Enclosure 1

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	Docket Nos. 50-237
	)	
COMMONWEALTH EDISON COMPANY	)	50-249
	)	
(Dresden Station, Units 2 and 3)	)	(Spent Fuel Pool Modifications)

AFFIDAVIT OF KENNETH S. HERRING
EVALUATING COMMONWEALTH EDISON'S PROPOSAL
TO INSTALL FIVE HIGH DENSITY FUEL STORAGE RACKS

I, Kenneth S. Herring, do state as follows:

I am employed by the United States Nuclear Regulatory Commission as a Senior Mechanical Engineer in the Systematic Evaluation Program Branch of the Division of Licensing. A statement of my professional qualifications is attached to this affidavit.

I have reviewed the information contained in the licensee's submittal, dated August 10, 1981, regarding the structural adequacy of the new racks and pool structures to resist the SSE loads resulting from the installation of five new high density spent fuel racks in addition to the existing spent fuel racks, minus the 13 existing racks which must be removed to accommodate the new racks.

EVALUATION

The inital licensee nonlinear analysis evaluations contained in their June 8, 1981 submittal, derived 0.76 in. of uplift of the full rack leg. Later analyses, performed using a slightly different time history in a nonlinear rack analysis which would be expected to yield results comparable to those obtained initially

given the dynamic characteristics of the racks, indicated about a 50% increase in uplift above that initially determined. In addition, the "factor of conservatism" given by the licensee for their determination of this value has not been adequately substantiated, considering variations in time histories as input to the nonlinear analyses of racks and variations in the coefficient of friction between the racks and pool floor. Therefore, uncertainty exists regarding the quantification of rack displacements and impact energies derived from the nonlinear analyses which have been performed by the licensee to date.

Previous licensee submittals (see June 8, 1981 letter from the licensee to D. Crutchfield, NRC) have demonstrated the capability of the new racks to withstand the impact of adjacent racks and their own impact with the pool wall with substantial margin. Therefore, remaining questions regarding the quantification of uplift and the effects of resulting impacts should not detrimentally affect conclusions regarding the adequacy of the racks in this respect.

The gaps between the pool walls and existing spent fuel racks, and the proposed five new racks, per an August 11, 1981 telephone conversation with the licensee, are about 32 in. between the west wall and new racks, 28 in. between the new and existing racks, and 8 in. between the north wall and new racks. The gaps between the west wall and existing racks should be sufficient to prelude impact with the new racks, considering the potential uncertainties in the displacements, as discussed above. However, the approximate 8 inch clearance between the north wall and new racks, while

sufficient to preclude impact for the fully loaded racks, may not be sufficient to preclude impact for cases where the racks would be partially full of fuel, given the displacements for such cases given in the June 8, 1981 licensee submittal. However, the licensee's evaluation of the full racks impacting wall, contained in this same submittal, demonstrated that the wall could resist impact loads, derived using the energy determined from initial analyses with margin. Since impact energies for the partially loaded cases where impact cannot be precluded will be less than assumed, and a substantial portion of this energy would be absorbed in displacing the racks the several inches needed for impact to occur, the wall capacity should not be exceeded given proper consideration of the uncertainties in the rack displacements.

In addition, the licensee has not demonstrated the adequacy of applying the "Housner Method", Method 1 in its August 10, 1981 submittal, to flexible systems such as the racks and pool structure. Also, the multiple impacts of racks on the slab during a seismic event, and potential further amplification of the responses computed using the "Housner Method" has not been addressed by the licensee. However, in their August 10, 1981 submittal, by applying Method II for the calculation of responses which considered 100% of the energy resulting from 1 inch of uplift of the five fully loaded rack legs to be absorbed by the rack/pool structural system, the licensee demonstrated that margin exists in the racks and pool structure. Proper consideration of the uncertainties in rack displacements and the calculation of the resulting impact loads should not detrimentally affect the conclusions drawn from this analysis regarding the structural integrity of the new racks and pool structures.

Therefore, given:

1. The margins in the new racks and pool structures indicated by the licensee analyses performed to date for the five rack installation; and

2. Consideration of the fact that the Systematic Evaluation Program derived site specific ground spectra have a substantially lower peak ground acceleration than that assumed in the licensee's analyses;

I feel that there is reasonable assurance that the structural integrity of the new racks and pool structure will be maintained given the occurrence of a postulated earthquake at the Dresden 2 and 3 sites for the installation of five new high density spent fuel storage racks.

CONCLUSION

Based on my review of Commonwealth Edison's five rack structural analysis, I conclude that five new spent fuel racks may be safely added to the spent fuel pool at this time. Our review of the 33 rack installation will be completed upon receipt of the additional information requested from Commonwealth Edison. I swear that the foregoing is true and correct to the best of my knowledge and belief.

Kenneth S. Herring

Kenneth S. Herring
Systematic Evaluation Program Branch
Division of Licensing

Sworn to before me this 13th day of August, 1981

Patricia J. Noonan

Notary Public

My Commission Expires: July 1, 1982

PROFESSIONAL QUALIFICATIONS
OF
KENNETH S. HERRING

EDUCATION:

State University of New York at Stony Brook - Bachelor of Engineering - May 1973:

University of Illinois at Urbana-Champaign - Master of Science in Civil Engineering (Structures) - August 1974:

Continuing Education Courses: Legal Aspects of Safety, Westinghouse PWR System Fundamentals, BWR-4 System Fundamentals, Teledyne ASME Code Seminar on Code Design of Nuclear Components, MARC Computer Code Users' Course.

ENGINEER-IN-TRAINING: New Jersey

TECHNICAL SOCIETIES:

American Society of Mechanical Engineers - Associate Member - May 1973 to September 1975.

American Society of Civil Engineers - Associate Member - April 1974 to Present.

ASME BOILER AND PRESSURE VESSEL CODE COMMITTEES:

Section XI - Subgroup on Containment - Member - January 1979 to Present.

AWARDS:

U. S. Nuclear Regulatory Commission - High Quality Award - January 1979.
U. S. Nuclear Regulatory Commission - High Quality Award - June 1980.

EXPERIENCE:

January 1977 to Present

U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Senior Mechanical Engineer (2/81 to Present)
Systematic Evaluation Program Branch
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Office of Nuclear Reactor Regulation

Senior Structural Engineer (4/80 to 2/81)
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Senior Structural Engineer (10/79 to 4/80)
Engineering Branch
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Structural Dynamicist (1/79 to 10/79)
Engineering Branch
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Office of Nuclear Reactor Regulation

Applied Mechanics Engineer (1/77 to 1/79)
Engineering Branch
Division of Operating Reactors
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Responsible for managing, coordinating, and conducting the review, the analyses, and the evaluation of structural and mechanical aspects related to safety issues for reactor facilities licensed for power operation, and test reactor facilities, including the formulation of regulations and safety criteria. An emphasis is placed on seismic, impact and other dynamic loading considerations, in addition to static loading considerations, and linear and nonlinear, concrete, masonry and steel behavior.

Responsible for managing and coordinating nuclear plant system reviews.

Responsible for managing and coordinating various outside technical assistance contractor programs related to nuclear power plant safety issues.

Serve as an expert witness in public hearing proceedings.

August 1974 to December 1976	Stone and Webster Engineering Corporation 3 Executive Campus Cherry Hill, New Jersey Structural Engineer in the Structural Mechanics Group. Responsible for conducting static and dynamic, including seismic, finite element analyses and design of structures in nuclear power generation facilities. Responsible for maintaining the Structural Mechanics computer facilities at CHOC. Fortran IV programming experience.
August 1973 to August 1974	University of Illinois Department of Civil Engineering Urbana, Illinois 61801 Research Assistant Responsible for conducting an investigation into the material properties of fiber reinforced concrete using quick-setting cements for the Department of Transportation, Federal Railroad Administration.

PUBLICATIONS:

U. S. Nuclear Regulatory Commission Report, NUREG-0766 entitled, "Reconnaissance Report: Effects of November 8, 1980 Earthquake on Humboldt Bay and Eureka, California Area," March 1981.

Department of Transportation, Federal Railroad Administration Report, FRA-ORDD 75-7 entitled, "Concrete for Tunnel Liners: Behavior of Steel Fiber Reinforced Concrete Under Combined Loads," August 1974.

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Enclosure 2

August 10, 1981

Mr. Dennis M. Crutchfield, Chief
Operating Reactors Branch 5
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Subject: Dresden Station Units 2 and 3
Seismic Analysis for Installation
of Five and Ten High Density Fuel
Storage Racks
NRC Docket Nos. 50-237/249

Reference (a): "Licensing Report Dresden Nuclear
Power Plant Units 2 and 3
Spent Fuel Rack Modification",
Rev. 5, dated 1-19-81.

Dear Mr. Crutchfield:

Enclosed for your review are the results of the seismic analyses performed for the installation of five and ten new high density spent fuel storage racks of the type described in Reference (a).

These analyses were performed using the conservative assumptions discussed in the conference call between Commonwealth Edison Company, Quadrex Corp., and the NRC staff on July 24, 1981.

As discussed previously, interim installation of five or ten new fuel storage racks in the Dresden 3 fuel pool will preclude the need to transfer fuel in fuel shipping casks from the Dresden 3 fuel pool to the Dresden 2 fuel pool in order to support the January 1982 Dresden 3 refueling outage. The fuel transfers would be necessary to maintain the ability to unload the core to facilitate NRC required modifications to the feedwater spargers. The five or ten new racks will provide the additional spaces necessary to accomplish core unloading without fuel transfers.

Please address any questions concerning this matter to this office.

Dure
8108140224

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

Very truly yours,

T. J. Rausch

T. J. Rausch
Nuclear Licensing Administrator
Boiling Water Reactors

cc: Region III Inspector, Dresden
Mr. John Wolfe, Esq.
Dr. Linda W. Little
Dr. Forest J. Renwick
Ms. Mary Jo Murray

lm

2389N

Dresden Nuclear Station

Evaluation of Spent Fuel Pool and Racks

for Five and Ten Racks Impacting on Pool Floor

1.0 INTRODUCTION

It has been proposed that existing spent fuel racks at the north end of the Dresden 3 pool be replaced with 5 or 10 new high-density fuel racks. The scope of the present evaluation is to determine whether the spent fuel pool floor and walls can withstand the additional loads resulting from rocking of the racks during a postulated safe shutdown earthquake. In the absence of nonlinear rocking and sliding analysis for the loaded rack, the magnitude of the maximum uplift was computed earlier using an energy-balance method based on the maximum sliding velocity of an empty rack. This uplift value was computed to be 0.76 inch. However, for the present evaluation to be conservative, this uplift value was arbitrarily increased to 1.0 inch. The existing racks are bolted to the floor, hence, no uplift of these racks was considered.

2.0 ANALYSIS AND EVALUATION OF POOL SLAB

An energy-balance method of analysis was used for evaluating the loads on the pool floor. To determine an upperbound of this load, it was assumed that the entire energy resulting from impact will be absorbed by the strain energy of the pool floor and the impacting rack in a single impact. In other words, the energy that would be left in the rack which would cause it to rebound and rock was not subtracted to compute the energy that needs to be absorbed in the pool floor.

2.1 Input Kinetic Energy

For 1-inch uplift, the angular velocity of the rack was computed to be 0.177 radian per sec. This was determined by equating the restoring moment of the tilted rack to the product of its moment of inertia and angular acceleration and solving the equation of motion. This method of computing the angular velocity, outlined in Reference 1, uses the rack geometry, mass and the uplift value and assumes realistically that the rack behaves as a rigid body while dropping from the tilted position. The vertical impact velocity at the uplifted end of the rack, computed from this angular velocity, was 10.02 in/sec. The velocity of different parts would vary between this maximum value and zero (at the pivoting end). The kinetic energy of impact was calculated using linear velocity distribution. This method of characterizing the motion of a tilted structure has been experimentally verified in Reference 2 and is considered a more accurate representation of the actual phenomena as compared to the alternate method described in the next paragraph. This method is designated as Method I.

An alternative method (Method II) of computing the kinetic energy was also investigated. In this alternative method, the rack was idealized as a beam having a length equal to the width of the rack. The depth of the beam was ignored. The tip of the beam was assumed to be uplifted by 1.0 inch and allowed to be dropped. The velocity of the rack at the uplifted end was computed assuming a free-fall, i.e., velocity equal to $\sqrt{2gh}$ where g is acceleration due to gravity and h is the drop height (i.e., 1.0 inch). Using the uplifted-end velocity (27.8 in/sec.) and a linear velocity distribution, the impact energy was computed by integrating

over the length of the idealized beam. The peak velocity computed this way assumed a free drop and is not an accurate representation since the rack does not fall freely, rather rotates pivoting about the other leg. Thus, the phenomenon assumed in this Method II analysis is not considered a true representation of the actual situation, even though, for the purpose of comparison, the pool structures have been evaluated using the input kinetic energy values obtained by both methods.

2.2 Energy Absorbing Characteristics of the Pool Slab

Impact energy is dissipated in the pool slab in three ways:

a) Inertia of the pool slab to movement

b) Strain energy resulting from the local compression of the concrete underneath the rack legs

c) Strain energy resulting from the overall behavior of the floor slab acting as a plate

Since the weight of the supporting pool floor target mass is relatively high, some of the applied energy will be absorbed because of the inertia of the member to movement. The target mass was assumed to be equal to the mass of that portion of concrete pool slab which is contained in the volume bounded by 45-degree inclined planes from the edge of the rack leg and the bottom surface of the pool slab (Reference 3) as shown in Figure 2-1. For computing the maximum rack load and the concrete bearing stress, energy absorption due to inertia of the target mass was not considered.

Strain energy resulting from the local compression of the concrete underneath the rack leg was computed assuming a nonlinear distribution of compressive stress under the rack leg as shown in Figure 2-2. The compressive stress at the interface is equal to the bearing stress and, at the bottom surface, this stress is zero. The variation of this stress across the depth was assumed to be parabolic in computing the equivalent linear spring to represent the local energy absorbing characteristics of the slab.

Strain energy resulting from the overall behavior of the floor slab acting as a plate would depend upon the location of the impact. For the five-rack impact, two such locations were considered:

a) Location A, corresponding to rack legs close to the north wall of the pool, and

b) Location B, corresponding to rack legs away from the wall.

Location A is only 9 inches from the wall. Hence, it was assumed that the energy dissipation resulting from the overall plate-type behavior of the floor would be small and negligible when the rack legs impact in Location A. Location B is about 5.5 ft. from the wall. Energy absorption characteristics for impact at this location were represented by a linear spring, properties of which were computed from the bending behavior of the slab. The moment of inertia of the slab cross-section was computed using the formula in ASCE Standard Manual 58 (Reference 4). This formula provided a moment of inertia larger than that obtained from ACI-318-77 code (Reference 5) (corresponding to actual moment resulting from a floor load which produces shear equal to the shear capacity of the slab). Thus, the use of this formula is considered conservative.

For the ten rack case, the racks impact along two parallel lines: Legs nearest the north wall and legs farthest from the north wall (two parallel rows of 5 racks each). The analysis utilized the same methodology described above and the results reported in Table 2-2a are for impact of the legs nearest the wall which generates the highest lead at the support and at a distance, d, from the support.

2.3 Energy Absorbing Characteristics of the Rack

When the racks impact on the pool floor, a part of the impact energy will be absorbed in the form of strain energy of the rack. This energy absorption characteristic was represented by a linear spring constant. This linear spring constant was derived using the vertical stiffness and deflection characteristics of the detailed finite element model of the rack which was originally used for dead load stress analysis.

2.4 Analysis Results and Evaluation of Pool Slab

Using the energy balance method described above, the rack impact load on the pool floor was computed. The computed dynamic amplification factors are listed in Table 2-1. Combining the rack impact load with the dead loads and hydrostatic loads and vertical seismic loads, and assuming very conservatively that all the racks impact simultaneously, the shear load in the pool slab was computed and compared with the allowable values. These are listed in Tables 2-2 and 2-2a, for five and ten racks, respectively. Shear loads were computed at the floor support location (Location A) as well as at a distance d (equal to depth of the slab) from the support (Location B). Comparison of the computed shear loads with the allowable values shows that the pool slab can adequately withstand the total loads, including the impact loads resulting from 1-inch uplift of the loaded racks.

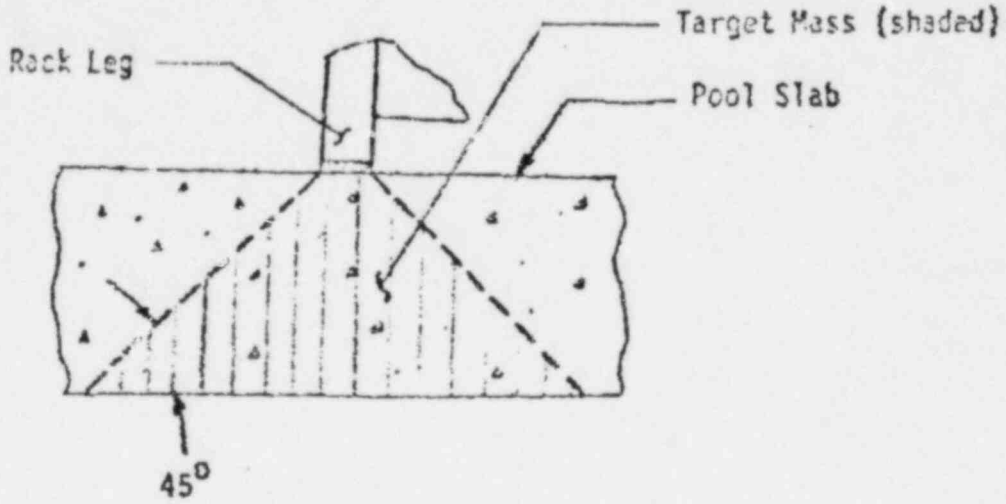

FIGURE 2-1 TARGET MASS UNDER RACK LEG

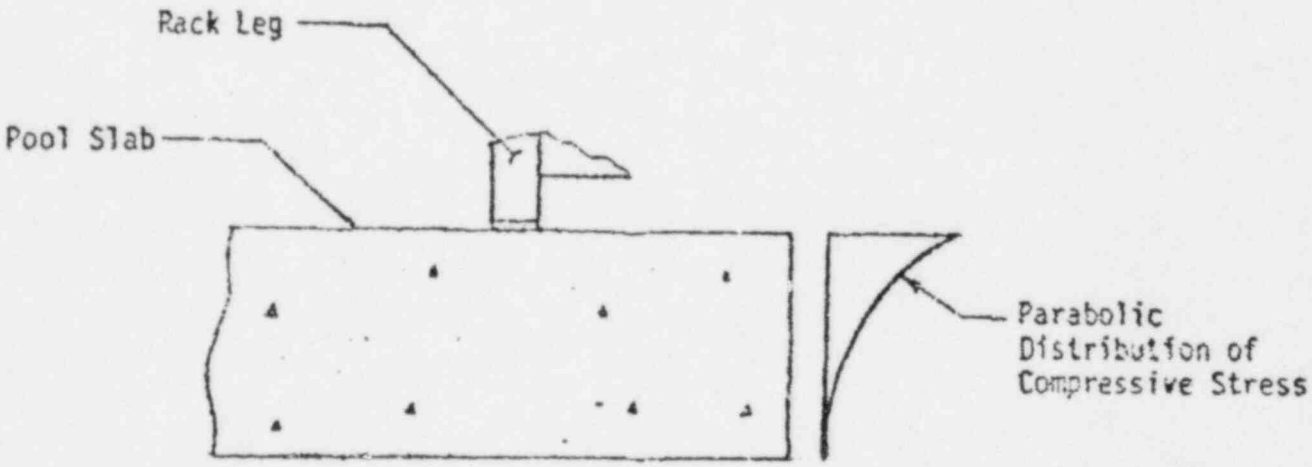

FIGURE 2-2 ASSUMED DISTRIBUTION OF COMPRESSIVE STRESS UNDER RACK LEG

TABLE 2-1

DYNAMIC AMPLIFICATION FACTORS[1]
RESULTING FROM RACK IMPACT

	Location of Drop	
	A	B
Method I	6.5	6.1
Method II	18.1	17.1

NOTE[1]: Dynamic Amplification factor is defined as the ratio of rack leg force during rack impact to the rack leg force due to buoyant weight of the rack applied as a static load.

TABLE 2-2

POOL SLAB ANALYSIS RESULTS FOR
FIVE-RACK IMPACT

Estimated Upperbound Uplift = 0.76 in.
Assumed Uplift = 1.0 in.
Maximum Impact Velocity--
Method I : Based on Angular Velocity = 10.02 in/sec
Method II: Based on Free Fall = 27.8 in/sec (See Note 4 below)

Equivalent Uniform Dead Load Without Impact--
5 New Racks + Old Racks = 0.86K/ft^2

Total (including seismic) = 4.99K/ft^2

Location of Drop	Shear Load (Kip/ft) Using Method I(1)			
	At Support		At Distance d From Support	
	Computed	Allowable	Computed	Allowable
A	65.5	522.2(2)	<40.1	82.6(3)
B	<65.5	522.2(2)	<40.1	82.6(3)

NOTES: (1) Average shear load at the north end of the pool floor

(2) Based on Section 11.7.3 of Reference 5 (Shear Friction)

(3) Based on Section 11.3.1.1 of Reference 5 (i.e., the short formula)

(4) Computed for the purpose of comparison only

TABLE 2-2 (continued)

POOL SLAB ANALYSIS RESULTS FOR
FIVE-RACK IMPACT

Location of Drop	Shear Load (Kip/ft) Using Method II[4][1]			
	At Support		At Distance d From Support	
	Computed	Allowable	Computed	Allowable
A	129.0	522.2[2]	<40.1	82.6[3]
B	<129.0	522.2[2]	40.1	82.6[3]

NOTES: (1) Average shear load at the north end of the pool floor

(2) Based on Section 11.7.3 of Reference 5 (Shear Friction)

(3) Based on Section 11.3.1.1 of Reference 5 (i.e., the short formula)

(4) Computed for the purpose of comparison only

Table 2-2a

Pool Slab Analysis Results for
Ten-Rack Impact

Equivalent Uniform Dead Load Without Impact--
10 New Racks + Old Racks = 0.98 k/ft^2
Total (including seismic) = 5.13 k/ft^2

Method	Shear Load (kip/ft)(1)			
	At Support		At Distance d From Support	
	Computed	Allowable	Computed	Allowable
Method I	84.1	522.2(2)	52.5	82.6(4)
Method(3) II	179.2	522.2(2)	87.2	85.1(5) 90.8(6)

NOTES: (1) Average shear load at north end of the pool floor.
(2) Based on Section 11.7.3 of Reference 5 (shear friction)
(3) Computed for the purpose of comparison only.
(4) Based on Section 11.3.1.1 of Reference 5.
(5) Based on Section 11.3.2.1 of Reference 5.
(6) Based on Section 11.3.2.1 of Reference 5 with f'_c increased by 15% due to aging.

3.0 EVALUATION OF RACK STRESSES

Table 3-1 presents the maximum loads on rack legs when the impact force resulting from 1-inch uplift is considered. These are based on impact at Location A, which gives the most critical rack loads. Also, the effect of energy dissipation in target mass inertia was ignored to maximize these rack loads, and no overall pool slab flexibility was considered. Table 3-1 also presents the rack loads from the original fixed-base analysis for the purpose of comparison. Table 3-2 lists the stresses in various critical components of the racks when the effect of impact resulting from 1-inch uplift is included. Comparison of these stresses with the allowable stresses shows that the rack design is adequate and no overstress condition is expected.

TABLE 3-1

MAXIMUM RACK LEG FORCES DUE TO RACK IMPACT ON POOL SLAB

Consideration	Maximum Force (kips)			
	Method I		Method II(1)	
	Corner Leg	Middle Leg	Corner Leg	Middle Leg
Considering Rack Impact Near Wall	92.3	115.4	256.2	320.2
Original Fixed-Base Analysis	179.8	208.9	179.8	208.9

Note: (1) Computed for the purpose of comparison only.

TABLE 3-2

STRESS IN RACK COMPONENTS DUE TO RACK IMPACT ON POOL SLAB (3)

Rack Component	Load Combination	Critical Stress Type	Allowable[2] Stress (ksi)	Computed Stress (ksi)	
				Method I	Method II [1]
Tube Wall	D+B+E'	Membrane	33.5	11.53	31.98
Fuel Support Plate	D+B+E'	Membrane	33.5	9.28	25.74
Filler Plate	D+B+E'	Membrane	33.5	9.19	25.49
Base Grid	D+B+E'	Membrane	33.5	1.70	4.70
Rack Leg	D+B+E'	Membrane	33.5	8.55	23.71
Interface	D+B+E'	Bearing	4.76	0.87	2.42

Note: (1) Computed for the purpose of comparison only.

(2) Using a Dynamic Increase Factor of 1.2 per Reference 4.

(3) Deceleration loads resulting from impact are maximum at the uplifted end (i.e., the impacted end) and zero at the pivoted end. The rack leg reaction forces shown in Table 3-1 are for the impacted legs, and so are based on maximum deceleration values. Thus, the ratio between these reaction forces to the dead load reaction forces gives the conservative scaling factor with which the dead load stresses (from original finite element analysis) were multiplied to obtain the stresses in the rack for impact loads shown in this table.

4.0 EVALUATION OF NORTH WALL

Because of its proximity to the impacting racks, the north wall of the pool will have loads higher than the other walls. This wall was evaluated for the combined load (SSE load case) including the effect of five new racks impacting on the pool floor. Table 4-1 presents the comparison of the shear capacity with the computed shear loads based on impact at the critical Location A, for the five rack case. Table 4-1a presents the comparison of the shear capacity with the computed shear loads based on impact of the legs nearest the North wall, which is limiting, for the ten rack case. Results presented in Tables 4-1 and 4-1a show that the computed values are well within the shear capacity of the wall for both the five and ten rack cases.

TABLE 4-1

EVALUATION OF THE VERTICAL SHEAR CAPACITY OF THE NORTH WALL (3)

Load Combination	Vertical Shear (kips)		
	Allowable	Computed	
		Method I	Method II(1)
D+L+H+E'+Impact	15,724(2)	3814	4607

Note: (1) Computed for the purpose of comparison only.

(2) Considering the effect of vertical reinforcement in resisting diagonal tension resulting from shear.

(3) Based on the critical impact Location A.

Table 4-1a

EVALUATION OF THE VERTICAL SHEAR CAPACITY OF THE NORTH WALL (3)

Load Combination	Vertical Shear (kips)		
	Allowable	Computed	
		Method 1	Method II (1)
D+L+H+E'+Impact	13,600 (2)	4,270	5,811

Note: (1) Computed for the purpose of comparison only.

(2) Considering the effect of vertical reinforcement in resisting diagonal tension resulting from shear.

(3) Based on impact of legs nearest North wall.

5.0 REFERENCES

1. Housner, G. W., "The Behavior of Inverted Pendulum Structures during Earthquakes," Bulletin of Seismological Society of America, Vol. 53, No. 2, February, 1963.

2. Aslam, M., Godden, W. G., and Scalise, "Earthquake Rocking Response of Rigid Bodies," J. of Structural Division, ASCE, February, 1980.

3. Topical Report, "Design of Structures for Missile Impact," BC-TOP-9A, Revision 2, Bechtel Power Corporation, 1974.

4. ASCE Manual and Report on Engineering Practice No. 58, "Structural Analysis and Design of Nuclear Plant Facilities," 1980.

5. "Building Code Requirements for Reinforced Concrete," ACI-318-77, American Concrete Institute, 1977.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
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COMMONWEALTH EDISON COMPANY	)	Docket Nos. 50-237
	)	50-249
(Dresden Station, Units 2 and 3)	)	(Spent Fuel Pool Modification)

CERTIFICATE OF SERVICE

I hereby certify that copies of NRC STAFF RESPONSE TO APPLICANT'S MOTION FOR A PARTIAL INITIAL DECISION in the above-captioned proceeding have been served on the following by deposit in the United States mail, first class or, as indicated by an asterisk, through deposit in the Nuclear Regulatory Commission's internal mail system, this 13th day of August, 1981.

John F. Wolf, Esq., Chairman
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Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555 *

Atomic Safety and Licensing Appeal Board Panel
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555 *

Docketing and Service Section
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555 *

Richard J. Goddard
Counsel for NRC Staff