may be estimated as (6.96 ÷ 2.01 =) 3.46, (b) If this factor of conservatism is taken out for the partially loaded cases, the maximum value of A would be 2.26 inches only.

Thus, the maximum total displacement at the top of the rack would be equal to (2.26 + 1.01 =) 3.27 inches. Therefore, it can be concluded that the minimum factor of safety against impact (1.1) as stated in Standard Review Plan Section 3.8.5 for SSE motion is met and impact of racks on pool walls is not anticipated. However, to demonstrate that additional margin exists, the pool walls were evaluated assuming that impact of racks may occur. The method of analysis used and assumptions made are presented in the following paragraph.

Using the method described earlier in response to Question 1 and assuming the pool wall as a rigid target, the impact force on pool wall was calculated to be 7.24k/ft. This force was treated as a line load acting in a horizontal direction normal to the pool wall at an elevation corresponding to top edge of the racks. The pool walls were then analyzed, using conventional elastic structural analytical method. The results of this analysis showed that the maximum increase in the computed moment and shear reported earlier in Table 3.5-2 would be 1.0% and 27.0%, respectively. However, even with this increase, the loads on the walls would still be less than the allowable values. Based on reasons described in the response to NRC Question 1, it can be concluded that the stresses in the rack would not exceed those reported in Table 3.4-2 when the impact on pool wall is considered. Assuming that the impact force was uniformly distributed over the contact area between top edge of the impacted rack and pool wall, the contact pressure during the impact was computed to be 3.15 ksi (based on a contact area of only 1.1 sq. in. per tube wall). The allowable compressive stress of concrete pool wall for small confined area is computed to be at least 4.76 ksi, even if no credit is taken for the aging effect of the concrete and the dynamic nature of the impacted load. Hence, unacceptable local damage of the pool wall and steel liner plate is not expected.

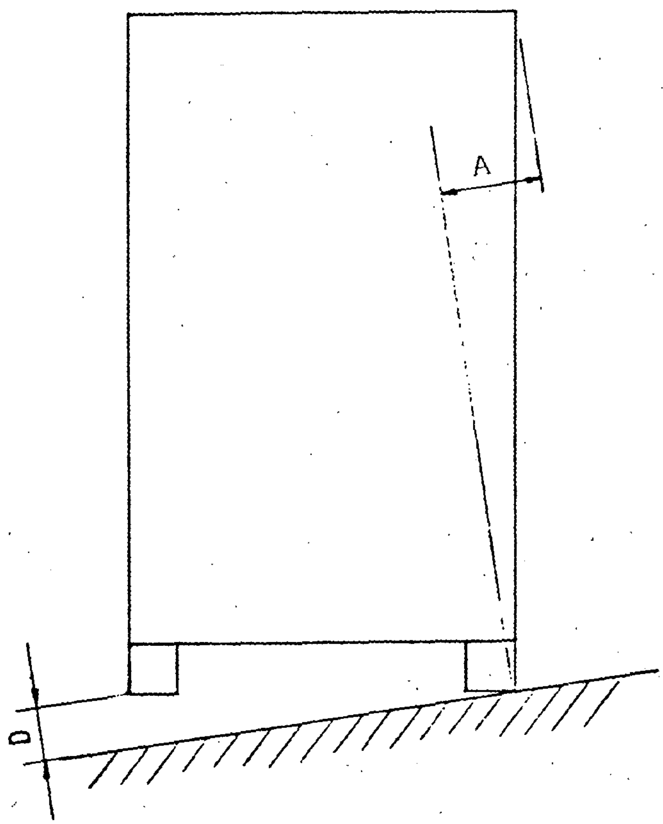

Figure 2.1 Rack in Rocking Position

Table 2-1 Lift-off and Tipping of Rack
Due to Rocking (Based on 9x13 Rack)

Postulated Rack Loading Conditions	Lift-off[1] of Rack D(in.)	Tipping of[1] Rack, A(in.)	Notes
Empty	0.56	2.01	2
	2.20	6.96	3
One row loaded	2.43	7.83	3,4
Two rows loaded	2.30	7.48	3,4
Three rows loaded	2.01	6.47	3,4
Four rows loaded	1.65	5.30	3,4
Fully loaded	0.76	2.40	3

NOTES:
1. Refer to Figure 2.1.
2. Values are obtained from nonlinear sliding analysis considering rocking effect.
3. Sliding velocity of empty rack from nonlinear analysis is used for kinetic energy calculation.
4. For partially loaded rack, row or rows of tube cells loaded are assumed on pivoted side of the rocking rack.

QUESTION 3:

Considering both rocking and sliding of the racks under all postulated rack loading conditions and considering the variations in the coefficient of friction between the rack feet and pool floor, provide the results of your analyses to demonstrate that the local and gross effects of the uplift and resulting impact of the rack lets with the pool floor are acceptable for both the rack and pool slab structures. Revise Tables 3.4-2 and 3.5-2 to reflect any increase in loads and stresses, as appropriate. Clearly state and justify all assumptions made in your analyses.

RESPONSE:

Due to rocking and sliding of the racks, the rack legs will impact the pool floor. The kinetic energy with which the rack legs impacted the pool slab was determined as follows:

Using the conservative uplift distance (0.76") for loaded rack (refer to Table 2-1 in the response to NRC Question 1), and treating the rack as an inverted pendulum when it rotates downward toward the pool slab, the impacting kinetic energy of the fully loaded rack on pool slab was computed to be 7965 in-lbs per rack. The total kinetic energy transmitted to pool slab resulting from rack impact was computed, assuming that half of all the racks within the fuel storage pool impact on the pool slab simultaneously. This assumption is judged to be very conservative from the following considerations:

- The duration of rack legs impacting at pool slab is extremely short.

- The responses of 9 x 11 racks and 9 x 13 racks are not in phase.

In the above kinetic energy calculation, the energy dissipated during the impact was neglected conservatively.

For the purpose of analyzing the pool slab subject to impact of rack legs, the pool slab was idealized as a single degree-of-freedom system with force-displacement characteristics as shown in Figure 3.1. The parameters related to this force-displacement characteristic were then determined for the pool slab and the following values were obtained:

(a) Maximum resistance (R_m), computed on the basis of ultimate moment capacity of the pool slab, was equal to 30016.8 kips.

(b) Elastic spring constant (k) was 382100 kips/in. The effect of concrete cracks was also taken into consideration.

(c) Total existing uniform load (R_e) acting on the pool slab was 8334.5 kips. Weights of water, slab, loaded racks and loads resulting from SSE are all included.

(d) Corresponding displacements of the pool slab were 0.0786 in. for δ_m and 0.0218 in. for δ_e.

Using energy-balance method (i.e. total strain-energy absorbed by the pool slab minus the strain-energy due to existing loads equals the input kinetic energy resulting from impact of rack legs), the final deflection (δ) and applied load (R) on the pool slab were obtained. These values were 12707 kips (for R) and 0.0333 in. (for δ), respectively. The shear capacity of the pool slab was investigated to determine the margin of safety against shear failure. Based on the ultimate shear capacity due to diagonal tension at a distance equal to effective depth of pool slab, the ultimate load capacity for the pool slab was computed to be 17643 kips. Therefore, the pool slab remained within the elastic range and was safe from shear failure while subjected to impact of rack legs.

Force acting on rack legs resulting from the impact was derived from the total impact force applied to the pool slab, which was equal to R minus R_e.

The maximum loads on the rack legs were 89.0 kips for the corner legs and 111.3 kips for middle leg. If these reactions are compared to the dead load, it is observed that for the impacting half of the rack load, the effective dynamic load factor (DLF) is 6.8. Such a high DLF is very conservative and was due to the fact that linear elastic assumptions were made. Comparing these computed forces with those obtained from rack analysis with a fixed end condition (see Table 1-1 in response to Question 1), it can be concluded that stresses for the overall rack structure would be less critical than values shown in Table 3.4-2 of the licensing report, NSC-COM-0219-R001.

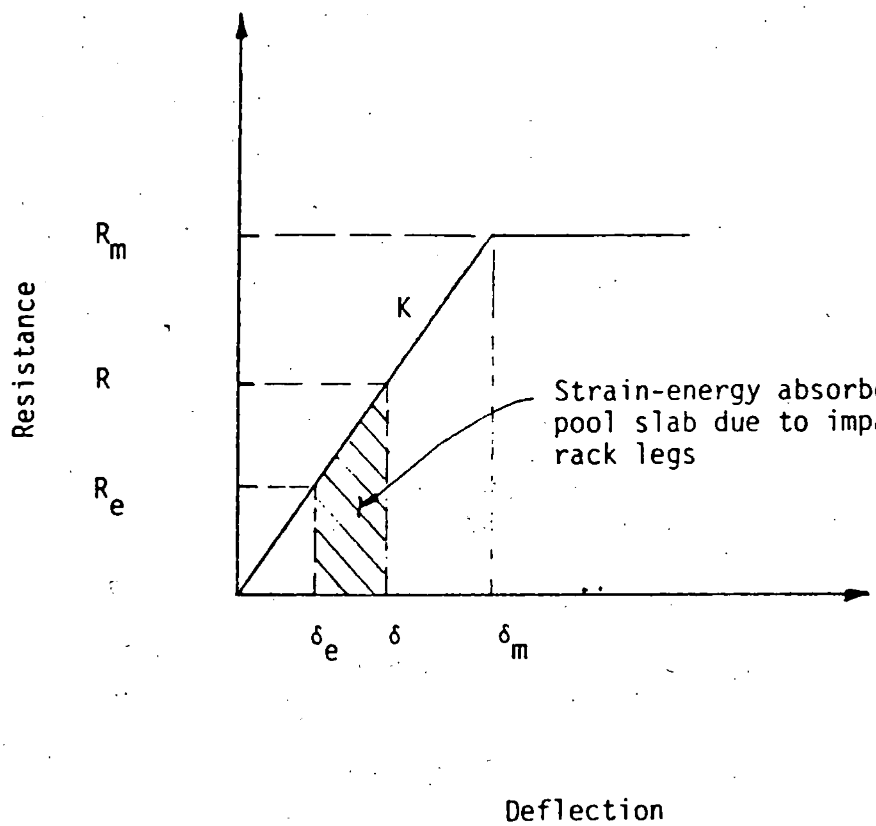

Figure 3.1 Force-Displacement Function for Pool Slab

QUESTION 4:

For all loading conditions considered in the above analyses, provide the location of the center of gravity assumed.

RESPONSE:

The location of the center of gravity (cg) of racks for various loaded condition considered in the preceding analyses are as follows (see Figure 4.1 for symbols):

(a) For empty rack and fully loaded rack, b is 28.35 inches and h is 97.13 inches.

(b) For rack loaded with one row of tube cells on pivoted side of the rack, b is 18.1 inches and h is 97.13 inches.

(c) For rack loaded with two rows of tube cells on pivoted side of the rack, b is 15.58 inches and h is 97.13 inches.

(d) For rack loaded with three rows of tube cells on pivoted side of the rack, b is 15.62 inches and h is 97.13 inches.

(e) For rack loaded with four rows of tube cells on pivoted side of the rack, b is 16.8 inches and h is 97.13 inches.

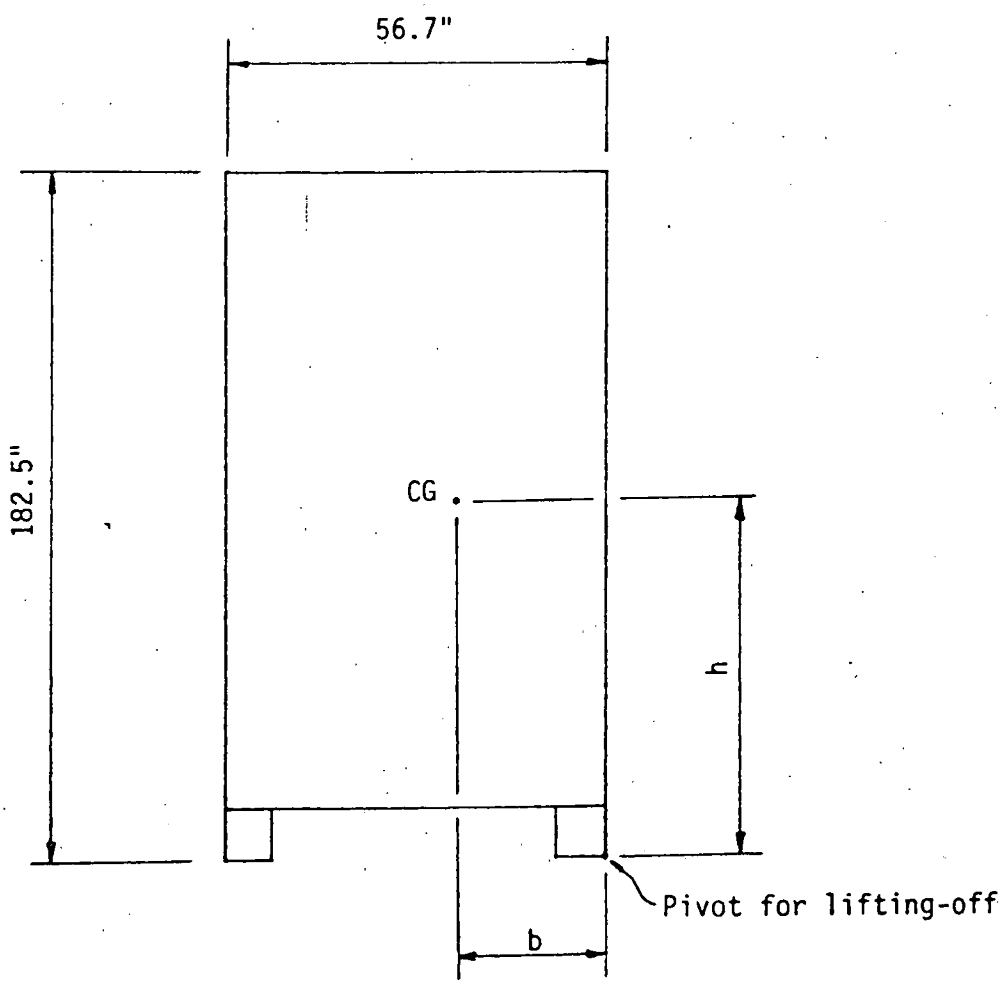

Figure 4.1 Showing Location of CG of 9x13 Rack