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April 20, 1981

Director of Nuclear Reactor Regulation
U S Nuclear Regulatory Commission
Washington, D C 20555

PRAIRIE ISLAND NUCLEAR GENERATING PLANT
Docket No. 50-282 Licensing No. DPR-42
50-306 DPR-60

Supplemental Information - License Amendment Request
Dated January 31, 1980

Attached are additional responses to questions sent to L O Mayer from R . Clark on November 5, 1980, concerning the Prairie Island NGP Fuel Storage Facility modification.

Additional responses to questions SEB 4,9 and 10 are enclosed.

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L O Mayer, PE
Manager of Nuclear Support Services

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cc: J G Keppler
NRC Resident Inspector
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Attachment to April 20, 1981 Letter

Structural Engineering Branch (SEB)

SEB-4 Additional Response

For Cases 1(a) and 1(b) (Refer to Page A-1 of Reference 1), maximum shear force was computed from the dynamic equilibrium of the beam subjected to the dropping rack (neglecting, conservatively, local crushing effects but considering the flexibility of the beam). For Case 1 (c), the maximum reaction force was computed by an energy balance method as outlined on page 5 of Reference 2. The maximum shear force is bounded by this later case, 1(c). The computed shear forces and shear stresses are compared with the permissible shear stresses in the table below. Comparison shows that the stresses are well within permissible values.

Loading Case	Wt. of Rack (kips)	Max. Shear (kips)	Shear Stress(1) (ksi)	Permissible(2) Shear Stress (ksi)
1(a)	24.8	25	7.06	27.5
1(b)	24.8	83.3	23.5	27.5
1(c)	24.8	275(3)	7.06	27.5

(1) Shear area is greater for Case 1 (c) than for Cases 1 (a) and 1 (b)

(2) Allowable shear stress for normal operating design loads per AISC code is 0.4 fy. For accident condition, USNRC Standard Review Plan permits the use of allowable stress values which are at least 1.6 times the basic allowable values. Thus, direct use of USNRC SRP criteria would permit an allowable shear stress value of 0.64 fy. However, from shear failure theory, the maximum shear stress should not exceed 0.57 fy. A value of 0.55 fy was used in order to be consistent with the original pool cover design.

(3) Upper bound shear force value.

SEB-9 Additional Response

The effect on the overall fuel rack structure of rack to rack impact during a seismic event has been determined. The methods used for the evaluation and the results are presented in the following sections:

1.0 INTRODUCTION

During a postulated seismic event, free-standing racks may slide towards each other and impact. This impact will cause additional stresses in the rack. This analysis is to evaluate the adequacy of the rack structure to withstand the stresses resulting from this impact load.

Using a minimum friction coefficient of 0.2, the maximum sliding velocity of the rack was computed to be 4.43 inches/sec. This corresponds to a maximum kinetic energy of 3759 in-lb. This energy is less than the energy required to tilt and rotate the rack to cause rack-to-rack impact at the top. Hence, the impact would occur only at the level of the base assembly.

2.0 LOADING CASES

Two impact loading cases were considered:

a) Load Case 1, in which two adjacent racks slide towards each other and impact as shown in Figure 9-1.

b) Load Case 2, in which two racks on two sides of a central rack slide towards the central rack and impact on it as shown in Figure 9-2.

3.0 ANALYSIS AND EVALUATION

The analysis and evaluation of the rack structure subjected to the impact loads was performed by computing an upperbound impact load and comparing the inertial loads resulting from the impact with those obtained from earlier seismic response analyses of the sliding rack.

3.1 Load Case 1: Two Racks Impacting on Each Other

When Racks A and B (Figure 9-1) slide and move towards each other their motion is resisted by friction force F_o. Let the maximum seismic force during this motion before the impact be denoted by S. Let P be the external force and acting at the base assembly level and I be the equal and opposite inertial load resulting from this impact. (The two racks are at rest at the instant these forces are at their peak values, hence $F_f = 0$). Thus, just before and during the impact, the forces acting on the rack are:

S = Maximum seismic inertial load before the impact
F_o = Friction force before the impact resisting seismic inertia load.
I = Inertial load due to the impact = P
F_f = Friction force when P and I are at their peak = 0
P = Impact force = I

In addition, the dead loads are also present. It is noted here that S and I are not concentrated loads, rather these are inertial loads distributed all over the rack body.

The object of the present evaluation is to determine the maximum stresses in the structural members of the sliding rack when the rack is subjected to the above loads-before and during the impact.

The racks are sliding before the impact; hence the seismic inertia load S is smaller than or equal to the limiting friction force, F_o. Stresses in all members of the sliding rack were computed assuming that S is equal to its upperbound value of F_o. This stress determination utilized the results of the earlier seismic response analysis in which the rack was modeled in detail but was assumed not free to slide (i.e. equivalent to infinite friction). This was done by scaling the stresses from the detailed finite element model in proportion to the limiting seismic inertia load of $S = F_o$. The computed stresses are shown in Table 9-1 in terms of stress index SI_s, meaning stress index for the sliding rack, before impact.

At the instant of the impact, the impact points of the two base assemblies will be at rest, but the rest of the body of the two racks will have a tendency to move close to each other, causing these to bend in the direction of motion. This bending is the time-dependent response I resulting from and equal to the impact force P.

Since the impact is postulated from a seismic event which produces to-and-fro sliding motion of the rack, it is conceivable that, before the rack body could respond and dissipate all the kinetic energy in bending and straining the racks, the racks would start moving away from each other. However, to maximize the stresses resulting from I = P, it was conservatively assumed that no such motion would occur before all the kinetic energy is spent in bending and straining the racks.

Impact load P (=I) was calculated using energy balance method. This was done by equating strain energy of the rack bending in response to the impact inertia load I with the kinetic energy of motion. The stresses in the rack body were then computed for inertia load I by scaling the stresses computed earlier for seismic response analysis in proportion to the magnitude of I.. The resulting stresses are shown in Table 9-1 in terms of stress index values and denoted as SI_i, meaning stress index values for impact load.

3.2 Load Case 2: Two Racks Impacting on a Central Rack

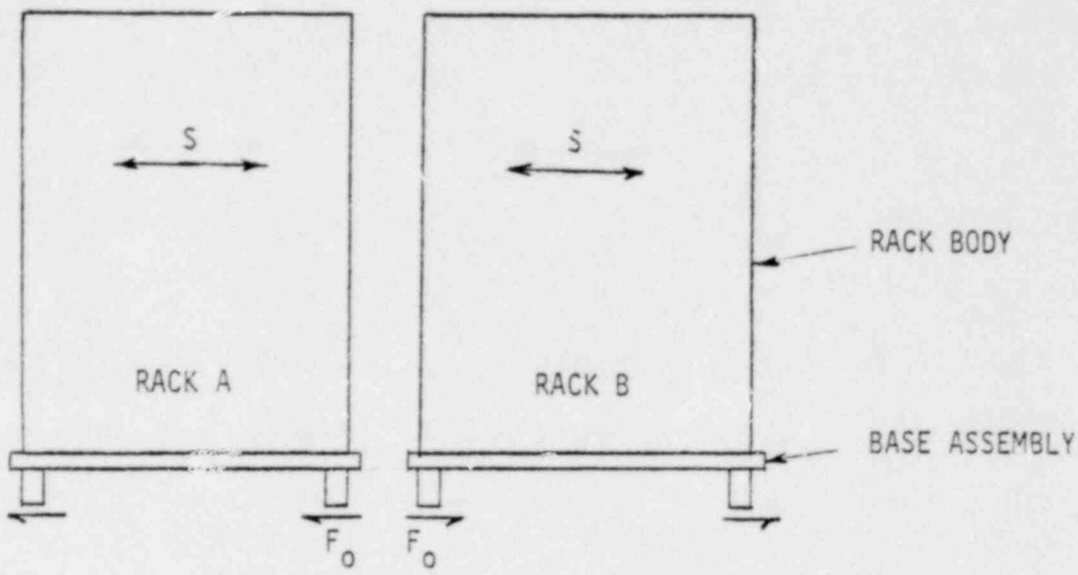
The scenario for this load case assumes that Racks A and B (Figure 9-2) impact on the central Rack C simultaneously at an instant when the sliding velocities of both of the impacting racks (Racks A and B) are at their peaks. In addition, it also assumes that the central Rack C is at rest the instant of the impact. A comparison of Load Case 2 with Load Case 1 reveals that, the forces acting on Racks A and B just before and during the impact are identical for the two load cases. For Load Case 2, the impact force P will cause compressive stresses in Rack C base assembly. Similar smaller compressive stresses will also occur in Racks A and B base assemblies and are bounded by Rack C's base assembly compressive stresses. The compressive stresses in Rack C's base assembly were computed and added to the other stresses in Rack A & B's base assembly. The result is shown in Table 9-1.

3.3 Stress Evaluation

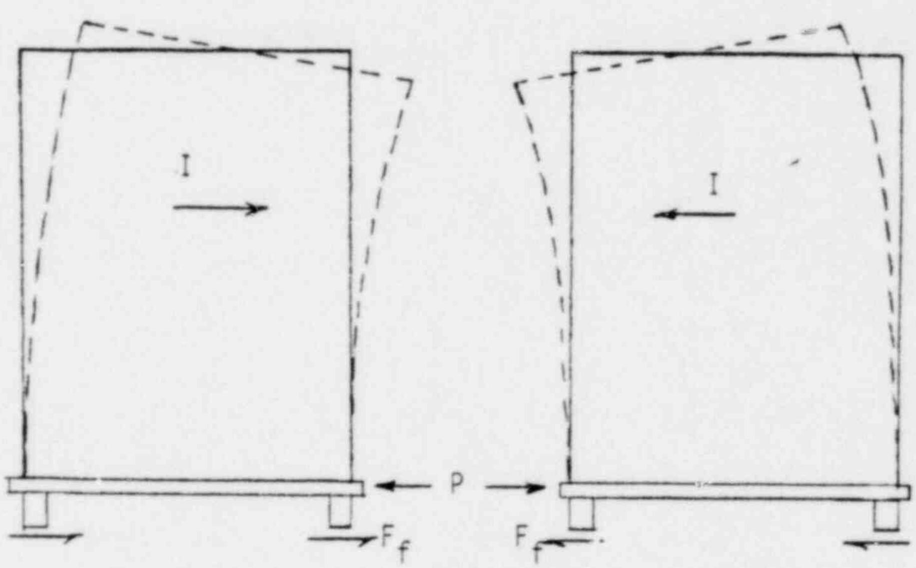
Stresses in the rack structural members resulting from seismic sliding motion and impact were computed. Dead load stresses are very small, and have been conservatively included in the stress index values for computational efficiency. Since the impact was assumed to occur when the velocity is maximum it is very likely that the usual seismic stresses are very low at that time because, seismic stress at any instant is proportional to the relative acceleration which is minimum when the velocity is maximum. Also, significant probability exists that the direction of the usual seismic stresses in the critical locations is opposite to those due to the impact. For these reasons, it was considered appropriate to combine the seismic stresses with the impact stresses by SRSS method. Theses combined stresses are also shown in Table 9-1 in terms of combined stress index value denoted as SI_{comb}.

4.0 CONCLUSION

Inspection of the stress index values shown in Table 9-1 shows that the additional stresses resulting from an event of rack-to-rack impact, when combined with those from other loads, are within the permissible stress index value of 1.0.

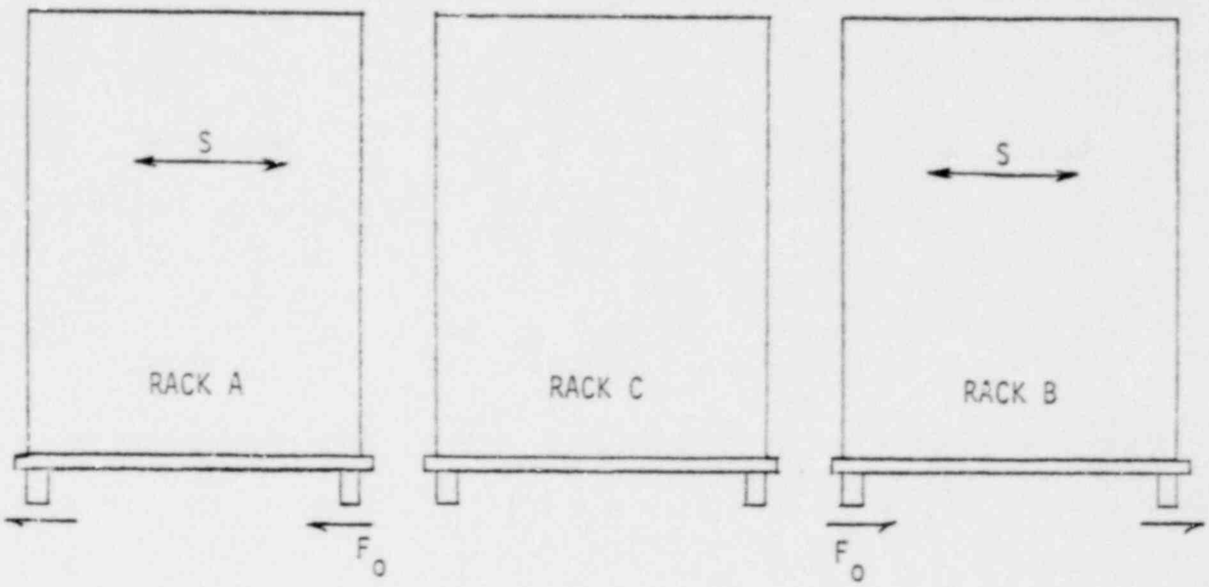


LOAD CASE 1, BEFORE IMPACT

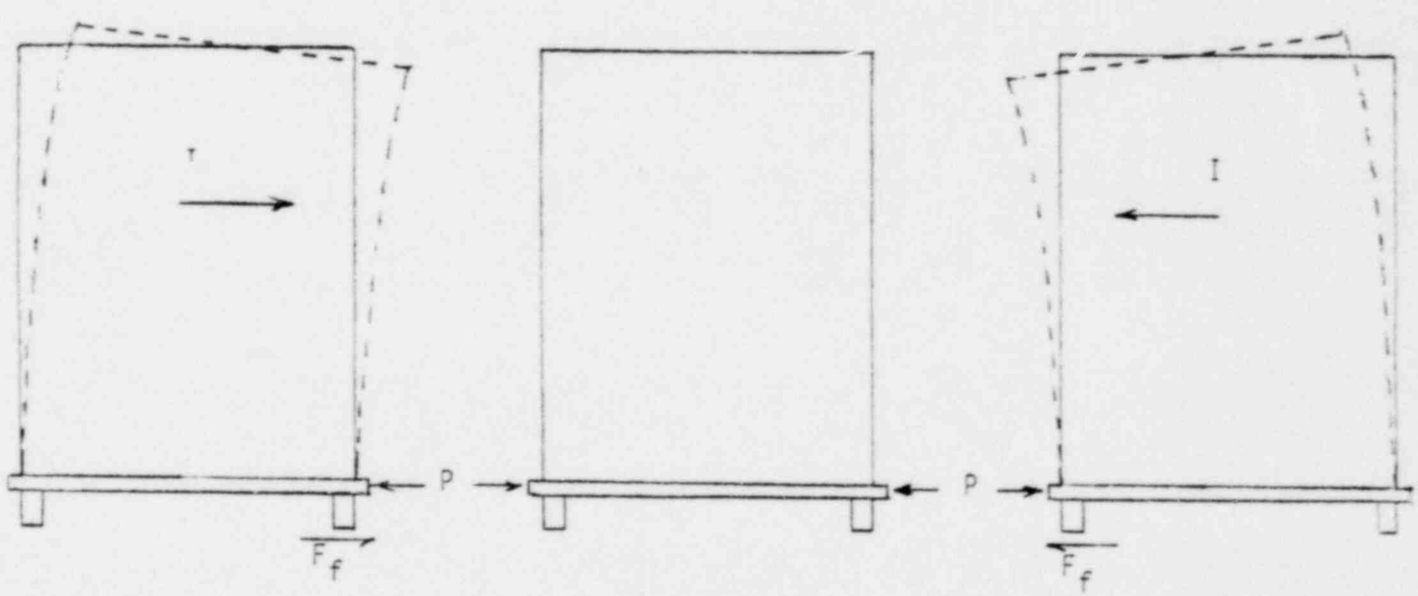


LOAD CASE 1, DURING IMPACT

FIGURE 9-1 LOAD CASE 1



LOAD CASE 2, BEFORE IMPACT



LOAD CASE 2, DURING IMPACT

FIGURE 9-2 LOAD CASE 2

TABLE 9-1

MAXIMUM STRESSES IN A SLIDING RACK
DUE TO SEISMIC, IMPACT AND DEAD LOAD

RACK COMPONENT	SI_i	SI_s	SI_{comb}
Upper Grid	0.642	0.155	0.66
Lower Grid	0.42	0.102	0.432
Vertical Corner L Beam	0.173	0.042	0.178
Cross Brace	0.349	0.085	0.359
Middle Strap	0.548	0.133	0.564
Fuel Tube	0.200	0.048	0.205
Base Assembly(1) Box Beams	0.451	0.090	0.46
Rack Leg	0.249	0.060	0.256
Floor Leg Connection	0.311	0.075	0.32

(1) This includes the additional compressive stresses in rack "C" (load case 2) resulting from the two opposing impact forces.

SEB-10 Additional Response

The effects on the fuel pool liner and concrete structure resulting from a fuel assembly drop are described in the following sections:

1.0 INTRODUCTION

After the proposed modification to the storage system in the spent fuel pool, practically the whole pool-2 floor will be covered by the racks. There will be only a narrow strip of empty space all around the pool perimeter where a spent fuel assembly may accidentally be dropped. Considering the size of that strip (about 6 to 9 inches wide), the height of the drop, and the size of the fuel assembly, such an event is judged to have a very low probability of occurrence. However, assuming such an event to be credible, an evaluation is performed here, the objective of which is to estimate the extent of structural damage that may conceivable result from such drop and which may result in an unacceptable leakage problem.

In pool-1, if the four racks in the cask pad area are removed (which would be necessary to install a shipping cask) a section of pool floor is clear, and a fuel assembly could be dropped in this area. Most of the pool floor in this region is covered by the cask wear plate. The analysis described herein ignores the presence of that wear plate.

2.0 METHOD OF EVALUATION

The consequences of fuel assembly drop on the pool floor can be qualitatively described as follows:

a) The fuel assembly would probably perforate the liner and penetrate the concrete partially.

b) Since the pool floor slab consists of a six-foot thick concrete slab, it would be very stiff - - as compared to the fuel assembly which is long and slender. As a result, the fuel assembly is likely to buckle and collapse without causing structural damage to the floor.

Two analyses were performed to evaluate the above hypothesis quantitatively: One analysis to determine the depth of penetration in the concrete floor slab, and the other analysis to determine the load resulting from the drop and the capacity of the floor slab to withstand it.

2.1 DETERMINATION OF PENETRATION DEPTH

The impact velocity of the fuel assembly was computed to be 50.8 ft/second. This assumes no fluid drag force and a drop height of 40 feet (conservative). Using the Ballastic Research Lab Formula for penetration and assuming a rigid fuel assembly, it was determined that the fuel assembly will performate the liner (but not the cask wear plate), and that its residual velocity would be 28.6 ft/second. The depth of concrete penetration was computed using the Modified Petri Formula, and was found to be 0.17 inch.

2.2 IMPACT LOAD ON POOL FLOOR

The impact load on the pool floor will be limited by the buckling or compressive load capacity of the fuel assembly, since the floor slab will not bend significantly. The vertical structural members of the fuel assembly which would be loaded during impact are the guide tubes. These guide tubes are laterally braced by grids at about every 19 inches. The buckling load for each of the 17 guide tubes was computed to be 942 lbs., which is very close to it's compressive strength of 990 lbs. Thus, the maximum load that the fuel assembly can exert on the floor is limited to 990 X17 = 16.8 kips. Since this is an approximate analysis, an uncertaintity factor of 2 was used to compute the impact force. Thus, the postulated impact force is about 34 kips.

3.0 EVALUATION AND CONCLUSION

The results of the analysis show that the liner is likely to be perforated, but the depth of penetration in concrete will be less than 0.25 inch. The computed impact load of 34 kips, when combined with the dead load shear of 69 kips per foot is well within the minimum shear capacity of 273 kips per foot.

It is thus concluded that in the unlikely event of dropping a fuel assembly directly on the pool floor, the structural damage to the floor will be too small to result in any leakage through the concrete.

REFERENCES

1. Letter from L O Mayer to the Director of Nuclear Reactor Regulation dated February 3, 1981 - Supplemental Information - License Amendment Request dated January 31, 1980.

2. Letter from L O Mayer to the Director of Nuclear Reactor Regulation dated March 31, 1980 - Supplemental Information - License Amendment Request dated March 31, 1981.