ATTACHMENT 1

TECHNICAL EVALUATION REPORT

EVALUATION OF SPENT FUEL RACKS STRUCTURAL ANALYSIS

FLORIDA POWER AND LIGHT COMPANY

TURKEY POINT UNITS 3 AND 4

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CONTENTS

8

CONTENTS (Cont.)

FOREWORD

This Technical Evaluation Report was prepared by Franklin Research Center under a contract with the U.S. Nuclear Regulatory Commission (Office of Nuclear Reactor Regulation, Division of Operating Reactors) for technical assistance in support of NRC operating reactor licensing actions. The technical evaluation was conducted in accordance with criteria established by the NRC.

The following staff of the Franklin Research Center contributed to the technical preparation of this report: R. Clyde Herrick, Vincent K. Luk, and Balar S. Dhillon (consultant).

1. INTRODUCTION

1.1 PURPOSE OF THE REVIEW

This technical evaluation report (TER) covers an independent review of the Florida Power & Light Company's licensing report [1] on high-density spent fuel racks for Turkey Point Units 3 and 4 with respect to the evaluation of the spent fuel racks' structural analyses, the fuel racks' design, and the pool's structural analysis. The objective of this review was to determine the structural adequacy of the Licensee's high-density spent fuel racks and spent fuel pool.

1.2 GENERIC BACKGROUND

Many licensees have entered into a program of introducing modified fuel racks to their spent fuel pools that will accept higher density loadings of spent fuel in order to provide additional storage capacity. However, before the higher density racks may be used, the licensees are required to submit rigorous analysis or experimental data verifying that the structural design of the fuel rack is adequate and that the spent fuel pool structure can accommodate the increased loads.

The analysis is complicated by the fact that the fuel racks are fully immersed in the spent fuel pool. During a seismic event, the water in the pool, as well as the rack structure, will be set in motion resulting in fluid-structure interaction. The hydrodynamic coupling between the fuel assemblies and the rack cells, as well as between adjacent racks, plays a significant role in affecting the dynamic behavior of the racks. In addition, the racks are free-standing. Since the racks are not anchored to the pool floor or the pool walls, the motion of the racks during a seismic event is governed by the static/dynamic friction between the rack's mounting feet and the pool floor, and by the hydrodynamic coupling to adjacent racks and the pool walls.

Accordingly, this report covers the review and evaluation of analyses submitted for Turkey Point Units 3 and 4 by the Licensee, wherein the structural analysis of the spent fuel racks under seismic loadings is of primary concern due to the nonlinearity of gap elements and static/dynamic friction, as well as fluid-structure interaction. In addition to the evaluation of the dynamic structural analysis for seismic loadings, the design of the spent fuel racks and the analysis of the spent fuel pool structure under the increased fuel load are reviewed.

2. ACCEPTANCE CRITERIA

2.1 APPLICABLE CRITERIA

The criteria and guidelines used to determine the adequacy of the high-density spent fuel racks and pool structures are provided in the following documents:

- OT Position for Review and Acceptance of Spent Fuel Storage and Handling Applications, U.S. Nuclear Regulatory Commission, January 18, 1979 [2]

- Standard Review Plan, NUREG-0800, U.S. Nuclear Regulatory Commission

  Section 3.7, Seismic Design
  Section 3.8.4, Other Category I Structures
  Appendix D to Section 3.8.4, Technical Position on Spent Fuel Pool Racks
  Section 9.1, Fuel Storage and Handling

- ASME Boiler and Pressure Vessel Code, American Society of Mechanical Engineers

  Section III, Subsection NF, Component Supports
  Subsection NB, Typical Design Rules

- Regulatory Guides, U.S. Nuclear Regulatory Commission

  1.29 - Seismic Design Classification

  1.60 - Design Response Spectra for Seismic Design of Nuclear Power Plants

  1.61 - Damping Values for Seismic Design of Nuclear Power Plants

  1.92 - Combining Modal Responses and Spatial Components in Seismic Response Analysis

  1.124 - Design Limits and Loading Combinations for Class 1 Linear-Type Component Types

- Other Industry Codes and Standards

  American National Standards Institute, N210-76

  American Society of Civil Engineers, Suggested Specification for Structures of Aluminum Alloys 6061-T6 and 6067-T6.

2.2 PRINCIPAL ACCEPTANCE CRITERIA

The principal acceptance criteria for the evaluation of the spent fuel racks' structural analysis for Turkey Point Units 3 and 4 are set forth by the NRC's OT Position for Review and Acceptance of Spent Fuel Storage and Handling Applications (OT Position Paper) [2]. Section IV of the document describes the mechanical, material, and structural considerations for the fuel racks and their analysis.

The main safety function of the spent fuel pool and the fuel racks, as stated in that document, is "to maintain the spent fuel assemblies in a safe configuration through all environmental and abnormal loadings, such as earthquake, and impact due to spent fuel cask drop, drop of a spent fuel assembly, or drop of any other heavy object during routine spent fuel handling."

Specific applicable codes and standards are defined as follows:

"Construction materials should conform to Section III, Subsection NF of the ASME* Code. All materials should be selected to be compatible with the fuel pool environment to minimize corrosion and galvanic effects.

Design, fabrication, and installation of spent fuel racks of stainless steel materials may be performed based upon the AISC** specification or Subsection NF requirements of Section III of the ASME B&PV Code for Class 3 component supports. Once a code is chosen its provisions must be followed in entirety. When the AISC specification procedures are adopted, the yield stress values for stainless steel base metal may be obtained from the Section III of the ASME B&PV Code, and the design stresses defined in the AISC specifications as percentages of the yield stress may be used. Permissible stresses for stainless steel welds used in accordance with the AISC Code may be obtained from Table NF-3292.1-1 of ASME Section III Code."

Criteria for seismic and impact loads are provided by Section IV-3 of the OT Position Paper, which requires the following:

- o Seismic excitation along three orthogonal directions should be imposed simultaneously.

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* American Society of Mechanical Engineers Boiler and Pressure Vessel Codes, Latest Edition.

** American Institute of Steel Construction, Latest Edition.

- o The peak response from each direction should be combined by the square root of the sum of the squares. If response spectra are available for vertical and horizontal directions only, the same horizontal response spectra may be applied along the other horizontal direction.
- o Increased damping of fuel racks due to submergence in the spent fuel pool is not acceptable without applicable test data and/or detailed analytical results.
- o Local impact of a fuel assembly within a spent fuel rack cell should be considered.

Temperature gradients and mechanical load combinations are to be considered in accordance with Section IV-4 of the OT Position Paper.

The structural acceptance criteria are provided by Section IV-6 of the OT Position Paper. For sliding, tilting, and rack impact during seismic events, Section IV-6 of the OT Position Paper provides the following:

> "For impact loading the ductility ratios utilized to absorb kinetic energy in the tensile, flexural, compressive, and shearing modes should be quantified. When considering the effects of seismic loads, factors of safety against gross sliding and overturning of racks and rack modules under all probable service conditions shall be in accordance with the Section 3.8.5.II-5 of the Standard Review Plan. This position on factors of safety against sliding and tilting need not be met provided any one of the following conditions is met:
>
> (a) it can be shown by detailed nonlinear dynamic analyses that the amplitudes of sliding motion are minimal, and impact between adjacent rack modules or between a rack module and the pool walls is prevented provided that the factors of safety against tilting are within the values permitted by Section 3.8.5.II.5 of the Standard Review Plan
>
> (b) it can be shown that any sliding and tilting motion will be contained within suitable geometric constraints such as thermal clearances, and that any impact due to the clearances is incorporated."

3. TECHNICAL REVIEW

3.1 MATHEMATICAL MODELING AND SEISMIC ANALYSIS OF SPENT FUEL RACK MODULES

The submerged spent fuel rack modules exhibit highly nonlinear structural behavior under seismic excitation. The sources of nonlinearity can generally be categorized by the following:

a. The impact between fuel cell and fuel assembly: The fuel assembly standing inside a fuel cell will impact its four inside walls repeatedly under earthquake loadings. These impacts are nonlinear in nature and when compounded with the hydrodynamic coupling effect will significantly affect the dynamic responses of the modules in seismic events.

b. Friction between module base and pool liner: The modules are free-standing on the pool liner, i.e., they are neither anchored to the pool liner nor attached to the pool wall. Consequently, the modules are held in place by virtue of the frictional forces between the module base and pool liner. These frictional forces act together with the hydrodynamic coupling forces to both excite and restrain the module during seismic events.

All modules at Turkey Point Units 3 and 4 have nearly square cross sections across the axes of fuel cells [1]. Modules of this design geometry generally behave in three-dimensional fashion under earthquake loadings. Hence, the modules will exhibit three-dimensional nonlinear structural behavior in seismic events, and all seismic analyses of modules should therefore focus on characterizing this behavior.

There are two types of modules at Turkey Point Units 3 and 4 [1]. The modules in Region I have a center-to-center storage cell spacing of 10.6 in. They are reserved for temporary core off-loading, temporary storage of new fuel, and storage of spent fuel above specified levels of reactivity. The modules in Region II, with 9.0-in center-to-center spacing, are used to store irradiated fuel below specific reactivity levels. The designs of modules in Regions I and II are shown in Figures 1 and 2, respectively.

The Licensee conducted the seismic analysis of modules in two parts. The first part was a time history analysis of a simplified two-dimensional nonlinear finite element model of an individual fuel cell shown in Figure 3.

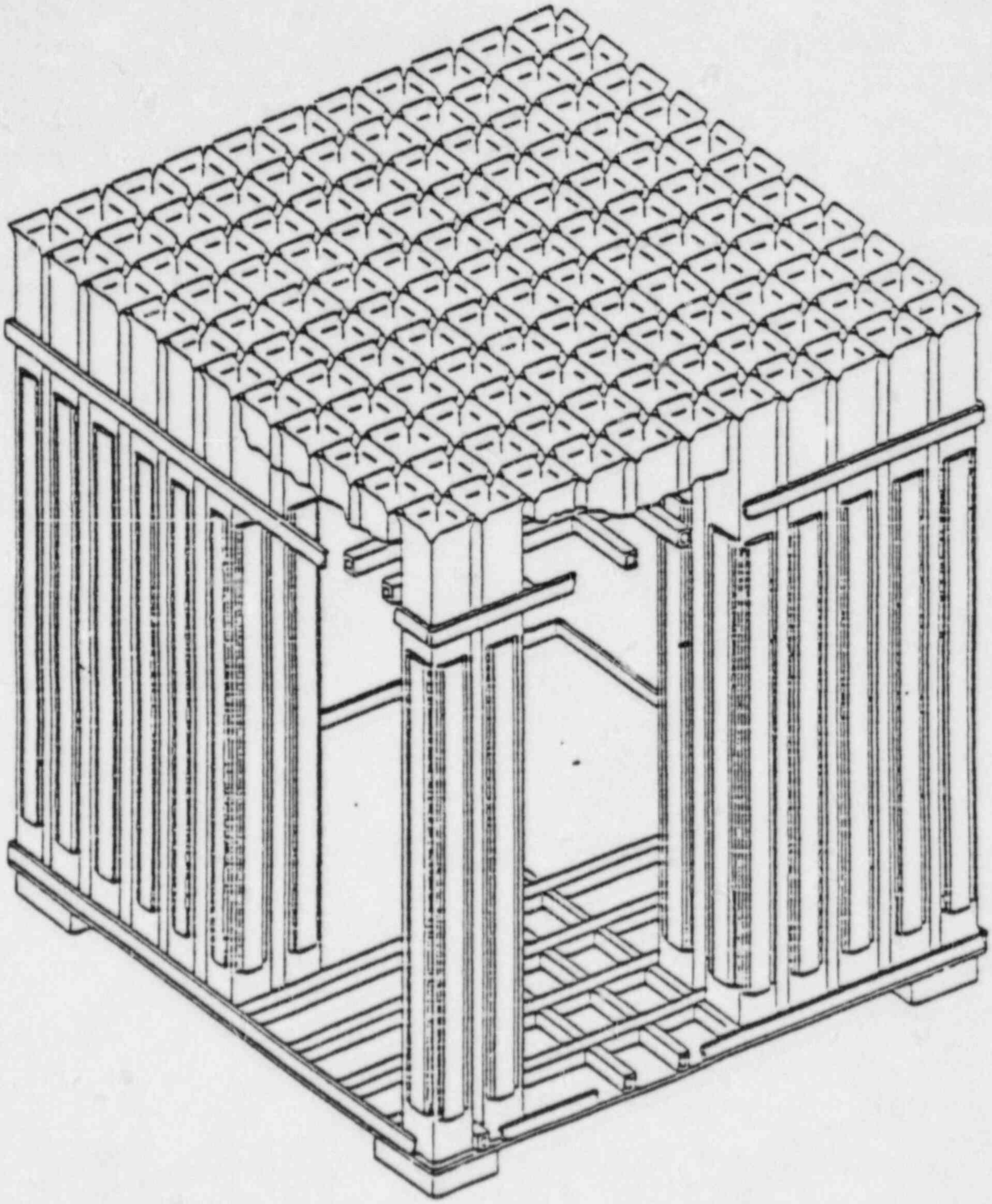


Figure 1. Fuel Storage Rack Assembly in Region I

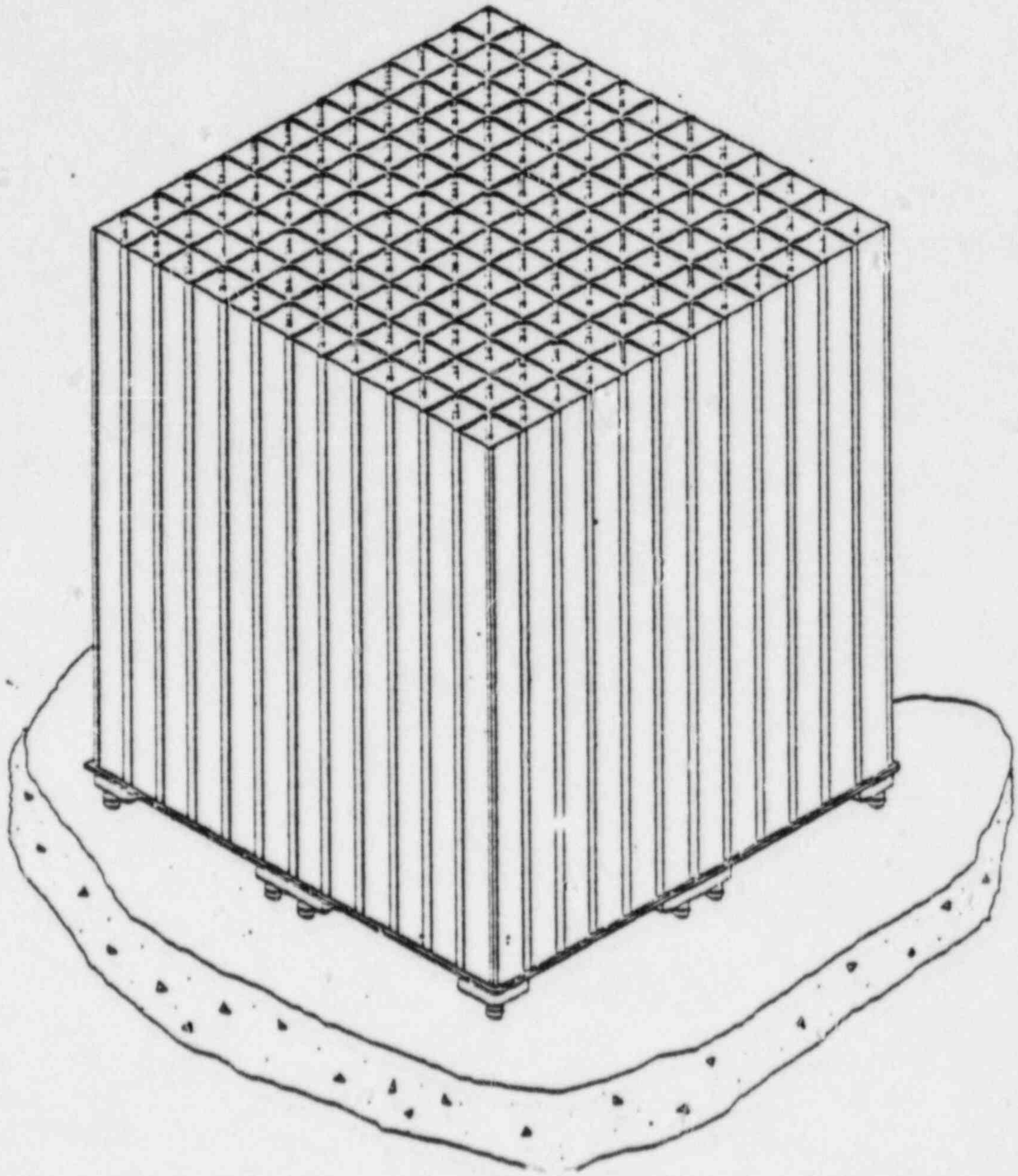


Figure 2. Fuel Storage Rack Assembly in Region II

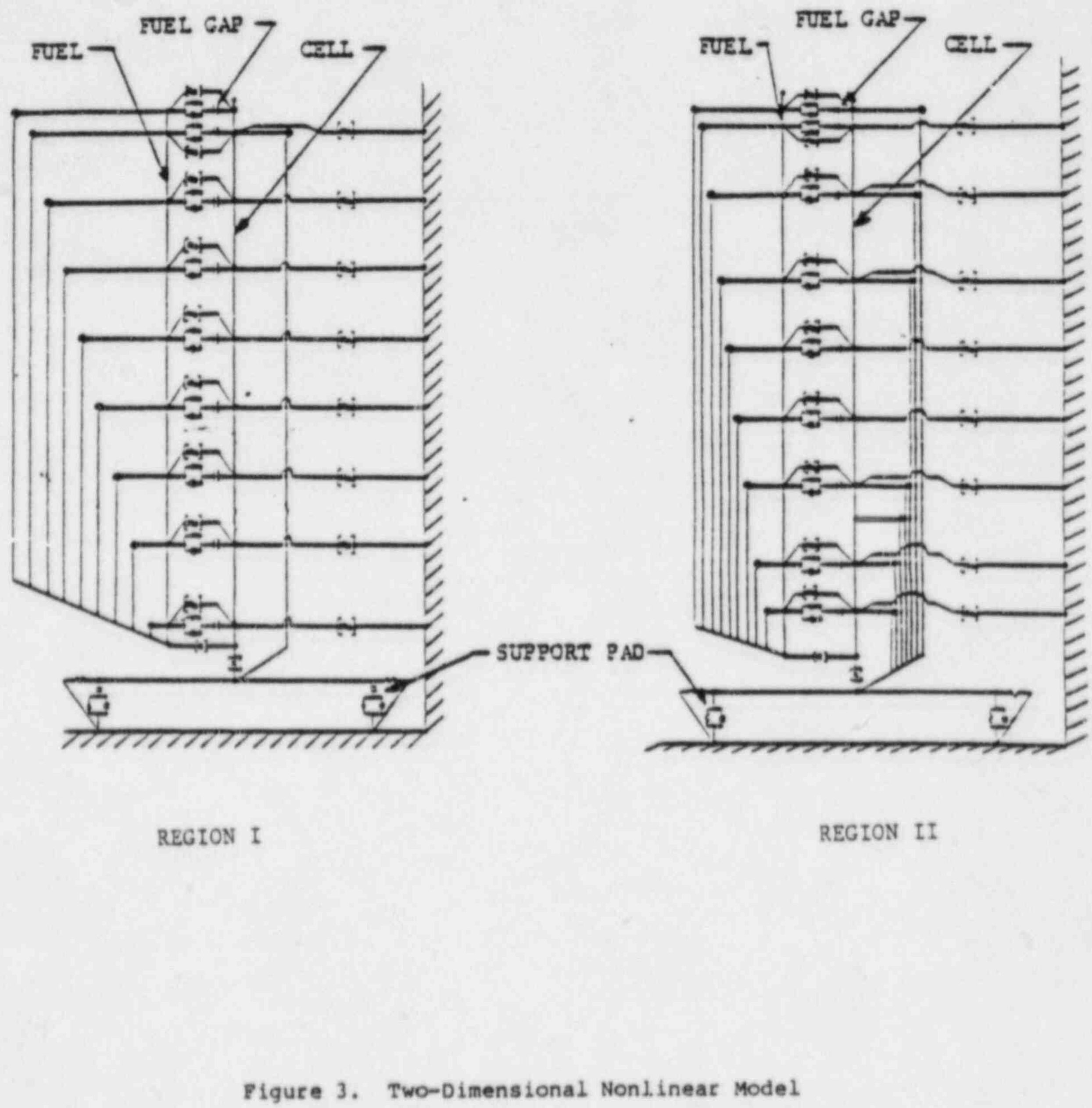


Figure 3. Two-Dimensional Nonlinear Model

The second part was a response spectrum analysis of a detailed three-dimensional linear finite element model of a rack assembly shown in Figure 4. Both modules consisted of two models to reflect the two different designs of modules in Regions I and II. Structural damping of 2% was used in the seismic analysis for both the operating basis earthquake (OBE) and the safe shutdown earthquake (SSE).

In a previous review of similar spent fuel racks, the following issue concerning the modeling technique used in the analysis was discussed [3]:

> The simplified two-dimensional model does not fully simulate the more complicated three-dimensional structure behavior exhibited by the modules. The two-dimensional model essentially uncouples the two mutually perpendicular horizontal motions which are nonlinearly interrelated under seismic loadings. Thus, an approach using two models (nonlinear, two-dimensional and linear, three-dimensional model) may have difficulty in resolving peak stresses.

The description and evaluation of the two models are addressed in detail in Sections 3.2 and 3.3. The displacement and stress results are discussed in appropriate subsections.

3.2 EVALUATION OF THE SIMPLIFIED TWO-DIMENSIONAL NONLINEAR MODEL

3.2.1 Description of the Model

The simplified two-dimensional model was developed to simulate the major structural characteristics of an individual fuel cell within a submerged rack assembly. Two versions of this model are shown in Figure 3 to reflect two different module designs in Regions I and II. The model was developed in accordance with the WECAN (Westinghouse Electric Computer Analysis) code.

A time history analysis of the model was performed by the Licensee with the simultaneous application of a vertical and a horizontal component of seismic loads. Nonlinear gap elements were used in the model to represent the possible impact between the fuel cell and the fuel assembly, as well as the friction between the module base and the pool liner. The hydrodynamic coupling effect between fuel cell and fuel assembly, as well as between fuel cell and rigid wall, is simulated by appropriate coupling springs. A damping

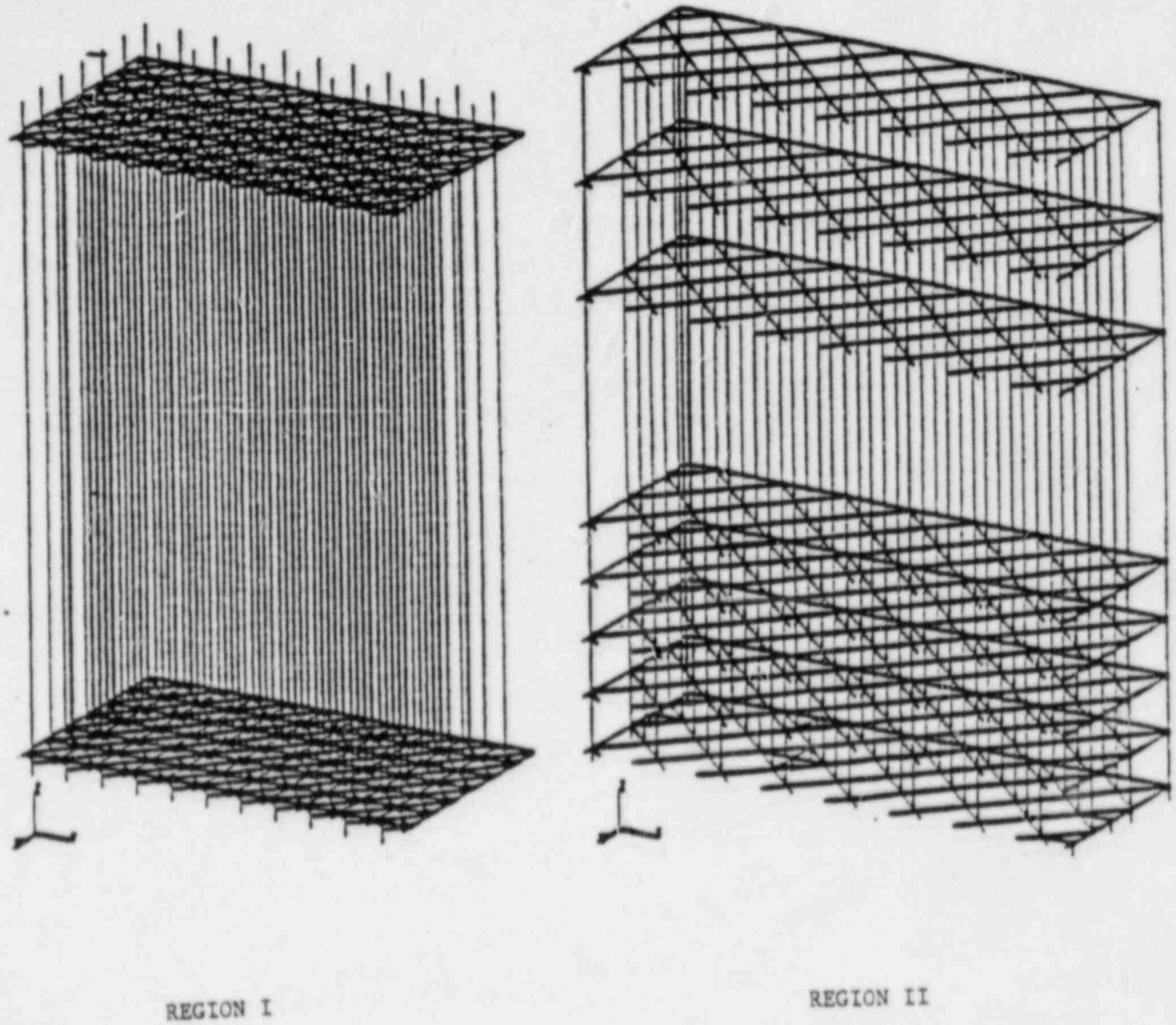


Figure 4. Three-Dimensional Linear Model

value of 25% was used to represent the impact damping of the fuel assembly [4]. This impact damping value was determined from a test consisting of the fuel assembly in air impacting on a grid surface [5].

3.2.2 Assumptions Used in the Analysis

The following assumptions were used in the seismic analysis of the model:

a. A structural damping value of 2% was used for both OBE and SSE events.

b. The fluid damping was conservatively neglected.

c. Only a constant value of friction coefficient was considered in each seismic analysis. The coefficient of friction remained unchanged whether the module was stationary or in motion. Analysis was performed for static friction coefficients of $u = 0.2$ and 0.8. These two cases would envelop the values of intermediate friction coefficients.

d. The initial status of the gap between fuel cells and fuel assembly is immaterial because all fuel cells would move in phase soon after an earthquake occurred. Adjacent modules would also move in phase in seismic events.

e. The sloshing movement of the water is in the upper elevations of the spent fuel pool above the top of the modules. Therefore, no sloshing loads are imposed on the module structure.

The assumption in Item d may be valid when adjacent modules are fully loaded, but the out-of-phase response will most likely occur when some modules are either partially loaded or empty.

3.2.3 Hydrodynamic Coupling Between Fluid and Cell Structure

The hydrodynamic coupling effect between adjacent modules and between the fuel cell and fuel assembly plays a significant role in affecting the dynamic responses of the module in seismic events. As stated in Section 3.2.2, the modules were assumed to move in phase. This assumption led to consideration of the motion of an individual cell surrounded on all four sides by rigid boundaries which are separated from the cell by equivalent gaps as an equivalent representation of the entire rack assembly. The hydrodynamic coupling mass between the rack module and the pool wall, as shown in Figure 3, was calculated by evaluating the effects of the gap between the modules and the pool wall using the method outlined in the paper by Fritz [6].

The technique of potential flow and kinetic energy was used in assessing the hydrodynamic coupling mass between the fuel cell and the fuel assembly. This mass, which depends on the size of fuel assembly and the inside dimensions of the fuel cell, was calculated by equating the kinetic energy of the hydrodynamic coupling mass to that of the fluid flowing around the fuel assembly within the fuel cell. The concept of this method was discussed in a paper by De Santo [7].

Fritz's [6] method for hydrodynamic coupling is widely used and provides an estimate of the mass of fluid participating in the vibration of immersed mass-elastic systems. Fritz's method has been validated by excellent agreement with experimental results [6] when employed within the conditions upon which it was based, that of vibratory displacements which are very small compared to the dimensions of the fluid cavity. Application of Fritz's method for the evaluation of hydrodynamic coupling effects between rack modules and a pool wall has been considered by this review to serve only as an approximation of the actual hydrodynamic coupling forces. This is because the geometry of a fuel rack module in its clearance space, is considerably different than that upon which Fritz's method was developed and experimentally verified.

Thus, the limitations of Fritz's [6] modeling technique for hydrodynamic coupling of rack modules adjacent to other rack modules or a pool wall reinforce the position of this review that the Licensee's fuel rack dynamic model be considered conservative only for dynamic displacements that are small relative to the available displacement clearance.

3.2.4 Seismic Loading

The model was subject to a simultaneous application of a vertical and a horizontal component of seismic loads. The horizontal seismic loads are identical in the north-south and the east-west directions, but there are two different sets of hydrodynamic coupling masses in these two horizontal directions. Conservative results were obtained by the Licensee by conducting one time history analysis in the horizontal direction having the more severe hydrodynamic coupling mass.

3.2.5 Integration Time Step

The Licensee performed a time step study in an effort to find the correct integration time step to yield a converged solution [5]. It was found that the convergence of solution occurred at a time step of 0.001 sec for modules in Region I and 0.005 sec for modules in Region II [4]. These time steps are much greater than the 2.0×10^{-4} sec reported by Gilmore of Westinghouse in a similar analysis [8]. The Licensee explained that the wide range of time steps that yield convergence may be responsible for these differing values.

3.2.6 Rack Displacements

The Licensee claimed that the displacement of the module would be the same as that of the individual cell found in this model because of the in-phase motion assumption used in this analysis. The Licensee found that the maximum combined seismic and thermal module displacements are 0.256 inch in Region I and 0.214 inch in Region II [5]. Both results are smaller than the nominal spacing of 1.11 inch between adjacent modules, and consequently, no collision will occur between adjacent modules. While this result may not be conservative because the two-dimensional model used in this analysis uncouples the two horizontal responses under seismic loadings, it does indicate that the displacements are relatively small.

The detailed rack displacements are tabulated in Table 1 which is taken from the Licensee's response [5] to questions during the review. The moments and shear forces generated from this model were used to calculate the load correction factors. The load results from the detailed model were then multiplied by these factors to yield the stress results in the structural analysis of the module, as discussed in Section 3.3 of this report.
A detailed review of this method was given in Reference 3.

Table 1. Computed Rack Displacements

SSE Seismic + Maximum Normal Thermal			REGION I	REGION II
			SSE Seismic + Normal Thermal	
Max. Sliding Distance, $\mu = .2$ (N-Linear Results)	Δs	in	.0001	0.007
Max. Structural Defl., $\mu = .8$ (N-Linear Results)	δ	in	.124	0.086
Total Displacement One Rack $\Delta = \Delta s + \delta$	Δ	in	.1241	0.093
SSRS Combined Displacement 2 Racks with only 1 sliding $\Delta max = \sqrt{\Delta^2 + \delta^2}$	Δmax	in	.175	0.127
Max. Normal Thermal Displacement	δ_T	in	.088	0.087
Max. Combined Thermal & Seismic Displacements $\bar{\Delta} = \delta_T + \Delta max$	$\bar{\Delta}$	in	.256	0.214
Rack to Rack Gap	GAP	in	1.11	1.11

SSE Seismic Sliding + Max Accident Thermal			REGION I	REGION II
			SSE Seismic Sliding + Thermal Accident	
Max. Sliding Distance, $\mu = .2$	Δs	in	.0001	0.007
Max. Accident Thermal Displacement	δ_T	in	.175	0.190
Combined Thermal & Seismic Sliding $\bar{\Delta} = \Delta_s + \delta_T$	$\bar{\Delta}$	in	.1751	0.197
Rack to Rack Gap	GAP	in	1.11	1.11

NOTE: THE RACK TO WALL GAPS ARE LARGER THAN THE RACK TO RACK GAPS.

Because load correction factors based on base moment and base shear force were employed by the Licensee to introduce the dynamic response from the nonlinear two-dimensional dynamic displacement analysis model to the linear three-dimensional stress analysis, the Licensee provided a comparison of the vertical mounting pad forces in the linear and nonlinear models. Figure 5, which is taken from the Licensee response [5], shows that the summation of vertical forces in the two analysis models is reasonably close and is considered to be satisfactory.

3.3 EVALUATION OF THE DETAILED THREE-DIMENSIONAL LINEAR MODEL

3.3.1 Description of the Model

A model was developed to simulate the major structural characteristics of the entire module submerged in the fuel pool. Two versions of the model are shown in Figure 4 to represent two different module designs in Regions I and II. The WECAN code was used to develop these two models. Three-dimensional beam elements were used to construct the models.

According to Reference 5, the seismic analysis was done on the 10x11 module in Region I and the 10x14 module in Region II. The model of the module in Region I has two fine meshes of elements, one on the top and the other on the bottom of the model to represent the top and the bottom grip assembly of the module, respectively. There are eight horizontal meshes of elements in the model of the module in Region II to simulate the eight skip weld locations along the length of cells.

A response spectrum analysis of the three-dimensional models was performed. The three components of the seismic loads were applied to the models, one component at a time.

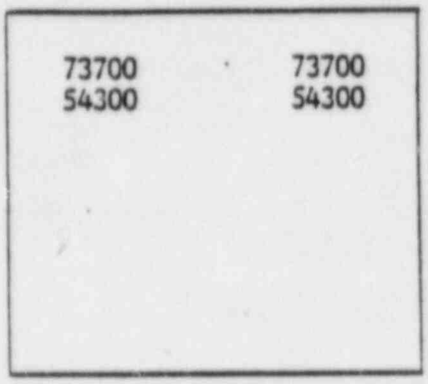
3.3.2 Assumptions Used in the Analysis

All the assumptions except the initial status of the gap between fuel cell and fuel assembly used in the analysis of the two-dimensional model are applicable here. A few additional assumptions used in this analysis are described below:

NON LINEAR MODEL PAD LOADS

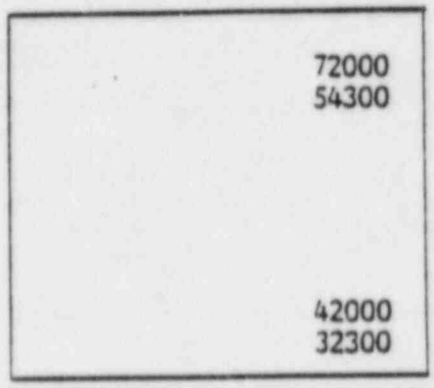
REGION I 10x11

NS + DW



	Linear	Non Linear
Total NS + DW	147400	149600
Total DW	108600	112800
Ratio (NS+DW)/DW	1.36	1.33

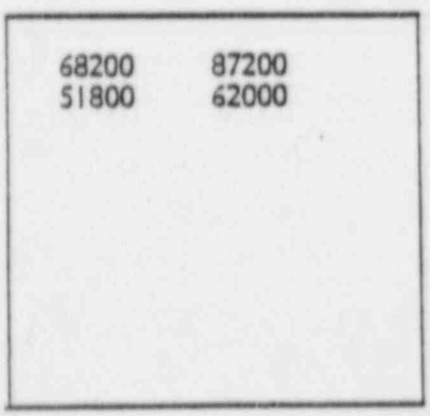
EW + DW



	Linear	Non Linear
Total EW + DW	114000	117000
Total DW	86600	88000
Ratio (EW+DW)/DW	1.32	1.33

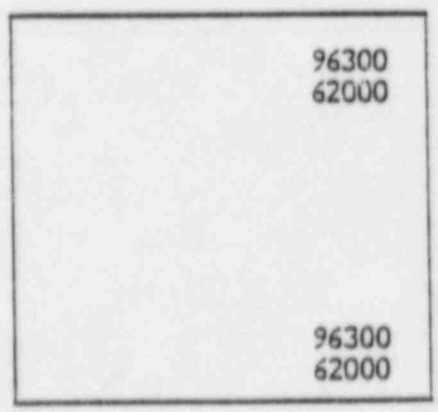
REGION II 10x14

NS + DW



	Linear	Non Linear
Total NS + DW	155400	145300
Total DW	113800	101900
Ratio (NS+DW)/DW	1.37	1.43

EW + DW



	Linear	Non Linear
Total ED + DW	192600	181600
Total DW	124000	114500
Ratio (EW+DW)/DW	1.55	1.59

Figure 5. Comparison of Mounting Pad Loads for the Nonlinear and Linear Rack Analysis Modules

a. A composite distributive mass density was used in the analysis to embody the masses of the fuel cell, the fuel assembly, the poison material, and the hydrodynamic coupling mass.

b. No impact between the fuel cell and the fuel assembly was considered.

c. The module base was stationary with respect to the pool liner at all times.

3.3.3 Load Correction Factor

Since the detailed model did not account for the nonlinear effect of a fuel assembly impacting a fuel cell and the support pad movements, the internal loads and stresses for the module assembly obtained from this model were modified by load correction factors. The calculation was focused on the bending moments and shear forces obtained at the base plate of this detailed model. The bending moment load correction factor was defined as the ratio of the bending moment obtained at the base of the simplified model to the average bending moment derived at the base of the detailed model. Similar definition was used for the shear force load correction factor. The maximum loads from this detailed model were multiplied by these load correction factors and were used in the structural analysis to obtain the streses within the module assembly. Further discussion is provided in Section 3.4.

3.3.4 Module Assembly Lift-Off Analysis

The modules having the largest difference between the two horizontal dimensions were chosen to study the possibility of lift-off. The 8x11 module in Region I and the 9x13 module in Region II were subject to investigation for this purpose. Both modules were found not to lift off the pool liner in seismic events [5].

3.3.5 Stress Results

The maximum responses of the detailed model from the seismic components in three directions were combined by the SRSS model in the structural analysis. Stresses from these responses and from dead weight are shown in Tables 2 and 3 for Region I racks and Region II racks, respectively. Tables

Table 2. Stresses, Region I Racks

REGION I RACKS

SUMMARY OF DESIGN STRESSES AND MINIMUM MARGINS OF SAFETY

Normal & Upset Conditions

		Design Stress (psi)	Allowable Stress (psi)	Margin of Safety
1.0	Support Pad Assembly			
1.1	Support Pad			
	Shear	2009	23150*	10.52
	Axial and Bending	5701	23150*	3.06
	Bearing	4230	23150*	4.47
1.2	Support Pad Screw			
	Shear	3675	9260	1.52
1.3	Support Plate			
	Shear	2152	9260	3.30
	Weld Shear	15672	21000*	.48
2.0	Cell Assembly			
2.1	Cell to Bottom Grid Weld			
	Weld Shear	15840	23150*	.46
2.2	Cell to Top Grid Weld			
	Weld Shear	15840	23150*	.46
2.3	Cell			
	Axial and Bending	.514	1.0**	.94
2.4	Cell to Wrapper Weld			
	Weld Shear	4517	9260	1.05
3.0	Grid Assembly			
3.1	Top Grid Box Member			
	Shear	2055	9260	3.51
	Axial and Bending	1659	13890	7.37
3.2	Top Grid Members			
	Weld Shear	13544	21000	.55
3.3	Top Grid Outer Member			
	Axial and Bending	1707	13890	7.14
	Shear	146	9260	62.51
3.4	Bottom Grid Structure			
	Shear	3349	9260	1.77
	Axial and Bending	12057	13890	.15
3.5	Bottom Grid Members Welds			
	Weld Shear	15702	21000	.34
3.6	Bottom Grid Base Plate Weld			
	Weld Shear	15941	21000	.32
1.0	Grid Assembly - Cont'd			
3.7	Bottom Grid Outer Member			
	Axial and Bending	12050	13890	.15
	Shear	768	9260	11.06
3.8	Base Plate Stiffener to Base Plate Weld			
	Weld Shear	13500	21000	.56

* Thermal Plus OBE Stress is Limiting

** Allowable Per Appendix XVII - 2215 Eq. (24)

Table 3. Stresses, Region II Racks

REGION 2 RACKS

SUMMARY OF DESIGN STRESSES AND MINIMUM MARGINS OF SAFETY

Normal & Upset Conditions

			Design Stress (psi)	Allowable Stress (psi)	Margin of Safety
1.0	Support Pad Assembly				
	1.1	Support Pad			
		Shear	3504	23150*	5.61
		Axial and Bending	10288	23150*	1.25
		Bearing	7631	23150*	2.03
	1.2	Support Pad Screw			
		Shear	6974	9260	.33
	1.3	Support Plate			
		Shear	4403	9260	1.10
		Weld Shear	16556	21000*	.34
2.0	Cell Assembly				
	2.1	Cell			
		Axial and Bending	.899	1.0†	.11
	2.2	Cell to Base Plate Weld			
		Weld Shear	15482	21000	.36
	2.3	Cell to Cell Weld			
		Weld Shear	18389	23150*	.26
	2.4	Cell Seam Weld			
		Weld Shear	1751†	2194††	.25
	2.5	Cell to Wrapper Weld			
		Weld Shear	10299	18520**	.80

* Thermal Plus OBE Stress is Limiting

** SSE Stress is Limiting

† Allowable per Appendix XVII-2215 Eq (24)

†† Design Load and Allowable Load in Lbs is Shown

2 and 3 were provided by the Licensee [5] and the support plate weld shear stress and allowable stresses were subsequently changed as discussed below. Tables 2 and 3 provide the final data which were found to be acceptable during the review.

For Tables 2 and 3, the allowable shear stress in the weld of Item 1.3, Support Plate, was changed to 21,000 psi to be in accordance with the allowable weld stress of Table NF-3292.1-1 of the ASME Code.* For Table 3, the weld shear stress for Item 1.3 was changed to 16,556 psi, recognizing that the support plate compressive load is carried in metal-to-metal contact and is not dependent upon the weld.

3.4 REVIEW OF SPENT FUEL POOL STRUCTURAL ANALYSIS

3.4.1 Spent Fuel Pool Structural Analysis

The spent fuel pool is a reinforced concrete plate structure supported on compacted limerock fill. The spent fuel pool walls are lined with 1/4-in stainless steel liner. The Licensee presented an analysis to demonstrate the structural integrity of the spent fuel pool for the postulated loading conditions for the new high density racks.

3.4.2 Analysis Procedure

The Licensee used the finite element method for the analysis of the spent fuel pool. The structure was modeled with three-dimensional solid elements and the ANSYS computer code. By approximating symmetry along the long (north-south) diretion of the pool, only half of the pool was modeled. The boundary conditions on the plan of symmetry were adjusted to represent symmetric and non-symmetric loading conditions. The liner plate was not considered to provide structural resistance in the pool analysis. The soil medium was represented by vertical compression spring elements. The thermal effects were obtained by imposing a uniform thermal gradient across solid elements.

* American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section III, Division 1, Subsection NF, 1980 Edition.

The following critical loading combinations were considered.

1. $Y = 1.25\ (D+P+L)$ with and without T
2. $Y = 1.25\ (D+P+L)$ with and without W
3. $Y = 1.25\ (D+P+L+E)$ with and without T
4. $Y = 1.0\ (D+P+L+E')$ with and without T

where Y = required yield strength of the structure
D = weight of the structure plus permanent loads
P = hydrostatic pressure of pool water
L = weight of loaded fuel racks in pool
E = design earthquake load, 0.05g horizontally, 2/3 (0.05g) vertically
E'= maximum earthquake load, 0.15g horizontally, 2/3 (0.15g) vertically
T = thermal load (inside face of walls 180°F, exposed face 30°F, and bottom face of slab 50°F)
W = wind load.

As a result of this analysis, the Licensee stated the following:

1. Seismic analysis for the new racks showed that these racks do not uplift during the seismic event and, therefore, no additional amplification factors for impact were considered.

2. The analysis showed that the seismic loading created a more severe effect than the combined effect of tornado, wind, and depressurization.

3. The resulting stresses in the elements caused by mechanical loads were evaluated by computing the capacities of individual sections and comparing the capacities to the actual normal forces and moments.

4. For the combinations of mechanical and thermal loads, the sections were analyzed following the approach shown in "Commentary to ACI 349-R-80."

5. A separate analysis was conducted to determine the effects of thermal, hydrostatic, and hydrodynamic loads on the functionality of the liner. The analysis showed that there was no loss of function.

The results of the structural analysis were summarized in the Licensee's Table A [5], reproduced here as Tables 4-a and 4-b.

Table 4-a. Spent Fuel Pool Load Combinations and Stresses

	MECHANICAL LOADS							MECHANICAL & THERMAL		
	1.25 (D + P + L)				1.25 (D + P + L) + E			1.25 (D + P + L) + E + T		
				(1)				(1)	(2)	(3)
Location	N (K/ft)	M K-ft/ft	M_m K-ft/ft	Mm/M	N (K/ft)	M K-ft/ft	M_m K-ft/ft	Mm/M	Rebar Stress	$\frac{ØFY}{\text{Rebar}}$ Stress
Base Mat	18.1	7.8	23	2.95	13.2	16.7	27	1.6	fs = 12.8 ksi (5)	2.81
East Wall (Canal)	9.6	-22 (fv = 82 psi)	-52	2.36 1.80(4)	25.0	-29.3 (fv = 142 psi)	-43	1.47 1.04 (4)	fv = 142 psi (6)	1.04 (4)
East Wall (Pool)	33.2	122	568	4.66	64.6	163	490	3.0	fs = 35.1 ksi f's = -9.6 ksi	1.03
North Wall	19.8	-96.6	-123	1.27	13.1	-140	-151	1.08	fs = 27.1 ksi f's = -2.65 ksi	1.33
South Wall	18.9	-38.5	-192	4.99	23.0	-76.1	-182	2.39	fs = 35.3 ksi(7) f's = 1.4 ksi	1.02
Middle Wall	28.5	22.1	209	9.46	2.6	32.5	218	6.7	fs = 9.6 ksi f's = 9.0 ksi	3.75

N = Applied normal force on section
M = Applied moment on section
M_m = Maximum elastic moment
(negative sign indicates compressive stress)

fs = Stress in tension steel
f's = Stress in compression steel
fv = Concrete shear stress

Table 4-b. Notes for Table 4-a

(1) Maximum elastic moment for a section with normal force N imposed on it.

(2) Based on a cracked analysis per the methodology discussed in Reference 2, reinforcing steel stress is obtained directly.

(3) Due to the self relieving nature of thermal loads on reinforced concrete, the ratio of maximum moment capacity to actual moment cannot be uniquely determined. As an alternative, the ratio of ϕF_y to computed reinforcing steel stress is provided. Since structural integrity is maintained beyond the allowable stress for thermal loading, the actual safety factor is greater than the ratio reported.

(4) Where shear stresses control, the ratio provided is that of allowable shear stress (conservatively taken as 148 psi) divided by f_v.

(5) This stress represents the maximum stress found in the top layer of reinforcing steel in the thinner center section of the base mat. The top steel in this area is important for transfer of the tensile loads imposed by the lateral water pressure from the pool. The bottom steel in the center portion of the base mat of the pool is used primarily for crack control. Since the base mat rests directly on competent fill material, stresses in this bottom (secondary) steel resulting from thermal loads have no adverse effect on the ability of the pool to transfer load. Therefore, the stress in the bottom steel is not included in Table A.

(6) As shown in Figure 6, this section occurs in the 3 foot wide by 18 inch thick section of the east wall between the two canal walls. Because of the short span of this section, and the large ratio of section thickness to span length, the section does not resist loads in the fashion of a shallow beam; shear stresses control the section capacity. Since shear stirrups are provided, the allowable shear stress in the concrete exceeds 148 psi. The reinforcing steel on the outside face of this section is used only for crack control and is not needed to resist mechanical loads. Therefore, the flexural stresses in this reinforcing steel are not included in Table A.

(7) This represents an average stress (total force on the total section) over the top 10 feet of the outside face horizontal reinforcing steel. The result indicates that the section in general remains below the minimum specified yield stress. However, a maximum stress of 38 ksi has been calculated for the reinforcing steel in the top element of the wall. Realizing the self-relieving nature of the thermal stresses and further acknowledging that the section in general remains elastic, pool function and structural integrity are maintained. Additionally, in accordance with the Turkey Point Updated FSAR, Appendix 5A, Section II, limited yielding is allowable provided the deflection is checked to ensure that the affected Class I systems and equipment are not stressed beyond their allowables. No Class I systems or equipment are attached to this section of wall. REV 10-17-84

3.4.3 Summary of Results

The results of the analysis listed in Table 4-a show that the stress levels under critical loading combinations remain within the specified allowable values, but with one exception. The review showed that:

1. The average bearing stress under the pool slab is below the allowable pressure of 10 ksf for the compacted limerock fill.

2. The maximum tensile stress in steel is shown to be 35.3 ksi compared to the allowable value, F_y = 36.0 ksi.

3. The shear stress in concrete controls the design in the 18-in-thick section of the east wall between the two canals. The ratio of the allowable shear stress to the maximum shear stress is shown to be 1.04.

The exception to stresses within the allowable values concerns the tensile stress in the steel of the south wall, which, in accordance with note 7 of Tables 4-a and 4-b, was computed to be a maximum of 38 ksi. For use in Table 4-a and for comparison to the allowable value, the Licensee averaged the maximum stresses in the steel over the upper 10 ft of wall to yield an average of 35.3 ksi which was compared to the allowable value of 36 ksi. Where this procedure may be questioned, the Licensee also cited Appendix 5A, Section II of Turkey Point's updated FSAR which states that limited yielding is allowable under certain accident conditions. This was reviewed and considered to be acceptable.

In addition, the Licensee's response [10] to USNRC Question No. 8 regarding the effects of 212°F water in the spent fuel pool concludes that stresses for the thermal load remain within the original design allowables. For simultaneous occurrences of seismic and thermal conditions, the Licensee reported [10] that localized steel stresses were slightly higher than the allowable stress of 36 ksi, and justified their magnitudes by the FSAR statement cited in the paragraph above that would permit local thermal stress yielding under certain accident conditions.

After considering this review, evaluation showed that the 212°F pool water temperature resulted from a cooling system pipe break during a seismic event. Thus, considering the hours it would take to raise the pool water temperature to 212°F and increase the thermal gradient in the pool structure, the short duration seismic event would have been long past sc that the structural considerations would remain to be those of thermal and deadweight only. The Licensee's response to USNRC Question No. 8 [10] indicates that analysis showed this to be 38 ksi versus the allowable value of 36 ksi and was justified by statements in the FSAR as discussed above.

This review concludes that the spent fuel structure is acceptable for the higher density loading.

3.5 FUEL ASSEMBLY DROP ACCIDENT ANALYSIS

With respect to accidental dropping of a fuel assembly, the Licensee provided the following:

> "In the unlikely event of dropping a fuel assembly, accidental deformation of the rack will not cause the criticality acceptance criterion to be violated.
>
> For the analysis of a dropped fuel assembly, three accident conditions are postulated. The first accident condition conservatively assumes that the weight of a fuel assembly, control rod assembly and handling mechanism of 3,000 pounds impacts the top end fitting of a stored fuel assembly from a drop height of 3 feet. Calculations will show that the impact energy is absorbed by the dropped fuel assembly, the stored fuel assembly, the cells and rack base plate assembly. If in the unlikely event that two adjacent cells are crushed together for their fuel length, critically, calculations show that $k_{eff} \leq 0.95$. Under these faulted conditions, credit is taken for dissolved boron in the water, and the critically acceptance criterion is not violated.
>
> The second accident condition is an inclined drop on top of the rack. Results will be the same as for the first condition.
>
> The third accident assumes that the dropped assembly (3,000 lbs) falls straight through an empty cell and impacts the rack base plate drom a drop height of 201 inches. The results of this analysis will show that the impact energy is absorbed by the fuel assembly and the rack base plate. Criticality calculations shown that $k_{eff} \leq 0.95$ and the critically acceptance criterion is not violated."

This statement was found to be acceptable during the review.

5. REFERENCES

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   Licensing Report on Turkey Point Units 3 and 4
   Spent Fuel Storage Facility Modifications, Safety Analysis Report
   NRC Docket Nos. 50-250 and 50-251

2. OT Position for Review and Acceptance of Spent Fuel Storage and Handling Applications, U.S. Nuclear Regulatory Commission
   January 18, 1979

3. Franklin Research Center
   Technical Evaluation Report, "Evaluation of Spent Fuel Racks Structural Analysis for Duke Power Company, McGuire Nuclear Station Units 1 and 2"
   August 10, 1984

4. Florida Power & Light Company
   Response to FRC's Request for Information
   October 5, 1984

5. Florida Power & Light Company
   Response to FRC's Questions
   October 1, 1984

6. R. J. Fritz
   "The Effect of Liquids on the Dynamic Motions of Immersed Solids"
   Journal of Engineering for Industry
   pp. 167-173, February 1972

7. D. F. De Santo
   "Added Mass and Hydrodynamic Damping of Perforated Plates Vibrating in Water"
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8. C. B. Gilmore
   "Seismic Analysis of Freestanding Fuel Racks"
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9. ASME Boiler and Pressure Vessel Code, Section III, Division 1, Subsection NF, 1980 Edition, Table NF-3292.1-1

10. Florida Power & Light Company
    Response to USNRC Question No. 8 regarding the effects of a sustained pool water temperature of 212°F on the pool and cooling system

4. CONCLUSIONS

Based upon the review and evaluation, the following conclusions were reached:

- The limitations of the modeling technique employed for hydrodynamic coupling of fuel assemblies within a fuel rack cell and of fuel rack modules to other rack modules and the pool walls indicate that the modeling technique contributes known accuracy only for the condition in which the displacements are small compared to the available clearance space. As the Licensee's reported displacements are small, an acceptable use of the hydrodynamic coupling was employed.

- Computed displacements are small relative to clearance between rack modules or between rack modules and the spent fuel pool walls. Thus, the use of two-dimensional dynamic rack module analysis was satisfactory for displacement.

- While the methodology employing two-dimensional nonlinear models and linear three-dimensional models correlated by load correcting factors to introduce the nonlinear impacting load characteristics to the three-dimensional linear model was not considered to be fully acceptable without further validation as a stress analysis method, a detailed step-by-step review of the stress analysis coupled with additional load tabulations requested and supplied indicates that, with the conservatisms noted to be present, the stress analysis is acceptable.

- The spent fuel pool structure has design margin to sustain the higher density floor loadings.