APPENDIX A

TECHNICAL EVALUATION REPORT

NRC DOCKET NO. 50-277, 50-278

NRC TAC NO. 59012, 59013

NRC CONTRACT NO. NRC-03-81-130

FRC PROJECT C5506

FRC ASSIGNMENT 26

FRC TASK 585

EVALUATION OF SPENT FUEL RACKS STRUCTURAL ANALYSIS

PHILADELPHIA ELECTRIC COMPANY

PEACH BOTTOM UNITS 2 AND 3

TER-C5506-585

Prepared for

Nuclear Regulatory Commission
Washington, D.C. 20555

FRC Group Leader: R. C. Herrick
NRC Lead Engineer: S. B. Kim

January 15, 1986

Prepared by:	Reviewed by:	Approved by:
R. C. Herrick	Vu Ngoc Con	D. P. Carfagno
Principal Author		Department Director
Date: Jan 15, 1986	Date: Jan 15/86	Date: 1-15-86

FRANKLIN RESEARCH CENTER
DIVISION OF ARVIN/CALSPAN
20th & RACE STREETS, PHILADELPHIA, PA 19103

8601170206

XA

CONTENTS

Section	Title	Page
1	INTRODUCTION	1
	1.1 Purpose of the Review	1
	1.2 Generic Background	1
2	ACCEPTANCE CRITERIA	3
	2.1 Applicable Criteria	3
	2.2 Principal Acceptance Criteria	3
3	TECHNICAL REVIEW	6
	3.1 Mathematical Modeling and Seismic Analysis of Spent Fuel Rack Modules	6
	3.2 Evaluation of the Nonlinear Dynamic Displacement Analysis	11
	3.2.1 Description of the Model	11
	3.2.2 Frictional Force Between Rack Support Pads and the Pool Liner	14
	3.2.3 Hydrodynamic Coupling Between Fluid and Rack Structure	15
	3.2.4 Seismic Loading	16
	3.2.5 Integration Time Step	17
	3.2.6 Rack Displacements	17
	3.3 Evaluation of the Detailed Three-Dimensional Linear Model	19
	3.3.1 Description of the Model	19
	3.3.2 Review of Stress Levels	20
	3.4 Review of Spent Fuel Pool Structural Analysis	20
	3.4.1 Spent Fuel Pool Structural Analysis	20
	3.4.2 Analysis Procedure	23
	3.4.3 Summary of Results	27

CONTENTS

Section	Title	Page
1	INTRODUCTION	1
	1.1 Purpose of the Review	1
	1.2 Generic Background.	1
2	ACCEPTANCE CRITERIA.	3
	2.1 Applicable Criteria	3
	2.2 Principal Acceptance Criteria	3
3	TECHNICAL REVIEW	6
	3.1 Mathematical Modeling and Seismic Analysis of Spent Fuel Rack Modules	6
	3.2 Evaluation of the Nonlinear Dynamic Displacement Analysis	11
	3.2.1 Description of the Model	11
	3.2.2 Frictional Force Between Rack Support Pads and the Pool Liner	14
	3.2.3 Hydrodynamic Coupling Between Fluid and Rack Structure	15
	3.2.4 Seismic Loading	16
	3.2.5 Integration Time Step	17
	3.2.6 Rack Displacements	17
	3.3 Evaluation of the Detailed Three-Dimensional Linear Model .	19
	3.3.1 Description of the Model	19
	3.3.2 Review of Stress Levels	20
	3.4 Review of Spent Fuel Pool Structural Analysis	20
	3.4.1 Spent Fuel Pool Structural Analysis	20
	3.4.2 Analysis Procedure	23
	3.4.3 Summary of Results	27

CONTENTS (Cont.)

Section	Title	Page
	3.5 Fuel Handling Accident Analysis	27
	3.5.1 Fuel Handling Crane Uplift	27
	3.5.2 Accidental Fuel Assembly Drop	30
4	CONCLUSIONS	31
5	REFERENCES	32

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.

1. INTRODUCTION

1.1 PURPOSE OF THE REVIEW

This technical evaluation report (TER) covers an independent review of the Philadelphia Electric Company's licensing report [1] on high-density spent fuel racks for Peach Bottom Units 2 and 3 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 Peach Bottom Units 2 and 3 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, Division 1

- 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.

2.2 PRINCIPAL ACCEPTANCE CRITERIA

The principal acceptance criteria for the evaluation of the spent fuel racks' structural analysis for Peach Bottom Units 2 and 3 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.
>
> Other materials, design procedures, and fabrication techniques will be reviewed on a case-by-case basis."

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.
- 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.

---

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

** American Institute of Steel Construction, Latest Edition.

- 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

Submerged spent fuel rack modules exhibit highly nonlinear structural dynamic 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.

Peach Bottom Units 2 and 3 plan to utilize high density fuel racks comprising nine variations in storage capacity that are arranged in the spent fuel pools as shown in Figures 1 and 2 [1]. Data pertaining to the rack module designs are provided in Table 1. Note that the clearance space between the rack modules and the pool structure is shown in Figures 1 and 2 by the boxed dimensions. The minimum rack module to rack module clearance is 1.68 inches, as reported by the Licensee [3].

The rack modules for each unit ranged in capacity (and size) from 9 x 14 cells to 19 x 20 cells. These largest and smallest racks were chosen by the Licensee for structural dynamics analysis. Since experience indicates that, for a given rack height, the rack module with the smallest horizontal dimensions will usually yield the highest rack displacements (tipping), the Licensee's choice of modules for analysis is acceptable.

The seismic analysis was performed by the Licensee in two parts. The first part was a three-dimensional, nonlinear, time-history analysis of dynamic rack displacements employing a mathematical model of a spent fuel rack module, modeled as shown in Figure 3, to include the fuel assemblies and hydrodynamic

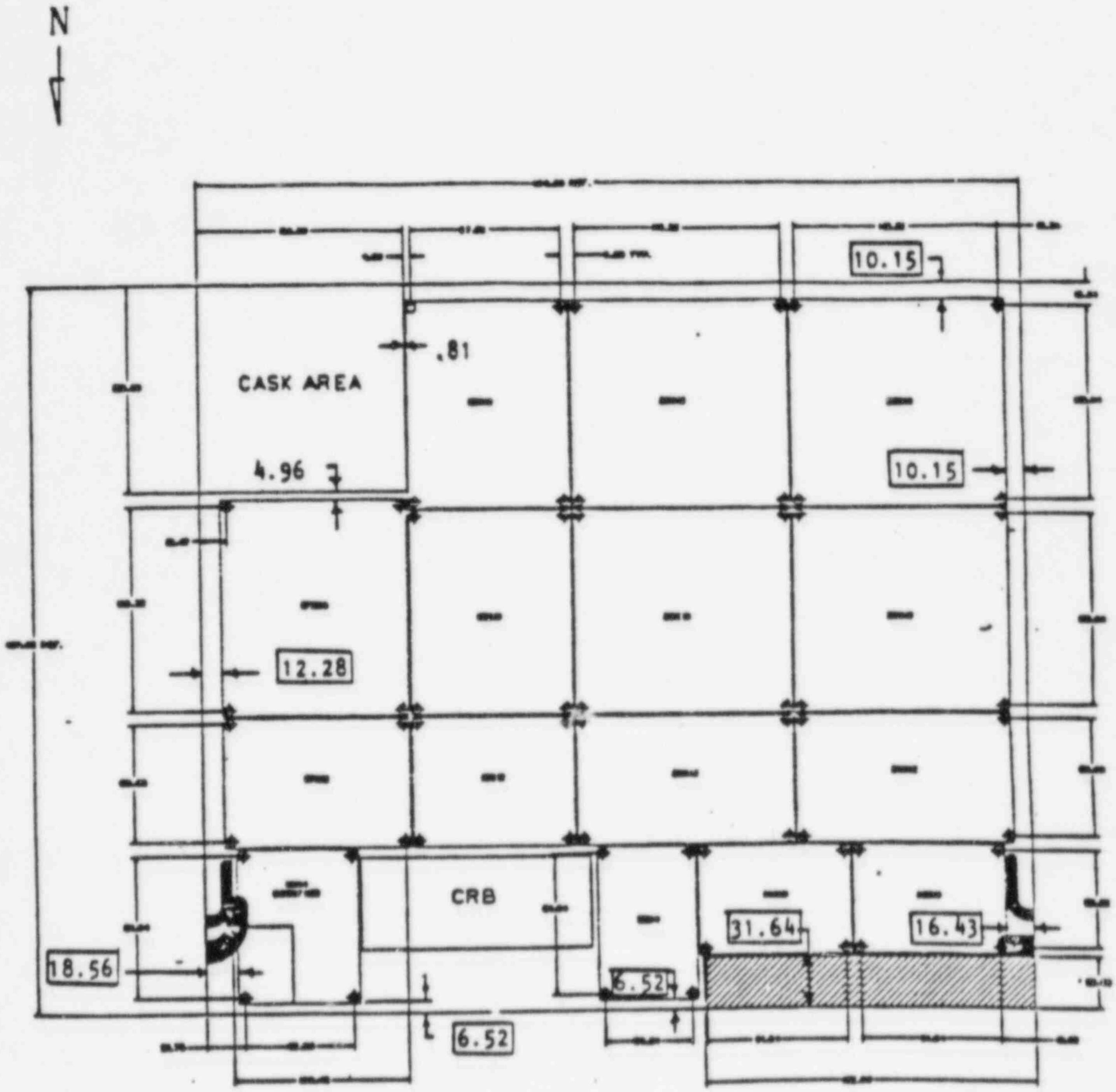

Figure 1. Spent Fuel Storage Rack Arrangement Unit 2

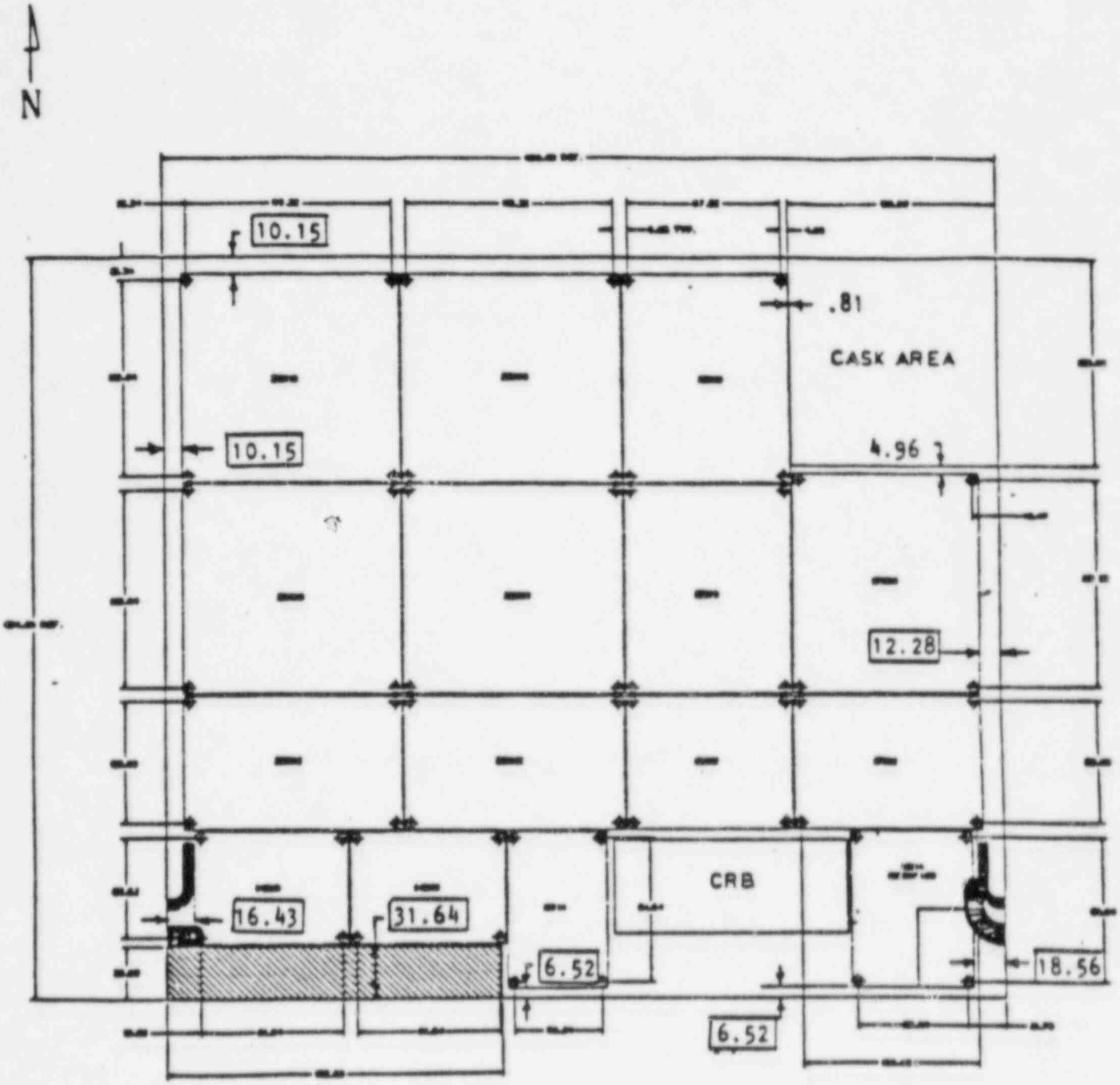

Figure 2. Spent Fuel Storage Rack Arrangement Unit 3

Table 1. Rack Module Data (Per Unit)

Qty	Array	Storage Locations	Rack Assembly Dimensions (inches)	Dry Weight (lb) Per Rack Assembly
1	9 x 14	126	54 x 89 x 180	10,000
2	10 x 14	280	64 x 89 x 180	11,200
1	11 x 14 Mod.	119	70 x 89 x 180	9,500
1	12 x 15	180	76 x 95 x 180	14,400
1	12 x 17	204	76 x 107 x 180	16,300
2	12 x 20	480	76 x 126 x 180	19,200
2	15 x 19	570	95 x 120 x 180	22,800
1	17 x 20	340	107 x 126 x 180	27,200
4	19 x 20	1,520	120 x 126 x 180	30,400
15 racks		3,819		

Storage locations center-to-center spacing (inches)	6.28
Storage cell inner dimension (inches)	6.07
Intermediate storage location inner dimensions (inches)	6.12
Type of fuel	BWR 8 x 8 BWR 8 x 8 (R) BWR 7 x 7

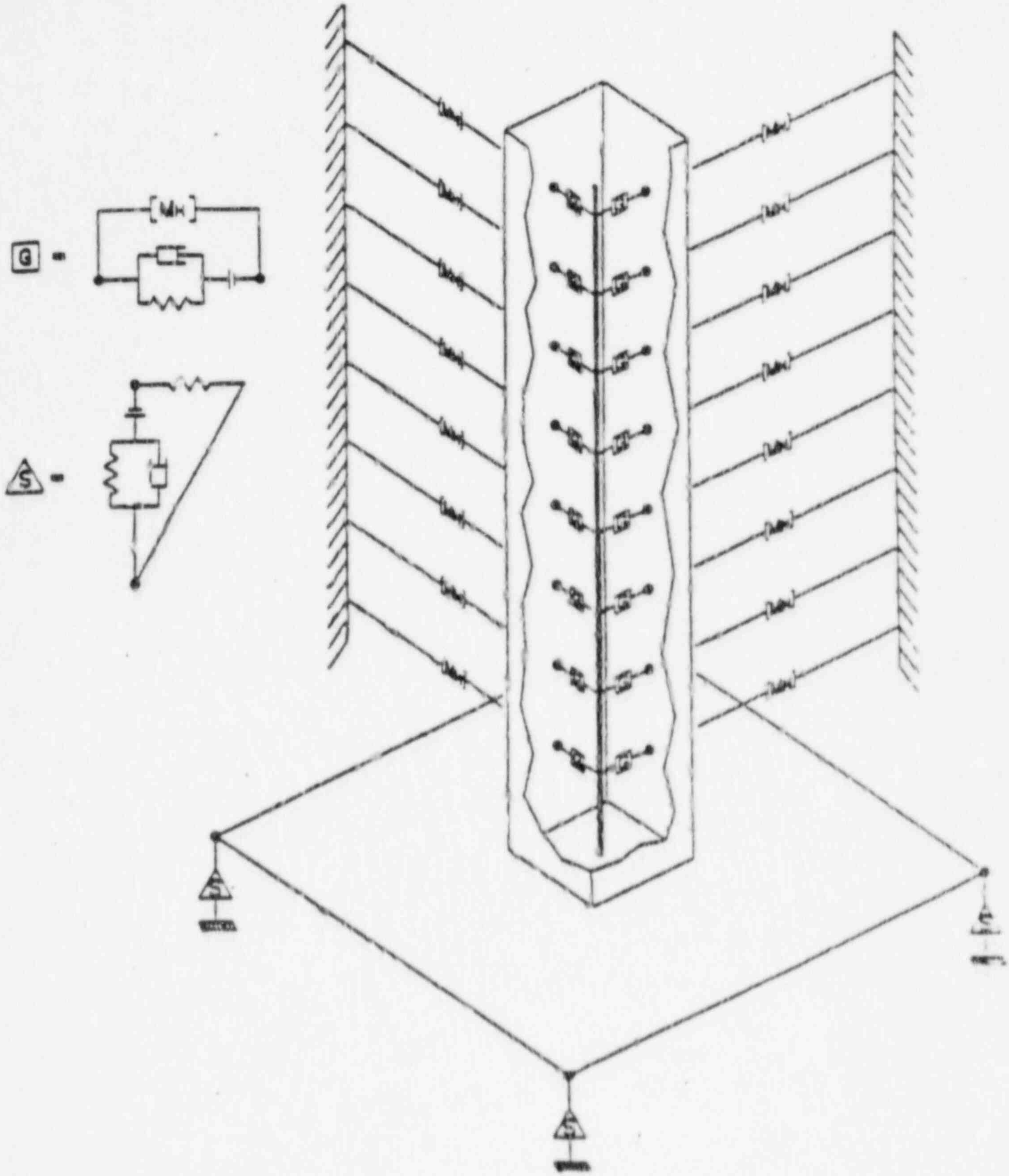

Figure 3. Three-Dimensional Nonlinear Seismic Model

coupling to other rack modules and/or the pool wall. The second part of the seismic analysis used a linear, three-dimensional, finite element model of the fuel rack, as shown in Figure 4, for the dual purposes of computing rack stresses and determining the rack module structural properties for use in the nonlinear dynamic displacement analysis.

The Licensee's seismic and stress analysis of the spent fuel rack modules considered full, partially filled, and empty rack modules.

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 NONLINEAR DYNAMIC DISPLACEMENT ANALYSIS

3.2.1 Description of the Model

The Licensee performed seismic displacement analyses of the free-standing fuel rack modules with the use of the Westinghouse Electric Computer Analysis (WECAN) Code [1]. The analysis was performed as a time-history analysis using the three-dimensional mathematical model shown in Figures 3 and 5, with simultaneous application of three orthogonal, independent, acceleration time-histories (two horizontal and one vertical).

The effective structural properties of the single cell model shown in Figure 3 were modeled by three-dimensional beam elements and were derived from linear three-dimensional analysis of the fuel rack to which the hydrodynamic mass of the water was added. The fuel assembly, modeled by beam elements and represented in Figure 3 by the heavy vertical line, was connected to the cell walls through springs, dampers, gap elements, and hydrodynamic mass of the water in the cell. This model enabled the simulation of fuel assembly motion in the clearance space between the fuel assembly and the rack cell walls, as well as impact with the cell walls.

Hydrodynamic mass coupling of the rack module to adjacent rack modules and to the spent fuel pool walls is shown in Figures 3 and 5, and is discussed in Section 3.2.3.

The Licensee provided the following description of the modeling of support pads [1]:

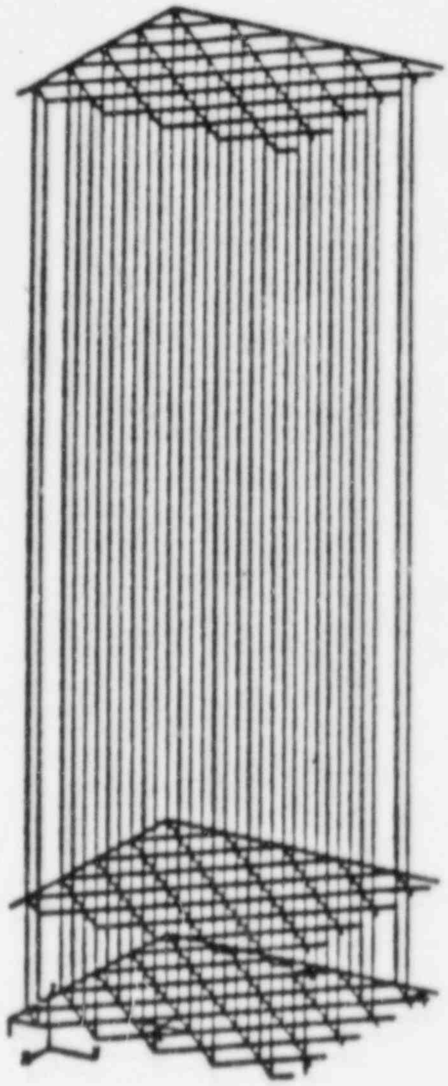

Figure 4. Structural Model (Quarter Rack)

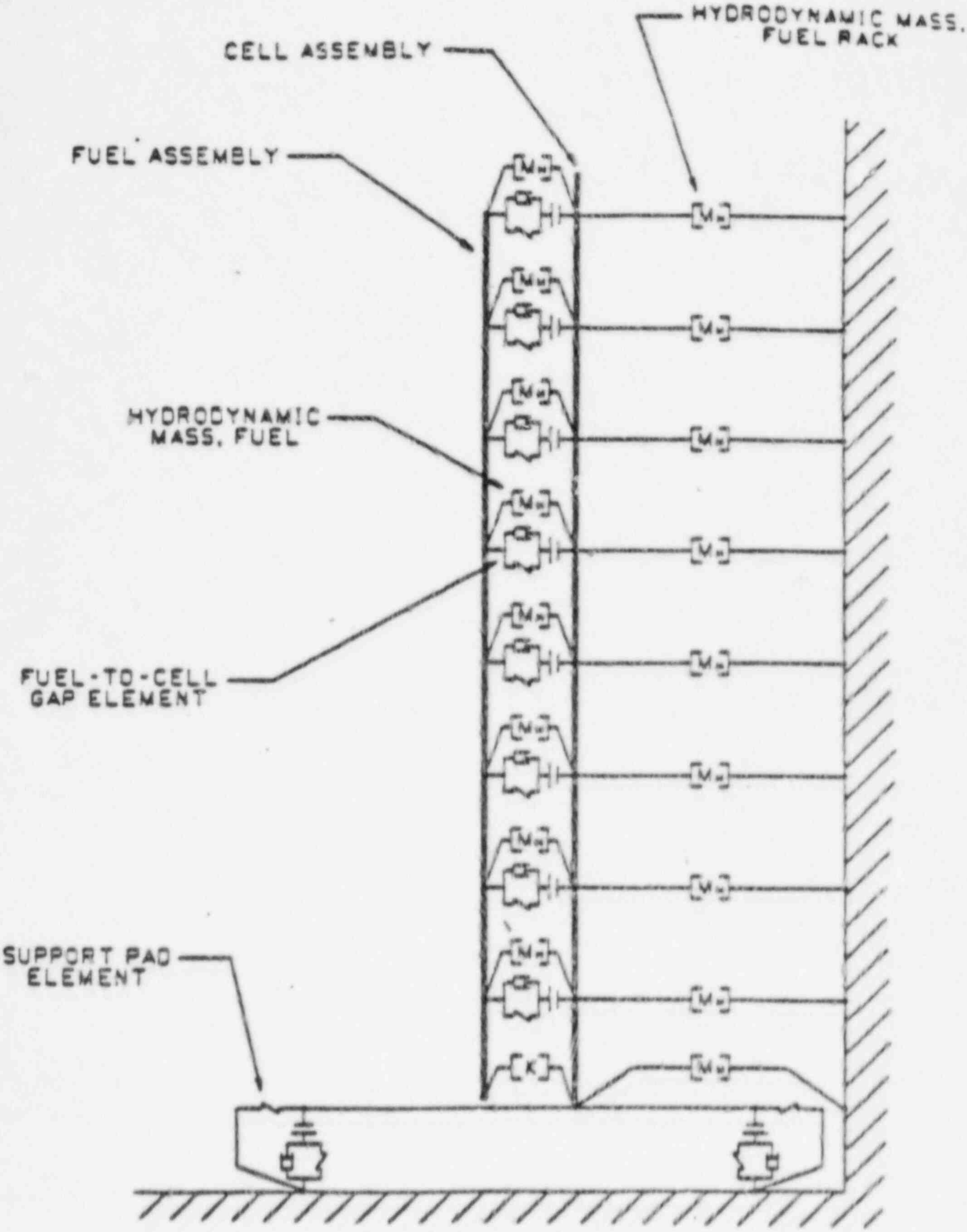

Figure 5. Section of Three-Dimensional Nonlinear Seismic Model

"The support pads are modeled by a combination of three-dimensional dynamic friction elements connected by a "rigid" base beam arrangement which produces the spacing of support pads. The cell and fuel assemblies are located in the center of the base beam assembly and form a model which represents the rocking and sliding characteristics of a rack module in both directions on a plane. Vertical grounded springs at the support pad locations are used to model and account for the interaction between the racks and the spent fuel pool structure. The friction elements are capable of reversing the direction of the restraining force when sliding changes direction."

Structural damping used in the analysis, with the exception of damping unique to fuel assembly impact, was 2% for the OBE event and 5% for SSE. Added damping due to submergence in the pool water was not considered.

Damping of the impact between the limber fuel assemblies and the walls of the storage cells requires consideration beyond that of usual structural damping. In response to a request for additional information, the Licensee provided the following [3]:

"Impact damping between the fuel assembly and the rack cell was included in the analysis. A damping ratio of 0.04 was used for both the top and bottom fittings of the fuel assembly and is a conservative value for impact damping of rigid structures since higher damping ratios are used in the seismic analysis for the reactor vessel and piping supports.

For the intermediate fuel grid assemblies a damping ratio of 0.25 was used. The grid assembly is a flexible structure with frictional connections at the fuel rods which produces large impact damping values. A review of GE fuel information by the Westinghouse Nuclear Fuel Division has determined that a grid assembly damping ratio of 0.25 is appropriate. This damping value is consistent with the grid damping ratio that has been determined for Westinghouse fuel by tests performed by the Westinghouse Nuclear Fuel Division using a fuel assembly in air impacting on a rigid surface."

The Licensee's modeling of the rack modules and use of fuel assembly impact damping is acceptable.

3.2.2 Frictional Force Between Rack Support Pads and the Pool Liner

The Licensee used a maximum value of 0.8 and a minimum value of 0.2 for the range of static friction coefficient between the rack support pads and the pool liner [1]. Rabinowicz, in a report to the General Electric Company [4], focused attention on the mean and the lowest coefficient of friction to be used in these circumstances. While Rabinowicz supported the range of static coefficient used by the Licensee, he also indicated that the dynamic, or sliding, coefficient of friction is inversely proportional to velocity. The Licensee did not indicate whether the analysis used an initial static coefficient of friction and a lower dynamic coefficient of friction once sliding motion began. While the use of a lower dynamic coefficient of friction may have yielded somewhat larger sliding displacements, the Licensee's computed sliding displacement was sufficiently small to dismiss further consideration of dynamic coefficients of friction. Thus, the Licensee's use of friction coefficient between the support pads and the pool liner is acceptable.

3.2.3 Hydrodynamic Coupling Between Fluid and Cell Structure

Hydrodynamic coupling acts between adjacent rack modules, between a rack module and the pool walls, and between fuel assemblies and the cells in which they are inserted. Hydrodynamic coupling can have a significant effect upon the dynamic response of a rack module during seismic events.

In response to a request for additional information, the Licensee indicated that the motion of adjacent racks may be out of phase or unrelated [3]. 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. 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 [5].

Fritz's [5] 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 [5] 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 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 [5] modeling technique for hydrodynamic coupling of rack modules adjacent to other rack modules or a pool wall indicate that the Licensee's fuel rack dynamic model should be considered conservative only for dynamic displacements that are small relative to the available displacement clearance.

3.2.4 Seismic Loading

The Licensee indicated that the earthquake loading was predicated upon an operating basis earthquake (OBE) at the site having a horizontal ground acceleration of 0.05 g, and that a safe shutdown earthquake (SSE) with a horizontal ground acceleration of 0.12 g was used to check the design to assure no loss of function [1]. The Licensee indicated further that these OBE and SSE designations correspond to FSAR designations of design earthquake (DE) and maximum credible earthquake (MCE), respectively [1].

In response to a request for additional information, the Licensee described the procedure used to determine the two orthogonal horizontal and one vertical simulated earthquake acceleration time-histories as follows [3]:

> "Simulated earthquake acceleration time histories in two orthogonal horizontal directions were generated from the Reactor Building seismic response spectra at the spent fuel pool floor evaluation using the SIMQKE* computer program. The results were evaluated to ensure that statistical independence was achieved and that the resulting response spectra adequately enveloped the original Reactor Building floor response spectra.
>
> The two horizontal acceleration time histories are generated from a single seismic floor response spectra which represented the worst case for the structure. Therefore, seismic analyses of the fuel racks are conservatively based on the worst case horizontal seismic loading applied in both horizontal directions simultaneously."

*SIMQKE, A program for Artificial Motion Generation, User's Manual and Documentation, Department of Civil Engineering, Massachusetts Institute of Technology, November 1976.

The Licensee has stated further that one of the two orthogonal, horizontal, acceleration time-histories was directed across the short dimension of the rack module in the analysis of the 9 x 14 cell rack module [6].

Evaluation indicated that the Licensee's development and application of simulated acceleration time-histories is acceptable.

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 [3]. Solutions using different time steps showed that the results were the same for time increments of 0.0025 sec and 0.00125 sec. The Licensee then performed the final analysis using a time step of 0.0025 sec.

3.2.6 Rack Displacements

The Licensee's analysis indicated that the maximum sliding displacement occurred with the minimum friction coefficient of 0.2, whereas the maximum rack displacement at the top of the rack due to bending and tipping occurred with the maximum friction coefficient of 0.8 [3].

The Licensee also noted that the maximum rack module displacements occurred for full racks and that the displacement of the 9 x 14 cell rack module in the 9-cell direction was the largest [3]. These largest displacements are presented in Table 2.

Maximum liftoff of a support pad from the pool liner was reported by the Licensee to be 0.0129 inch under the SSE event, and to occur on the 9 x 14 cell rack in the 9-cell direction [3].

The maximum computed displacements due to sliding, elastic deformation, and tipping are shown in Table 2, which provides the data supplied with the Licensee's response [3] to a request for additional information.

It is noted in Table 2 that each occurrence of sliding is relatively small with the sum of five OBE occurrences amounting to 0.049 inch.

Table 2. Rack Displacements: SSE Seismic + Maximum Normal Thermal

			SSE Seismic + Normal Thermal Displacements	
	Symbol	Units	Rack Top	Rack Base
Max. Sliding Distance, $\mu = 0.2$ $\Delta_s = (0.0098)5$*	Δ_s	in	0.049	0.049
Max. Structural Defl., $\mu = 0.8$	δ	in	0.647	0.0
Total Displacement One Rack $\Delta = \Delta_s + \delta$	Δ	in	0.696	0.049
SRSS Combined Displacement 2 Racks with Only 1 Sliding $\Delta_{max} = \Delta 2 + \delta 2$	Δ_{max}	in	0.950	0.049
Max. Normal Thermal Displacement	δ_T	in	0.087	0.087
Max. Combined Thermal & Seismic Displacements $\Delta = \delta_T + \Delta_{max}$	Δ	in	1.037	0.136
Nominal Rack to Rack Gap	Δ	in	1.68	1.03

*This accounts for five OBE events.

Maximum structural deflection at the top of the rack was reported to be 0.647 inch which, when combined with accumulated sliding, yielded 0.696 inch [3]. For the case of adjacent dissimilar rack modules whose responses may be out of phase, the Licensee combined the displacement of the two rack modules by the square root of the sum of the squares to yield a combined displacement of 0.950 inch. After including the maximum normal thermal growth, the Licensee compared the maximum combined displacement of 1.037 inches to the installed clearance of 1.68 inches between racks (shown in Table 2). With the combined displacement of the two adjacent rack modules less than the available clearance space, the Licensee indicated that impact of the racks would not occur and that impact analysis of the rack modules is not necessary.

While the use of the square root of the sum of the squares is a reasonable approach to combining out-of-phase displacements of adjacent rack modules for comparison to the available clearance space, the worst possible case is that of direct summation of the rack's displacement. This worst case would represent the point in time when the responses are 180 degrees out of phase. Thus, using the Licensee's displacement data as shown in Table 2, it can be seen that even the direct sum of two total displacements is less than the clearance space of 1.68 inches. Note that the clearance space between the rack modules and pool structure, as shown by the boxed dimensions in Figures 1 and 2, is much larger.

The evaluation of the Licensee's computed maximum displacements and their comparison with the installed clearance space indicated that they are acceptable, and that rack module impacts with other rack modules and the pool structure is unlikely.

3.3 EVALUATION OF THE DETAILED THREE-DIMENSIONAL LINEAR MODEL

3.3.1 Description of the Model

The Licensee used a finite element model of the rack module to determine the stresses in the module. The Licensee's description of the procedure follows [1]:

"The structural model, shown in [Figure 4], is a quarter section representation of the rack assembly consisting of beam elements interconnected at a finite number of nodal points and general mass matrix elements. The beam elements model the beam action of the cell, the stiffening effect of the cell to cell welds, and the supporting effect of the support pads. The general mass matrix elements represent the hydrodynamic mass of the rack module. The beams which represent the cells are loaded with equivalent seismic loads and the model produces the structural displacements and internal load distributions necessary to calculate the effective structural properties of an average cell within the rack module. In addition to the stiffness properties, the internal load and stress distributions of this model are used to calculate stress peaking factors to account for the load gradients within the rack module."

The results of the seismic displacement analyses were searched throughout the full analysis time to obtain the maximum response forces. These maximum values were then adjusted by peaking factors from the structural model to account for stress gradients through the rack module [1].

Load combinations and acceptance stress limits used in the Licensee's stress analysis were in accordance with the NRC's OT Position Paper [2] and are shown in Table 3. The Licensee's computed stresses, allowable stresses, and safety margins are shown in Table 4 [1]. Note that the safety margins, computed in accordance with the following formula, are all greater than zero, thereby indicating acceptable conditions:

$$\text{Safety Margin} = \frac{\text{Allowable Stress}}{\text{Design Stress}} - 1$$

3.3.2 Review of Stress Levels

Evaluation of the rack module stresses indicated that the analysis, level of stresses, and acceptability criteria are satisfactory.

3.4 REVIEW OF SPENT FUEL POOL STRUCTURAL ANALYSIS

3.4.1 Spent Fuel Pool Structural Analysis

The spent fuel pool (SFP) structure was analyzed using linear and nonlinear finite element models to determine the maximum allowable fuel rack loads that could be imposed on the pool slab.

Table 3. Storage Rack Loads and Load Combinations

Load Combination	Acceptance Limit
D + L	Normal limits of NF 3231.1a
D + L + P_f	Normal limits of NF 3231.1a
D + L + E	Normal limits of NF 3231.1a
D + L + T_o	Lesser of $2S_y$ or S_u stress range
D + L + T_o + E	Lesser of $2S_y$ or S_u stress range
D + L + T_a + E	Lesser of $2S_y$ or S_u stress range
D + L + T_o + P_f	Lesser of $2S_y$ or S_u stress range
D + L + T_a + E'	Faulted condition limits of NF 3231.1c (See Note 3)
D + L + F_d	The functional capability of the fuel racks shall be demonstrated

Notes:

1. The abbreviations in the table above are those used in Standard Review Plan (SRP) Section 3.8.4 where each term is defined except for T_a, which is defined here as the highest temperature associated with the postulated abnormal design conditions. F_d is the force caused by the accidental drop of the heaviest load from the maximum possible height, and P_f is the upward force on the racks caused by a postulated stuck fuel assembly.

2. The provisions of NF-3231.1 of ASME Section III, Division I, shall be amended by the requirements of Paragraphs c.2, 3, and 4 of Regulatory Guide 1.124, entitled "Design Limits and Load Combinations For Class A Linear-Type Component Supports."

3. For the faulted load combination, thermal loads were neglected when they are secondary and self-limiting in nature and the material is ductile.

Table 4. Summary of Design Stresses and Minimum Margins of Safety
Normal and Upset Conditions

	Design Stress (psi)	Allowable Stress (psi)	Margin of Safety
1. Support Pad Assembly			
1.1 Support Pad			
Shear	1595	11000	5.90
Axial and Bending	10479	16500	.57
Bearing	13645	27500*	1.02
1.2 Support Pad Screw			
Shear	7958	11000	.38
1.3 Support Structure			
Axial and Bending	17626	27500*	.56
Shear	1233	11000	7.92
Weld Shear	19072	275000*	.44
2.0 Cell Assembly			
2.1 Cell			
Axial and Bending	.816	1.0**	.23
2.2 Cell to Base Plate Weld			
Weld Shear	19082	24000	.26
2.3 Cell to Cell Weld			
Weld Shear	16286	21000	.29
Pin Shear	7384	9260	.25
2.4 Cell to Wrapper Weld			
Weld Shear	8300	11000	.33
2.5 Cell Seam Weld			
Weld Shear	3501	4516***	.29
2.6 Cell to Cover Plate Welds			
Weld Shear	11854	24000	1.03

* Thermal Plus OBE Stress is Limiting
** Allowable per Appendix XVII -2215 Eq (24)
*** Design Load and Allowable Load in Lbs is shown

Loading combinations required by USNRC Regulatory Guide 1.142, USNRC Standard Review Plan 3.8.4, the American Concrete Institute, and the American Institute of Steel Construction were satisfied. These were consolidated into the set of load combination requirements shown in Table 5, and were satisfied using strength design methods for the concrete structures and plastic design methods for structural steel [1].

Thermal loads were based on pool water temperatures of 150°F resulting from a full core discharge under normal operating conditions, and saturation temperatures for accident conditions varying from 250°F at the bottom of the pool to 212°F at the free water surface. A conservative ambient air temperature of 68°F was used. A stress free-temperature of 70°F was assumed.

3.4.2 Analysis Procedures

3.4.2.1 Method of Analysis

The Licensee employed the MSC/NASTRAN general purpose finite element program to investigate the spent fuel pool structure, using a three-dimensional finite model that included the entire spent fuel pool structure as well as adjacent key structural members. The model is shown in Figure 6. The Licensee provided the following additional features of the model [1]:

> "Floor slabs and walls immediately adjacent to the SFP are modeled to simulate the proper lateral restraint on the pool structure. Complete fixity against translation and rotation is assumed at the base of the drywell shield wall. Cut-off boundaries of adjoining walls and slabs were restrained with translational springs. These springs permit the model to simulate the cantilever mode deflected shape of the Reactor Building under horizontal seismic loading. Translational springs simulate lateral stiffness of the remainder of the Reactor Building walls which were not included in the model. In-plane rotations of all interior grid points on slabs and walls are restrained."

The overall model was estimated to contain 11,000 independent degrees of freedom [1].

While this was a linear mathematical model, the Licensee applied the external loads in increments to perform a piecewise linear solution to the nonlinear problem of cracking in the concrete under tensile stresses. Checking of the computed stresses against the concrete cracking criterion and

Table 5. Spent Fuel Pool Governing Design Load Combinations

Reinforced Concrete

1. $U = 1.4D + 1.4F + 1.7T_o$
2. $U = 1.4D + 1.4F$
3. $U = 1.4D + 1.4F + 1.7L + 1.9E$
4. $U = D + F + L + E' + T_a$
5. $U = D + F + L + E'$
6. $U = 1.05D + 1.05F + 1.3L + 1.43E + 1.3T_o$

Structural Steel

7. $Y = 1.7D + 1.7F + 1.7L + 1.7E$
8. $Y = 1.3D + 1.3F + 1.3L + 1.3E + 1.3T_o$
9. $Y = 1.1 (D + F + L + E' + T_a)$

Notation:

D = dead load

E = OBE (design earthquake)

E'= SSE (maximum credible earthquake)

L = live load

T_a= thermal load produced by accident condition

T_o= thermal load during normal operation

U = section strength required to design loads based on the Strength Design method for reinforced concrete

Y = section strength required to resist design loads based on Plastic Design method for structural steel

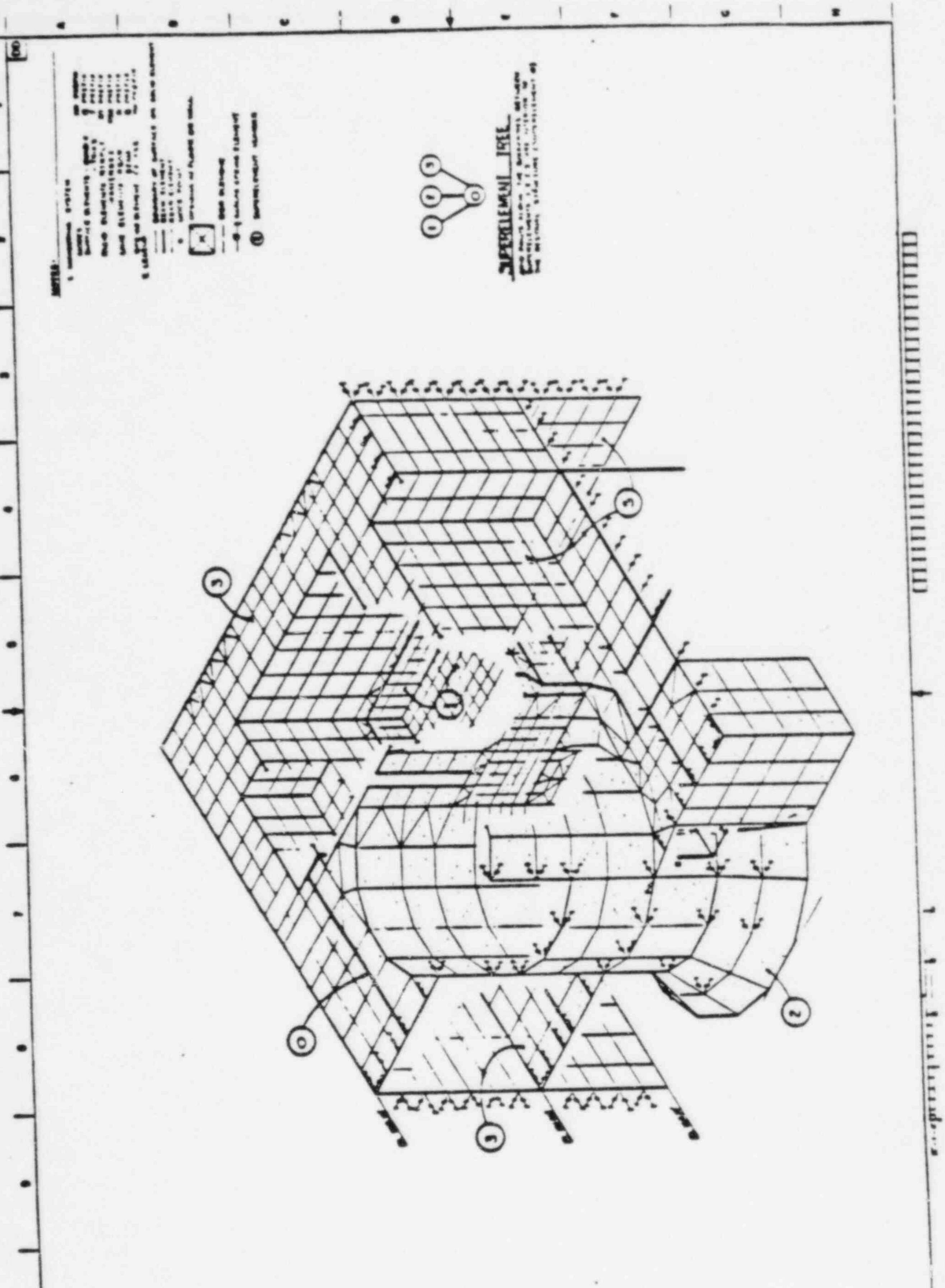

Figure 6. Reactor Building - Finite Element Model

the adjustment of material properties to reflect crack development was reported to have been performed manually at the end of each iteration. Thus, each new iteration was begun using the accumulated load that included the new load increment as well as stiffness properties reflecting crack development to that point.

Cracking criteria were applied primarily to the elements comprising the pool slab and lower portions of the pool walls. Application of the cracking criteria was carried out by comparing the local orthogonal tensile stresses against the modulus of rupture and adjusting the respective elastic modulus to reflect crack development.

The critical section for slab shear and bending was taken at the face of the walls in accordance with ACI Code provisions. The critical section in the wall was taken on the horizontal plane at the top of the slab elevation [1]. Shear capacities of the steel beams and connections were determined in accordance with Part 2 of the AISC specifications for plastic design.

With respect to thermal moment relaxation of local areas away from the pool slab, the approach used for the investigation was, in accordance with ACI 349 Appendix A, to assume the structure is uncracked for mechanical loads and cracked for thermal loads.

3.4.2.1 Supporting Analysis

In addition to the piecewise linear analysis described above, the Licensee performed a nonlinear finite element analysis of a simplified pool slab structure to provide an estimate of the pool slabs' ultimate load carrying capacity. The pool slab was modeled using the ADINA finite element program by which it was possible to compute the collapse load of the slab considering the beneficial effects of arching [1].

The Licensee reported that the nonlinear analysis indicated no reinforcement yielding and very little concrete cracking at the design load.

The Licensee halted the nonlinear analysis when the applied load approached three times the factored design load. At this point, the analysis indicated that some cracking at supports and at midspan would occur, that the top bar at supports would yield, but that collapse was not imminent [1].

3.4.3 Results of the Analysis

The Licensee reported the following [1]:

- "Reduced transverse shear capacity was used in the pool slab to reflect the small amount of membrane tension generated by the lateral fluid pressure on the pool walls. This shear capacity was compared against peak transverse shear forces from the MSC/NASTRAN finite element analysis results and is adequate."

- "The load transfer capacity of the wall/slab joints on the East and West sides of the pool were evaluated and found to be adequate."

- "Additional shear stresses due to increased spent fuel storage capacity are calculated to be 0.0020 kip/in^2 and 0.0032 kip/in^2 at EL. 180'-0" for OBE and SSE respectively. These shear stress increments are based on the MSC/NASTRAN finite element analysis results. These increments represent increases in total shear stresses from 89 percent to 92 percent of the allowable for OBE and from 69 percent to 70 percent for SSE. The resulting total concrete shear stresses are less than the allowable shear stresses."

- "Local areas of the North exterior wall of the Reactor Building were also evaluated due to the increased loads. The areas checked are the support points of the East and West walls of SFP. These areas are adequate for combined axial load and bending. Shear forces are also less than the shear capacity."

The Licensee's maximum allowable fuel rack/pool floor interface loads and stresses are reproduced in Table 6. The Licensee's comparison of the pool floor interface loads and stresses with allowable values is shown in Table 7.

Evaluation of the spent fuel pool analysis indicated that the analysis is satisfactory and that the spent fuel pool structure is adequate for the increased density of fuel storage.

3.5 FUEL HANDLING ACCIDENT ANALYSIS

3.5.1 Fuel Handing Crane Uplift

The Licensee provided the following with respect to crane uplift of a fuel assembly [1]:

"The objective of this analysis is to ensure that the rack can withstand the maximum uplift load of 4,000 pounds and a horizontal force of 1,000 pounds of the fuel handling crane without violating the critically acceptance criterion. The maximum uplift load is approximately two times

Table 6. Maximum Allowable Fuel Rack/Pool Floor Interface Loads

NO.	LOAD COMBINATION	TOTAL LOADS VERTICAL (KIP)	HORIZONTAL (KIP)	LOCAL BEARING (KSI)
1.	D + L	3,900.0[1]	N/A	2.4
2.	D + L + T_o	3,900.0[1]	N/A	2.4
3.	D + L + T_o + E	5,700.0	1,900.0	2.4
4.	D + L + T_a + E	5,700.0	1,900.0	2.4
5.	D + L + T_o + P_f	5,700.0	N/A	3.2
6.	D + L + T_a + E'	8,000.0	3,000.0	3.2
7.	D + L + F_d	8,000.0	N/A	4.76
	Alternate[1]			
8.	1.4 (D + L + T_o) + 1.9E	8,900.0	3,600.0	See Note 2
9.	1.4 (D + L + T_a) + 1.9E	8,900.0	3,600.0	See Note 2
10.	1.7 (D + L + T_o + E)	9,700.0	3,200.0	See Note 2
11.	1.7 (D + L + T_a + E)	9,700.0	3,200.0	See Note 2

Notes:

1. Additional structural limits specified in Load Combination No. 8, 9, 10, and 11 shall be satisfied if total vertical loads calculated for Load Combination No. 1 and 2 are less than 3,700.0 kip. Otherwise, Load Combination No. 8, 9, 10, and 11 may be used in lieu of Load Combination No. 1, 2, 3, 4, and 5.

2. When total loads are evaluated using Load Combination No. 8, 9, 10, and 11, local bearing pressures shall satisfy Load Combination No. 1, 2, 3, 4, and 5.

3. Notations used in this table are the same as defined in SRP 3.8.4, Appendix D.

Table 7. Pool Floor Loads

Load Combination	Condition*	Design Stress or Load	Allowable Stress or Load	Margin of Safety
1. D + L	Local Bearing	1.76	2.4	.36
2. D + L + To	Local Bearing	1.76	2.4	.36
3. D + L + To + E	Local Bearing	1.94	2.4	.24
4. D + L + Ta + E	Local Bearing	1.94	2.4	.24
5. D + L + To + Pf	Local Bearing	1.76	3.2	.82
6. D + L + Ta + E'	Vertical	6180	8000	.29
	Horizontal	1670	3000	.80
	Local Bearing	2.63	3.2	.22
7. D + L + Fd	Vertical	4130	8000	.94
	Local Bearing	4.39	4.76	.08
8. 1.4(D + L + To) + 1.9E	Vertical	7730	8900	.15
	Horizontal	1590	3600	1.26
9. 1.4(D + L + Ta) + 1.9E	Vertical	7730	8900	.15
	Horizontal	1590	3600	1.26
10. 1.7(D + L + To + E)	Vertical	8760	9700	.11
	Horizontal	1420	3200	1.25
11. 1.7(D + L + Ta + E)	Vertical	8760	9700	.11
	Horizontal	1420	3200	1.25

*Vertical refers to total pool floor vertical load in kips. Horizontal refers to total pool floor horizontal load in kips. Local bearing refers to pool floor bearing stress under the highest loaded support pad in ksi.

the capacity of the fuel handling crane. In this analysis the loads are assumed to be applied to a fuel cell. Resulting stresses are within acceptable stress limits, and there is no change in rack geometry of a magnitude which causes the criticality acceptance criterion to be violated."

3.5.2 Accidental Fuel Assembly Drop

The Licensee provided the following [1]:

"Three accident conditions are postulated. The first accident condition assumes that the weight of a fuel assembly and handling tool impacts the top end fitting of a stored fuel assembly or the top of a storage cell from a conservative drop height of 2 feet in a straight attitude. The second accident condition is similar to the first except the impacting mass is at an inclined attitude. The impact energy is absorbed by the dropped fuel assembly, the stored fuel assembly, the cells and the rack base plate assembly. Under these faulted conditions the criticality acceptance criterion is not violated and the pool liner is not perforated. The third accident condition assumes that the dropped assembly falls straight through any empty cell and impacts the rack base plate from a conservative drop height of 2 feet above the top of the rack. The results of this analysis show that the impact energy is absorbed by the fuel assembly and the rack base plate. The spent fuel pool liner is not perforated. Criticality calculations show the k_{eff} <0.95 and the criticality acceptance criterion is not violated.

In each of these accident conditions, the criticality acceptance criterion is not violated and the spent fuel pool liner is not perforated."

4. CONCLUSIONS

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

- The Licensee used three-dimensional, nonlinear dynamic displacement analyses with three simultaneous, independent, orthogonal, earthquake acceleration time histories to provide greater resolution of the rack module displacements than is possible with two-dimensional analyses combined by the square root of the sum of the squares method.

- 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 experimentally verified results only for displacements which are small compared with the available clearance space. While the Licensee's reported rack module displacements are not small relative to the clearance space, the techniques used are acceptable in association with the conservative assumptions employed.

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

5. REFERENCES

1. Philadelphia Electric Company
   Safety Analysis Report for Design and Installation of New High Density Spent Fuel Racks, July 1985

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

3. Philadelphia Electric Company
   Response to a Request for Additional Information, October 9, 1985

4. E. Rabinowicz
   "Friction Coefficient Value for a High-Density Fuel Storage System"
   Report to General Electric Nuclear Energy Programs Division November 23, 1978

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

6. Philadelphia Electric Company (Shields L. Daltroff), Letter to U.S. Nuclear Regulatory Commission (Daniel R. Muller), Dated December 26, 1985, Submitting Added Response to a Request for Additional Information