TECHNICAL EVALUATION REPORT

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EVALUATION OF SPENT FUEL RACKS STRUCTURAL ANALYSIS

NORTHEAST UTILITIES
MILLSTONE NUCLEAR POWER STATION UNIT 2

TER-C5506-586

Prepared for

Nuclear Regulatory Commission
Washington, D.C. 20555

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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 Northeast Utilities' licensing report [1] on high-density spent fuel racks for Millstone Nuclear Power Station Unit 2, 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 Millstone Nuclear Power Station Unit 2 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

  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 Millstone Nuclear Power Station Unit 2 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.

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* 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.

The Licensee plans to use high density fuels racks arranged in the spent fuel pool of Millstone Unit 2 as shown in Figure 1 [1]. Two regions within the spent fuel pool are planned. Region I is to accept high-enrichment core off-load spent fuel in five rack modules fitted with neutron-absorbing inserts as shown in Figure 2. Similar rack modules, but without the neutron-absorbing inserts, are planned for Region II. The Region II rack modules are shown in Figure 3. Minimum clearance between adjacent rack modules is 2.0 inches. Minimum clearance between a rack module and the spent fuel pool wall is 4 13/16 inches [1].

The five rack modules of Region I include cell capacities and configurations of 8 x 9 and 8 x 10 cells. Region II is comprised of 14 rack modules of the following cell configurations: 7 x 8, 7 x 9, 7 x 10, 7 x 11, and 8 x 10. Of these configurations, the Licensee analyzed the following rack modules under seismic loadings [3]:

<u>Region I</u>	<u>Region II</u>
8 x 10 module	7 x 8 module 7 x 9 module 7 x 9 module (modified)

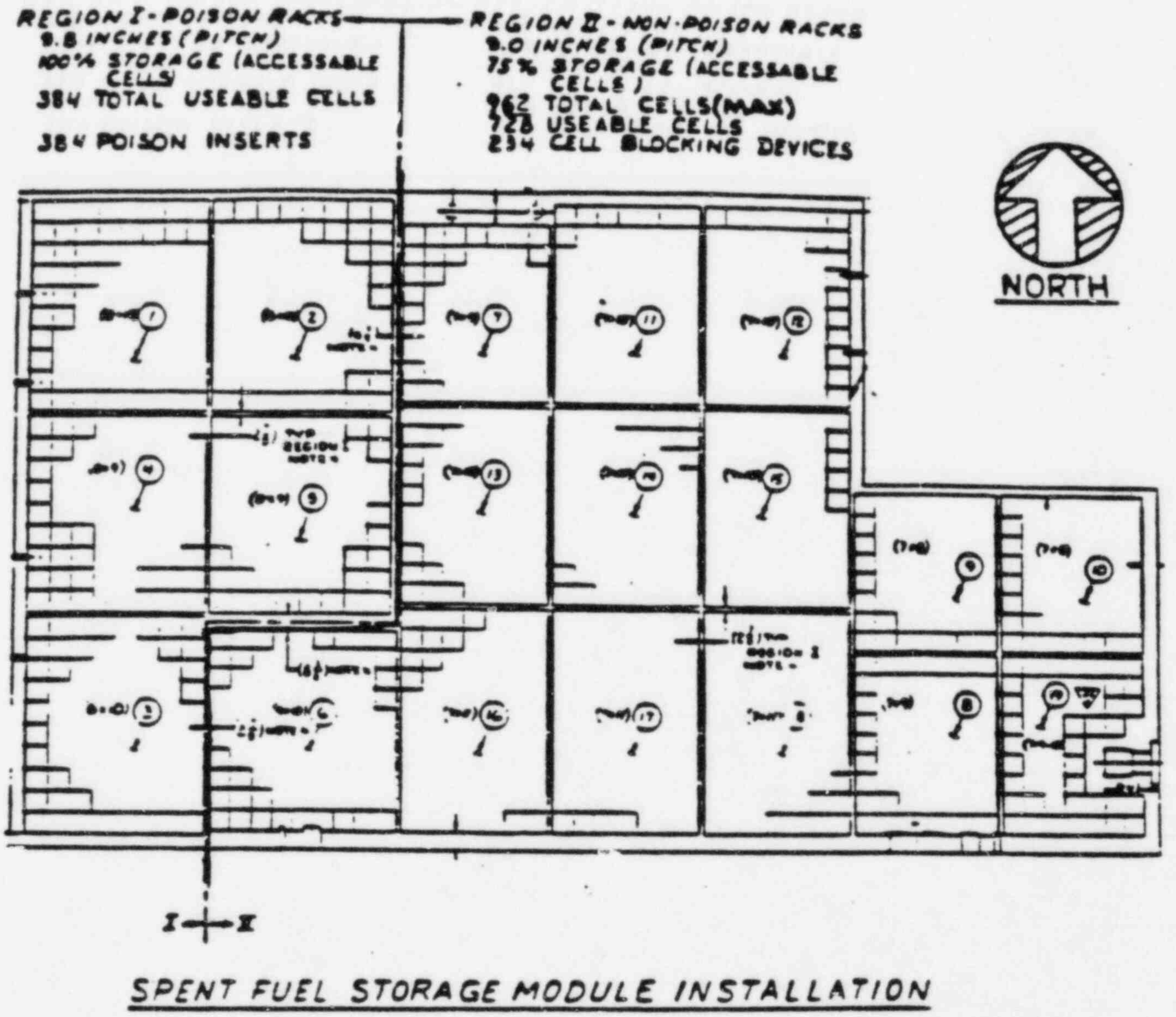

Figure 1. Fuel Pool Arrangement: Two Region

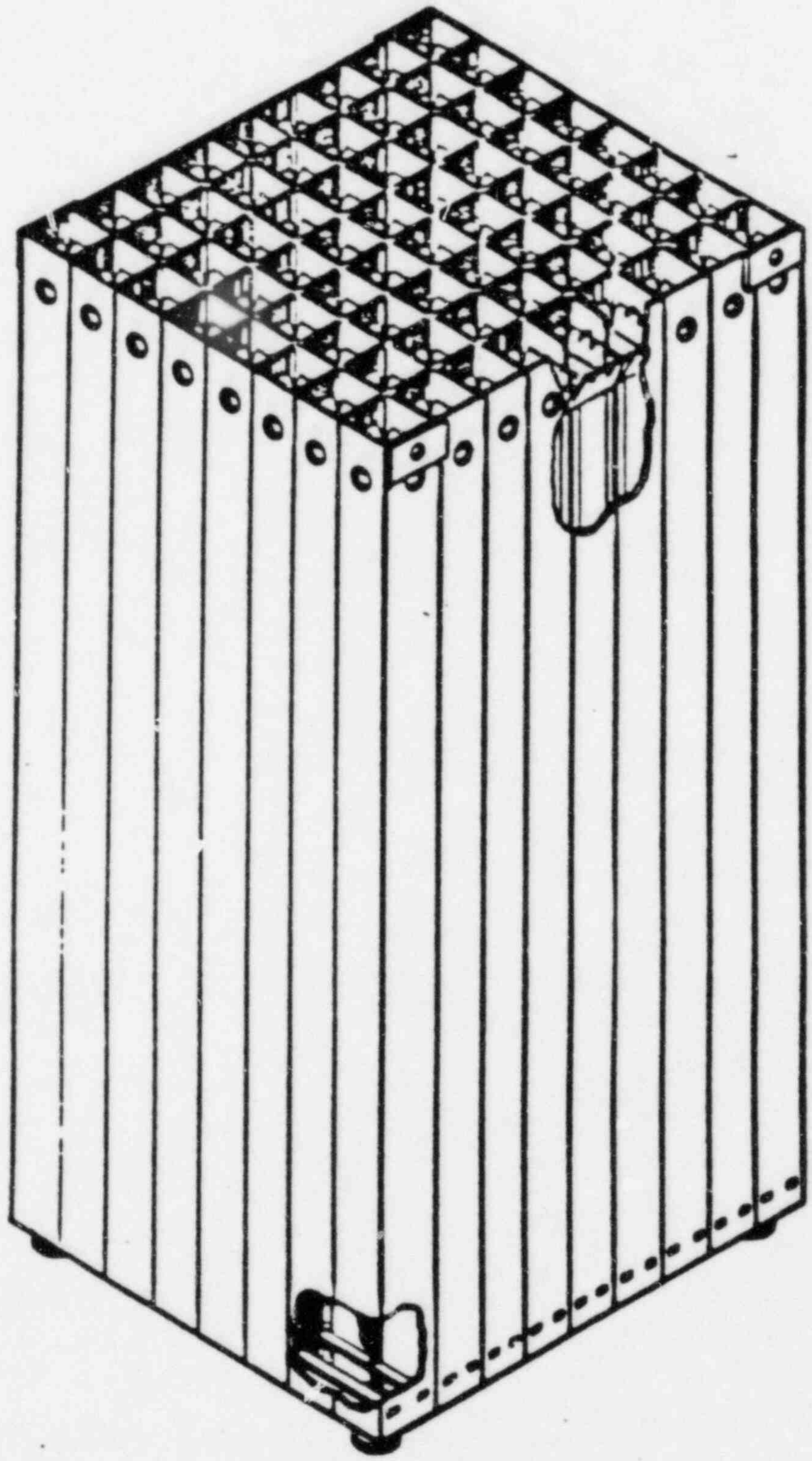

Figure 2. Typical Spent Fuel Rack Module/Poison Box (Region I)

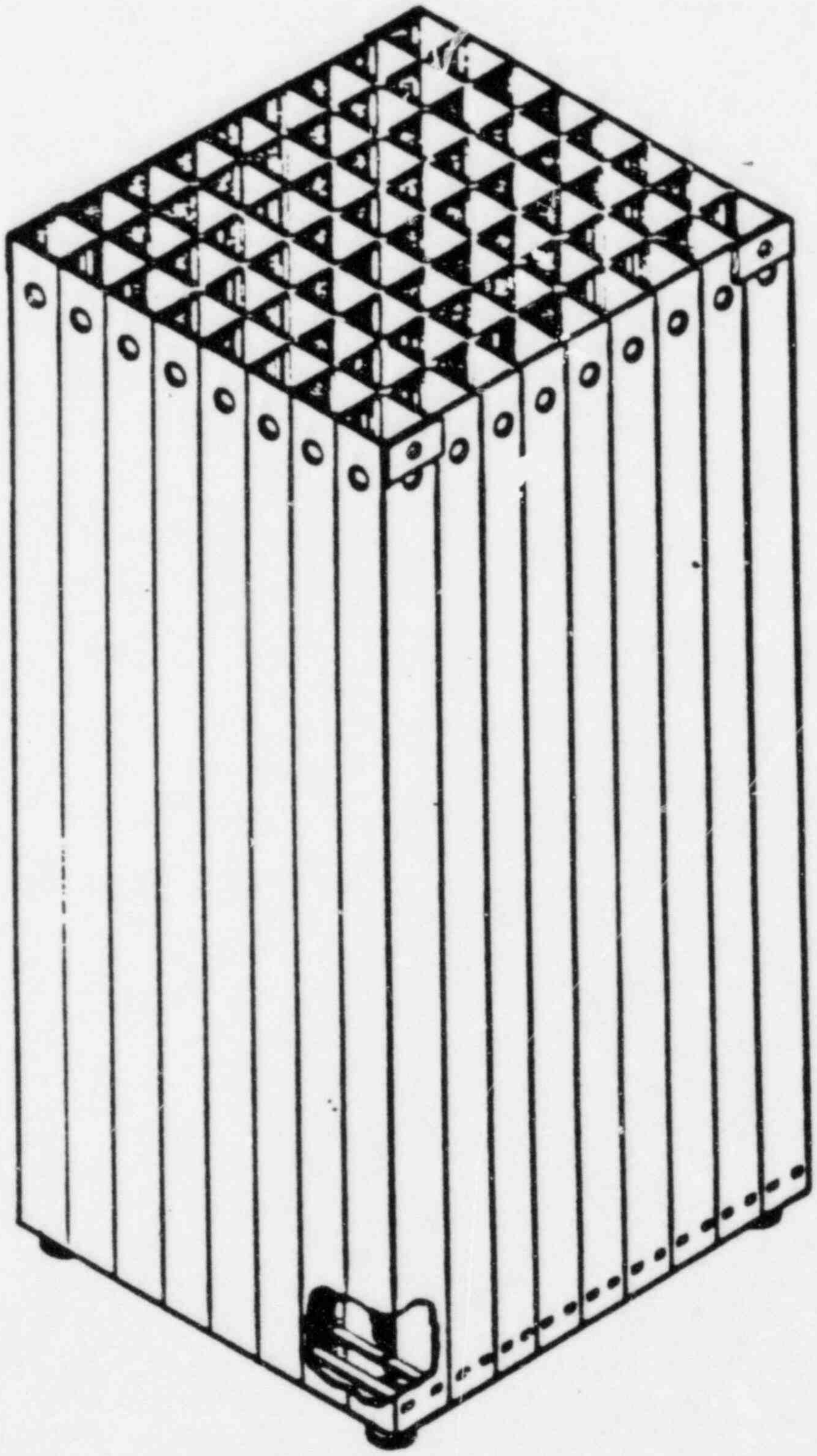

Figure 3. Typical Spent Fuel Rack Module (Region II)

The Licensee's choice of rack modules for analysis was based upon the rack's response to the dominant spectra of the earthquake accelerations as discussed in Section 3.2.5. For rack modules with natural frequencies excited by the earthquake spectra, displacements due to sliding are generally low compared with that due to tipping. Also, rack stresses are generally higher for those racks with increased natural frequency response to the seismic spectra. Accordingly, the Licensee's choice of rack modules for analysis is acceptable.

The seismic displacement analysis of free-standing, submerged rack modules was performed by the Licensee in two parts [1, 3]. The first part of the analysis used the SAP IV finite-element computer program [4] to perform a linear three-dimensional analysis of the rack structure for the determination of the natural frequencies and mode shapes of the rack for use in the non-linear dynamic displacement analysis. The dynamic characteristics were then incorporated into the nonlinear two-dimensional representation (Figure 4) of a rack module system that included the rack modules, the fuel assemblies stored in the rack cells, the hydrodynamic mass of entrained and displaced water, and the hydrodynamic coupling to adjacent racks or pool walls. The Licensee used the CESHOCK computer program for solution of the two-dimensional, nonlinear dynamic model. CESHOCK was described as a modified, proprietary version of the SHOCK [5] computer code.

The linear three-dimensional model was then used with the resulting maximum dynamic loadings from the two-dimensional nonlinear dynamics analysis to compute the maximum bending moments and stresses in the rack modules.

The Licensee's seismic and stress analysis of the spent fuel rack modules considered full, partially filled, and empty rack modules, including the effects of nonsymmetric fuel loadings in partially filled racks.

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

In conjunction with Figure 4, the Licensee described the mathematical model used for the nonlinear rack module dynamic analysis as follows [3]:

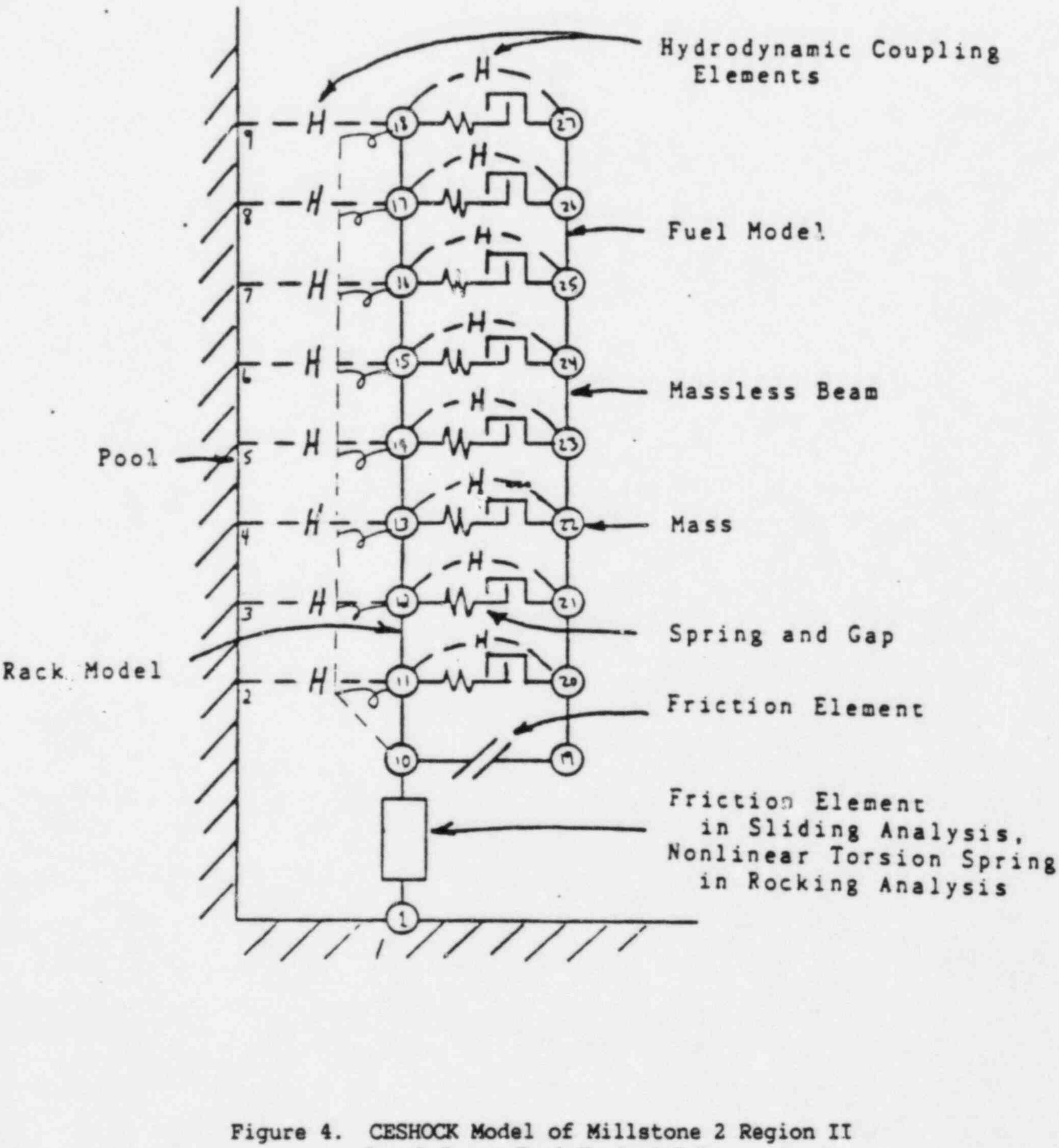

Figure 4. CESHOCK Model of Millstone 2 Region II 7 x 9 Spent Fuel Rack Module

"The model is two-dimensional, with each mass having a translational and a rotational degree-of-freedom. Mass nodes 1 through 18 were used to represent the fuel rack module. These mass nodes were linked by massless flexible elements. Similarly, mass nodes 19 through 27 were used to represent the fuel. Hydrodynamic couplings, designated by element H, are included between the rack module nodes and the pool structure nodes, and between the fuel nodes and the rack module nodes. Nonlinear gap-spring elements were used to represent the possibility of impacting between the fuel and the rack module. The fuel was coupled to the base of the rack module by a "slip-stick" friction element. An element at the interface of the module base and the pool liner represented a "slip-stick" friction element in the sliding analysis and a nonlinear torsion spring in the shear and rocking analyses."

The Licensee used a spring-gap element to model the impact between the fuel assembly and the rack cell wall in the nonlinear impact analysis [3]. The spring simulated the impact stiffness of the fuel assembly, the stiffness value of which was determined from full-scale fuel assembly impact tests and model-test correlations of test data with analytical results. While the Licensee reported that the tests were performed using Westinghouse fuel assemblies, it was reported that the Westinghouse fuel assemblies were stiffer than the Combustion Engineering fuel assemblies used by Millstone Unit 2. This stiffer spring constant was used for added conservatism in the nonlinear dynamic analysis. While fuel assembly impact damping greater than typical structural damping values can usually be justified for most fuel assemblies, the Licensee conservatively did not use impact damping.

Hydrodynamic mass coupling of the rack module to adjacent rack modules and to the spent fuel pool walls is shown in Figure 4, and is discussed in Section 3.2.3.

Evaluation indicates that the Licensee's modeling of the spent fuel racks for dynamic analysis is acceptable.

3.2.2 Frictional Force Between Rack Support Pads and the Pool Liner

The values of the friction coefficient used between the support pads and the pool liner is of considerable importance. Not only are seismic accelerations of the pool floor transmitted to the spent fuel rack modules by friction forces developed between the support pads and the pool liner, but any sliding that may develop is opposed by the friction forces.

The Licensee reported the following with regard to friction forces [3]:

"Friction between the pool liner and the module mounting feet is addressed in two ways. In the first approach, the rack module is not permitted to slide relative to the pool. In this case, the coefficient of friction is assumed to be extremely high to model the possibility of adhesion between the rack module and the pool which could occur over the design life of the modules due to one of several mechanisms. This fixed-base model provides conservative shear loads to both the module and the pool liner.

The second approach uses a sliding-base model in which a friction element connects the rack module base to the pool liner. The friction element used is a slip-stick friction element with a velocity dependent coefficient of friction. Realistic values for the coefficient of friction are used in this sliding base model. A static coefficient of friction of 0.55 was used. The coefficient of friction decreases linearly with increasing relative velocity of the module base with respect to the pool linear until a minimum dynamic coefficient of friction of 0.28 is reached at a relative velocity of 2.5 in/sec of the module base with respect to the pool liner. For relative velocities above 2.5 in/sec, the minimum dynamic coefficient of friction applies."

The Licensee provided the following justification for the approach to friction forces in the analysis:

The friction values used are based on the following sources:

"1. data from Combustion Engineering laboratory tests,

2. data obtained through a technical exchange agreement with Kraftwerk Union (KWU) of West Germany.

   Final Report of a Theoretical and Experimental Study for Further Development of Light Water Pressurized Water Reactors, "Wear Behavior of Friction Materials and Protective Layers with Regard to their Application Possibilities in Water Cooled Nuclear Reactors," written by P. Hoffman, Metallic Materials RT41, Fordervagsvorhaben BMFT-Inv. Reakt. 72/S11 Draftwert Union, August 1983, and

3. textbook Friction and Wear of Materials, Ernest Rabinowicz.

   Justification for the use of the stated values of friction coefficient lies in the basis of their selection being results of experimental studies. The values used in the analysis are values that have been derived from laboratory testing."

The Licensee's range of friction coefficients considered in the analysis is supported by Rabinowicz's report [6] on friction coefficient values for high-density fuel storage systems. The range of friction coefficients is acceptable.

3.2.3 Hydrodynamic Coupling Between Fluid and Cell Structure

Hydrodynamic coupling in the dynamic analysis was applied to the coupling between fuel assemblies and rack modules, and to the coupling between rack modules and the spent fuel pool walls. The hydrodynamic modeling was described by the Licensee as follows [3]:

> "In the nonlinear analysis models, hydrodynamic coupling is specified between the rack module and the pool, and between the fuel and the rack module. Potential theory (incompressible inviscid theory) is employed, using simple two-dimensional models of the structures coupled by the fluid, to estimate the hydrodynamic virtual mass terms based on the model configuration. Three-dimensional end effects were then accounted for by modifying the calculated hydrodynamic mass terms.
>
> For the rack module-to-pool hydrodynamic element, the rack modules were assumed to move in-phase and the potential theory model consisted of two bodies: the fuel rack module array within the spent fuel pool structure.
>
> To determine the resulting hydrodynamic mass terms, a finite element analysis using a computer code based on two-dimensional potential flow, was used. The ADDMASS computer code, C-E proprietary, was used to calculate the hydrodynamic masses of two dimensional bodies with arbitrary cross-sectional shapes with fluid finite elements between the bodies. ADDMASS is based principally on the following work: Yang, C. I., "A Finite - Element Code for Computer Added Mass Coefficients," Argonne National Laboratory Report No. ANL-LT-78-49, September 1978."

Hydrodynamic coupling is an important aspect of the nonlinear dynamic displacement analysis. The degree of conservatism and the amount of experimental verification in the modeling used, as applied specifically to spent fuel rack module dimensions and clearances, should be stated.

The Licensee did not indicate the degree of conservatism inherent in the method or the amount of experimental verification. However, a review of Yang's method [7], which formed the basis for the ADDMASS program used for the Licensee's analysis, indicated that it was developed for hydrodynamic coupling in reactor internals and is acceptable.

3.2.4 Integration Time Step

With respect to the time step of integration and the stability of the numerical solution, the Licensee provided the following [3]:

> The CESHOCK code numerically integrates the equations of motion using a Runge-Kutta-Gill technique. The initial integration timestep, calculated by CESHOCK, is one-twentieth of the period of the highest individual mass-spring frequency in the model. The timestep is continually checked and adjusted by the code as a function of the rate of change of the linear and angular accelerations. The timestep is held within the bounds of one-fifth times the initial timesteps to two times the initial timestep. With this procedure for selecting the integration timestep, the CESHOCK numerical solution has been shown to be stable and convergent."

This timestep variation, in conjunction with the more stable Runge-Kutta-Gil integration technique, is considered to be satisfactory.

3.2.5 Rack Displacements

This Licensee's analysis of rack module displacements included tipping of the rack module. The Licensee provided the following descriptions [3, 8].

> "The maximum horizontal displacement of the top of a module, including tipping, is determined from a nonlinear time history analysis of an individual module. Separate analyses are made for number of different modules with varying degrees of fuel loading, including empty, partially loaded, and fully loaded modules. In these individual module time history analyses, all the modules in the pool are assumed to move in-phase when determining the rack-to-pool hydrodynamic characteristics." [8]

> "An in-phase mode of vibration was conservatively considered in assessing the hydrodynamic coupling effects between adjacent rack modules. Because of the character of the site specific Millstone 2 seismic excitation, the higher rack module frequencies resulting from the in-phase mode analysis were conservative because they were closer to the frequencies of the response spectra peaks. An out-of-phase mode of vibration would have resulted in the lower frequencies farther away from the response spectra peaks. The lower frequencies result from high hydrodynamic masses produced by out-of-phase motion." [3]

With respect to the worst case of tipping, the Licensee supplied the following to indicate that the combined displacement of two adjacent rack modules would not exceed the 2.0-inch clearance between modules.

"The worst case for tipping was a Region II 7 x 8 module, partially loaded with fuel, excited by the East-West seismic component (7 cell direction). The worst case for shear load was a Region II 7 x 9 module fully loaded with fuel, excited by the North-South seismic component (9 cell direction)." [8]

"To calculate the peak intermodular relative displacement adjacent modules are assumed to move out-of-phase. The peak relative displacement is conservatively calculated by summing the absolute value of the peak displacements at the top of the module for the two modules considered. Using this approach, a peak intermodular relative displacement of 1.776 inches was determined. This value is less than the intermodular gap." [9]

Evaluation of the Licensee's rack module displacement and tipping analysis indicated that it was adequate.

3.3 EVALUATION OF THE THREE-DIMENSIONAL, LINEAR STRESS ANALYSIS

3.3.1 Description of the Methodology

As described in Section 3.1 of this report, the Licensee used linear three-dimensional finite-element analysis for rack module stress analysis as well as to determine the rack structural properties for displacement analysis.

The Licensee described the methodology for using the maximum loadings from the nonlinear, two-dimensional displacement analysis for the analysis of stresses as follows [3]:

"The results of the nonlinear time history analyses, performed in both horizontal directions, and the linear response spectrum analysis, performed for the vertical direction, provide a set of load multiplication factors to be applied to the three-dimensional SAP IV stress model. The horizontal load factor is defined as the ratio of the maximum horizontal shear load derived from the CESHOCK model nonlinear time history analysis to the horizontal empty rack (modal) weight from the SAP IV model. Likewise, the vertical load factor is defined as the ratio of the maximum vertical load determined from the response spectrum analysis to the vertical empty rack (modal) weight from the SAP IV model. The load factors are applied to the component stresses obtained from the SAP IV model. These stresses were obtained by applying a one-G response spectrum load to each of the three orthogonal directions. Maximum Base shears and load factors are tabulated below:

Base Shears	Region I Rack	Region II Rack
Maximum Horizontal:		
SSE	880#/Cell	977 #/Cell
OBE	Not Applicale	603 #/Cell

Base Shears	Region I Rack	Region II Rack
Maximum Vertical:		
SSE	3721 #/Cell	3423 #/Cell
OBE	SSE values for maximum vertical base shears were used. -	

Typical Load Factors	Region I Rack	Region II Rack
Horizontal (X-direction)	10.10	12.70
Horizontal (Y-direction)	9.39	11.59
Vertical (X-direction)	26.02	26.82

(Factors shown are based on 8 x 10 and 7 x 9 Racks).

The analysis to determine the structural adequacy of the fuel storage module under tipping was conducted using the following techniques: 1) Two loading conditions were applied to the SAP IV model including a 1-G horizontal load placed in the direction the module tips, and a 1-G vertical downward load. 2) Using the principal of superposition the vertical load is adjusted until the compression and tension in the feet which lift is reduced to zero, thereby creating a load state that approximates the module at the instant the module lifts off.

The actual horizontal seismic load, at the point of lift off, is determined in a similar fashion as described above using a nonlinear time history analysis. The 1-G horizontal and the adjusted 1-G vertical load can now be factored. This factor will be the seismic load due to the loaded module divided by the 1-G horizontal load of an empty module.

This approach can determine the stress state of the module due to module tipping under seismic effects. This approach is only valid for lift off of a few mils. The results of the nonlinear analysis indicates such a situation does exist."

"The vertical tipping displacement from the horizontal nonlinear analysis is used as the input to a separate vertical nonlinear CESHOCK model that is used to calculate vertical impact loads. These loads are used to determine the adequacy of the foot/rack design."

TYPICAL MULTIPLICATION FACTORS FOR SEISMIC EFFECT

Horizontal 1-G Factor = 6.895

Vertical 1-G Factor = 20.82

(Factors shown are based on 7 x 9 rack.)

3.3.2 Rack Module Load Combinations

The Licensee used the following load combinations in accordance with the NRC's position [2] on spent fuel storage and handling [3]:

Load Combination (Elastic Analysis)	Acceptance Limit
D + L	Normal limits of NF 3231.1a
D + L + E	Normal limits of NF 3231.1a
D + L + To	Lesser of 2Sy or Su stress range
D + L + To + E	Lesser of 2Sy or Su stress range
D + L + Ta + E	Lesser of 2Sy or Su stress range
D + L + Ta + E'	Faulted condition limits of NF 3231.1c

Floor-rack interface loads reported by the Licensee are [1]:

Floor-Rack Interface Loads (lb_F/Pad)

	North-South	East-West	Vertical (Down)
OBE + Dead Weight (4 pads in contact)	12,100	12,100	107,300
SSE + Dead Weight (2 pads in contact)	39,100	38,900	131,200
SSE + Dead Weight + Vertical Impact (4 pads in contact)	39,100	38,900	133,000

3.3.3 Review of Stress Levels

The Licensee's reported maximum stresses, allowable stresses, and design margins are shown in Table 1 [3]. Except for the support pad adjustment screw, the reported stresses are for the SSE load condition. Because these stresses are below the OBE allowable values, the stresses are acceptable for both the OBE and SSE load conditions. Stresses for the adjustment screw and their allowable limits were provided by the Licensee for both the OBE and SSE conditions.

Table 1. Rack Module Stresses and Allowable Values

Note: Stresses do not necessarily occur at the same location.

A. Monolith	Maximum Stress	Allowable Stress OBE	Design Margin
Membrane stress	17,560 psi	18,300 psi	4.2%
Membrane plus bending	21,760 psi	27,450 psi	26.2%
Primary plus thermal	28,511 psi	55,000 psi	92.9%
B. Support Bars			
Bending stress	5,454 psi	16,500 psi	202.3%
Shear stress	526 psi	11,000 psi	1991.3%
C. Adjustable Foot			
1. Block			
Shear stress	2,918 psi	11,000 psi	277.0%
Axial plus bending			
OBE	13,655 psi	16,500 psi	20.8%
SSE	19,290 psi	33,000 psi	71.1%
2. Adjustment Screw			
OBE Condition	Maximum Stress	OBE Allowable Stress	Design Margin
Axial stress	11,810 psi	49,360 psi	317.9%
Shear stress	18,230 psi	33,500 psi	83.8%
Bending stress	24,980 psi	50,250 psi	101.2%
Combined axial compress. plus bending	$\frac{fa}{Fq} + \frac{fb}{Fb} = 0.736$	1	20.8%
SSE Condition	Maximum Stress	OBE Allowable Stress	Design Margin
Axial stress	14,773 psi	91,000 psi	516%
Shear stress	29,400 psi	54,600 psi	85.7%
Bending stress	60,554 psi	91,000 psi	50.28%
Combined axial compress. plus bending	$\frac{fa}{Fq} + \frac{fb}{Fb} = 0.828$	1	20.8%
Thread shear	6,710 psi	11,000 psi	63.9%

The Licensee's design margin, defined as

(Allowable Stress/Design Stress - 1) x 100%

is shown in Table 1 for each reported stress.

With respect to the summation of stresses due to simultaneous loadings in different directions, the Licensee reported the following [3]:

> "Final stress combinations are derived from R.S.S. method of each component stresses magnitude regardless of the direction. (E.G., a typical element may be comprised of both tension and compression stress combined together.) The component stresses assumes a three directional earthquake having their peaks occurring simultaneously."

Review of the Licensee's analysis methodology and resulting stresses indicates that both are acceptable.

3.4 REVIEW OF SPENT FUEL POOL STRUCTURAL ANALYSIS

3.4.1 Spent Fuel Pool Structural Analysis

The spent fuel pool and the associated structures of the auxiliary building were analyzed for the increased density of spent fuel storage using a finite-element model comprising 9600 degrees of freedom.

Loads used in the analysis of the pool structure and liner were based upon a 2:1 consolidation of spent fuel.

Load combinations used for comparison of computed stresses with allowable values were chosen in accordance with Section 3.8.4 of the USNRC's Standard Review Plan. The composite load cases included dead load, live load, operating thermal and accident loads, operating basis earthquake (OBE) loads, and safe shutdown earthquake (SSE) loads. Normal operating thermal loads were based on 150°F pool water with temperatures of 55°F outside the pool. Accident thermal loads were based upon a pool/wall interface temperature of 212°F.

3.4.2 Analysis Procedures

3.4.2.1 Description of the Finite-Element Pool Structural Model

The Licensee's modeling of the spent fuel pool and associated structure is shown in Figure 5 and was described as follows [3]:

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

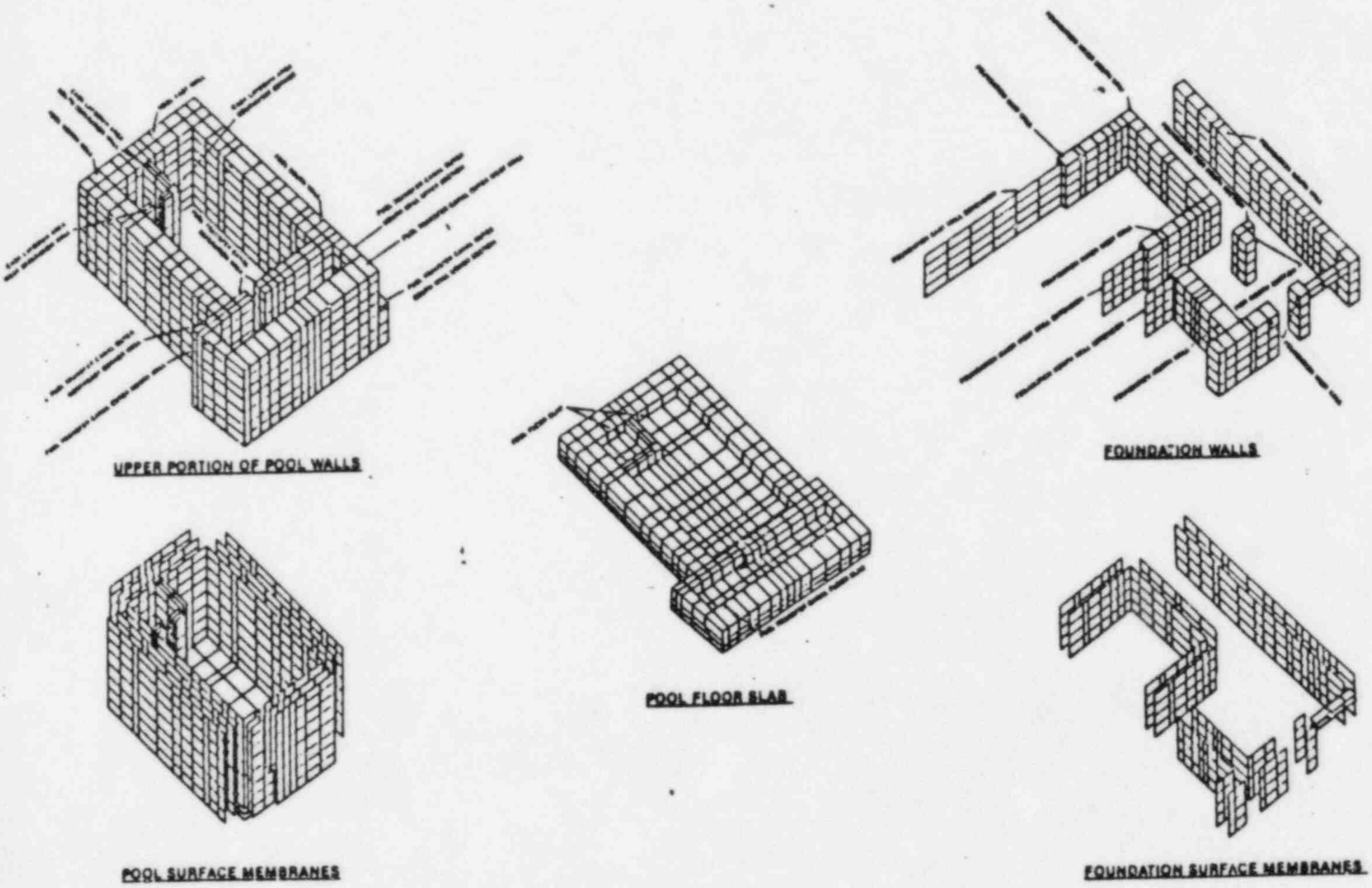

Figure 5. Isometric Views and Key Diagrams

"The extent of the structural model includes the pool walls, cask laydown and fuel transfer canal area walls (excluding the gates), pool floor slab and fuel transfer canal floor slab and the foundation walls directly beneath this portion of auxiliary building. All walls directly adjacent to the pool (including the fuel transfer canal inside wall and cask laydown area walls) and the pool floor slab are modeled with two layers of eight node solid elements to permit proper application of thermal gradients and to provide good definition of stress variations through the wall thickness. Four node membrane elements of negligible thickness were used on the inside, middle, and outside surfaces of the wall or floor to obtain stress values at the solid elements faces as well as at the solid element centroids. In this manner, five integration points through the walls and floors were obtained. The outer walls and floor slab of the fuel transfer canal area were modeled with a single layer of solid elements since these components were only included for their stiffness properties and were not evaluated according to stress criteria. The portions of the foundation which were modeled include the south, west, north, inner west, inner south and east foundation walls. These components were modeled with only one layer of solid elements with membrane elements on the inside and outside surfaces since there is no thermal gradient through the walls of the compartments at this elevation. The other structural components modeled in the foundation were the pier (solid elements) and the extensions of the inner west and east foundation walls (which were modeled with membrane elements to represent their in-plane stiffness).

Since rotations at the node points of the three-dimensional solid elements are not defined, all rotational degrees of freedom in the model were restrained. Stiffnesses of the walls and floors framing into the pool model were represented using direct matrix additions. The matrix coupling terms were computed assuming that, due to cracking, one-half of the wall or floor panel stiffness is available. The nodes at the base of the foundation which are remote from the structural areas of interest in the pool were completely restrained.

The liner plate was modeled such that all weld seams and anchor locations were coincident with node lines or node locations. Global and local coordinate systems were specified such that they were coincident with the pool floor slab elements in the SAP6 finite element model. All rotations and displacements normal to the plate were restrained. Lateral degrees of freedom are unrestrained for all nodes except weld seams and anchor locations, which were identified as boundary degrees of freedom at which displacements can be either specified or restrained."

3.4.2.2 Spent Fuel Pool Loads

The Licensee used twelve basic computer load cases, as shown in Table 2. Stresses resulting from these basic load cases were multiplied by appropriate factors and combined in accordance with the stated load cases of Section 3.8.4

Table 2. Individual Load Case Description

SAP6 Load Case Number	Description
1	1 g vertical acceleration for dead weight of concrete
2	Hydrostatic forces
3	1000 lb/ft^2 vertical slab load over Region 1
4	1000 lb/ft^2 vertical slab load over Region 2
5	Operating thermal (pool water at 150°F)
6	Accident thermal (pool water at 212°F)
7	1 g ZPA north earthquake. 2.34 g peak pool wall acceleration plus hydrodynamic forces (+X acceleration)
8	1 g ZPA west earthquake. 2.34 g peak pool wall acceleration plus hydrodynamic forces (+Y acceleration)
9	-1000 lb/ft^2 horizontal slab load over Region 1 in X direction (+X acceleration)
10	-1000 lb/ft^2 horizontal slab load over Region 2 in X direction (+X acceleration)
11	-1000 lb/ft^2 horizontal slab load over Region 1 in Y direction (+Y acceleration)
12	-1000 lb/ft^2 horizontal slab load over Region 2 in Y direction (+Y acceleration)

of the Standard Review Plan for comparison with allowable stresses. A brief description of the Licensee load cases follows [4]:

- o Dead weight of the pool structure was applied as a 1.0 g vertical acceleration (load case 1).
- o Hydrostatic loading of the structure was used for a pool water depth of 38 in 6 ft (load case 2).
- o "Load cases 3, 4, and 9 through 12 are nominal 1,000 pounds per square foot loads applied to the pool floor slab in the negative global z (vertical), x and y directions. These unit load cases were used to later formulate vertical (z) rack loads and lateral (X-y) loads. Application of the load in each direction was subdivided into two load cases to provide for the differential fuel rack configurations in regions 1 and 2 of the pool."
- o Load cases 5 and 6 were used for the normal operating thermal loads and accident thermal loads using the temperature mentioned previously.
- o Load cases 7 and 8 were used for building seismic effects and the associated hydrodynamic forces, wherein the earthquake response of the pool water was based on the methodology outlined in TID-7024, "Nuclear Reactors and Earthquakes." The Licensee reported that all acceleration values were taken from the "Seismic Analysis-Auxiliary Building," Millstone Nuclear Power Station Unit 2, Bechtel Power Corporation, Job No. 7604-01, Revision 3, July 1972 [4].

The Licensee did not include individual vertical load cases for vertical earthquake loads [4]. These were handled by applying load factors to the respective static loads cases to account for dynamic amplification of the seismic motion.

Table 3 provides a summary of the Licensee's load definition parameters [4].

Dead load included dead weight of the structure, hydrostatic pressure, and weight of the fuel rack modules, excluding their fuel complements.

Live load consisted entirely of the submerged weight of the consolidated fuel and storage boxes. The Licensee reported that the actual reracking live load in Region 1 would be 40 percent less, and that in Region 2 to be 68 percent less, than the loads for consolidated fuel used in the analysis.

Table 3. Summary of Load Definition Parameters

Item	Description
Pool Properties	
Pool Water Depth	38'-6"
Pool Normal Operating Temperature	150°F
Pool Accident Temperature	212°F
Pool Hydrodynamic Forces	TID 7024, App. F
Auxiliary Building Compartment Temperatures	
All Compartments	55°F
Thermal Stress - Free Temperature	55°F
Operating Conditions	
Fuel Transfer Canal	Dry
Cask Laydown Area	Dry
Seismic Ground Accelerations	
OBE Horizontal	0.09 g
OBE Vertical	0.06 g
SSE Horizontal and Vertical	1.8 (OBE)

With respect to earthquake loads, the Licensee reported as follows [4]:

"Operating basis earthquake (E) was specified as 0.09 g horizontal and 0.06 g vertical ZPA levels measured at the base of the foundation. Since amplification of the base motion acceleration levels was accounted for in the individual load cases, a coefficient of 0.09 was applied to the horizontal response loads (load cases 7 and 8). Similarly, the response to vertical earthquake is constant over the pool height as specified in the plant design manual, so a factor of 0.06 on the dead weight load was used for this load case. SSE horizontal and vertical reactions for the submerged racks were specified as 3,500 pounds per cell and 1,000 pounds per cell, respectively. OBE loads are calculated as 56 percent of the SSE loads. Based on these cell reactions, the OBE vertical loads are 569 psf over Region 1 and 923 psf over Region 2. The resulting OBE horizontal loads are 1,992 psf over Region 1 and 3,232 psf over Region 2."

Table 4 shows the manner in which results from the 12 basic computer loads were used to define each composite loading element. Similar SSE loads were formulated by multiplying the eight OBE cases by 1.8.

Table 5 shows the service and factored load combinations in accordance with Section 3.8.4 of the Standard Review Plan using all load components appropriate to the spent fuel pool and rack modules.

Table 6 presents the same service and factored load combinations after the Licensee eliminated the load components that were not meaningful for the pool structural analysis [4]. Table 6 also notes the load combinations that became equal or bounded by other load combinations through the removal of the unused load components.

3.4.2.3 Analysis Criteria and Computed Stress Levels

The Licensee described the source of criteria for allowable loads and stresses as follows [3]:

"The spent fuel pool was evaluated according to the criteria in the Millstone Point Unit 2 Design Criteria NRC Standard Review Plan. The original design was performed according to ACI-318-63 code criteria. For this evaluation Northeast Utilities has chosen to utilize load combinations specified in the NRC Standard Review Plan followed by evaluation of the reinforced concrete sections according to ACI 349-80. The pool wall and floor liner plate were evaluated according to the strain criteria specified by the ASME Code. A plate thickness tolerance of 16% was used, along with the weld offset, for computing membrane plus bending strains. Pool floor liner plate weld stresses were compared to AISC criteria. As shown in Table

Table 4. Composite Load Case Description

Individual Load Case Number:	1	2	3	4	5	6	7	8	9	10	11	12
Composite Load Case												
1 D – Dead Load	1.00	1.00	0.374	0.607								
2 L – Live Load			2.56	4.16								
3 To – Operating Thermal					1.00							
4 Ta – Accident Thermal						1.00						
5 E_1 – OBE	0.06	0.06	0.57	0.92			0.09	0.09	1.99	3.23	1.99	3.23
6 E_2 – OBE	0.06	0.06	0.57	0.92			-0.09	0.09	-1.99	-3.23	1.99	3.23
7 E_3 – OBE	0.06	0.06	0.57	0.92			-0.09	-0.09	-1.99	-3.23	-1.99	-3.23
8 E_4 – OBE	0.06	0.06	0.57	0.92			-0.09	-0.09	-1.99	-3.23	-1.99	-3.23

NOTES: 1. Four additional OBE cases are defined as -1.0 times E_1 through E_4, respectively.
2. SSE is taken as 1.8 times OBE.

Table 5. Standard Review Plan Load Combination Summary

Load Combination Number	Description
	SERVICE LOAD COMBINATIONS
i.b.1	$1.4D + 1.7L$
i.b.2	$1.4D + 1.7L + 1.9E$
i.b.3	$1.4D + 1.7L + 1.7W$
i.b.4	$0.75\ (1.4D + 1.7L + 1.7T_o + R_o)$
i.b.5	$0.75\ (1.4D + 1.7L + 1.9E + 1.7T_o + R_o)$
i.b.6	$0.75\ (1.4D + 1.7L + 1.7W + 1.7T_o + R_o)$
i.b.7	$1.2D + 1.9E$ or $0.9\ (1.4D) + 1.9E$
i.b.8	$1.2D + 1.7W$ or $0.9\ (1.4D) + 1.7W$
	FACTORED LOAD COMBINATIONS
ii.a	$D + L + T_o + E'$
ii.b	$D + L + T_o + R_o + W_1$
ii.c	$D + L + T_a + R_a + 1.5\ P_a$
ii.d	$D + L + T_a + R_a + 1.25\ P_a + 1.0\ (Y_r + Y_j + Y_m) + 1.25E'$
ii.e	$D + L + T_a + R_a + 1.0\ P_a + 1.0\ (Y_r + Y_j + Y_m) + 1.0E'$

Table 6. Applicable Standard Review Plan Load Combinations

Load Combination Number	Description	
	SERVICE LOAD COMBINATIONS	
i.b.1	$1.4D + 1.7L$	
i.b.2	$1.4D + 1.7L + 1.9E$	
i.b.3	$1.4D + 1.7L$	(Identical to i.b.1)
i.b.4	$0.75\ (1.4D + 1.7L + 1.7T_o)$	
i.b.5	$0.75\ (1.4D + 1.7L + 1.9E + 1.7T_o)$	
i.b.6	$0.75\ (1.4D + 1.7L + 1.7T_o)$	(Identical to i.b.4)
i.b.7	$1.2D + 1.9E$ or $0.9\ (1.4D) + 1.9E$	(Bounded by i.b.2)
i.b.8	$1.2D +$ or $0.9\ (1.4D)$	(Bounded by i.b.1)
	FACTORED LOAD COMBINATIONS	
ii.a	$D + L + T_o + E'$	(Bounded by ii.d)
ii.b	$D + L + T_o$	(Bounded by ii.c)
ii.c	$D + L + T_a$	
ii.d	$D + L + T_a + 1.25E'$	
ii.e	$D + L + T_a\ 1.0E'$	(Bounded by ii.d)

3.1-1 [of Reference 3], a stress allowable criteria is used in evaluating the anchors for nonthermal loads versus a displacement criteria for thermal load combinations."

The presentation of computed loads and stresses and their comparison with allowable values was shown by the Licensee as follows:

Table [of Reference 3]	Description
4.1-1	Tabulation of controlling section resultant moments
4.1-2	Tabulation of resultant transverse shear forces
4.1-3	Tabulation of resultant in-plane shear forces
4.1-4	Pool floor linear plate analysis summary

Typically, each table provides the location in the pool under consideration, the controlling load combination case, the resultant load or stress, the allowable value, and the ratio of the resultant load to the allowable value. For the most part, these ratios are low compared with 1.0. However, five such ratios are larger than 0.90. These are identified in the following statements:

- o Table 4.1-1; for the mid-span horizontal section of the fuel transfer canal, south wall portion, the resultant section moment was reported to be 0.96 that of the allowable value.
- o Table 4.1-1; the ratio of the section moment to the allowable value was 0.98 for the vertical section of the south foundation wall, west portion.
- o Table 4.1-2; the ratio of the section shear to the allowable value was 0.94 for the horizontal section of the pool cast wall, top of wall.
- o Table 4.1-2; the ratio of the section shear to the allowable value was 0.95 for the horizontal section, mid-height of the south portion of the fuel transfer canal separation wall.
- o Table 4.1-4; weld stress of the pool liner was 0.99 of the allowable value under the controlling thermal load combination.

While these reported loads and stresses approach the allowable values, it was noted that the Licensee's loads were based upon a 2.1 fuel consolidation. Actual reracking loads and stresses were reported to be between 40% and 68% less (see Section 3.4.2.2).

Evaluation of the Licensee's spent fuel pool analysis indicated that there is satisfactory design margin in the pool structure to accommodate the high density fuel assembly 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 [3]:

> "An analysis of a typical fuel rack indicated that the force required to deform an individual canister or to overcome the dead weight of the rack is significantly greater than the load which the spent fuel handling machine can impart."

3.5.2 Accidental Fuel Assembly Drop

The Licensee provided the following [3]:

> "The fuel drop accident was evaluated to determine the effect of the dropped assembly on the functional and structural integrity of the racks. The analysis indicated that the impact of the fuel assembly on the support bars caused plastic deformation of the support bars and the fuel cell wall supporting the bars. For conservatism it was assumed that further displacement of the bars occurs, resulting in the fuel and support bars resting on the pool floor. No functional or structural integrity of the racks was impaired.
>
> A fuel bundle drop vertically through the rack to the fuel support has resulted in the side walls of the rack; shearing however, the bundle and support bars did not impact the floor, resulting in no damage to the pool liner. (The active fuel length of the bundle will remain contained within the storage rack.)"

4. CONCLUSIONS

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

- The Licensee used two-dimensional nonlinear dynamic displacement analyses coupled with three-dimensional, linear, finite-element stress analysis to evaluate the rack module stresses and displacements under simulated earthquake acceleration time histories. Stresses combined by the square root of the sum of the squares method and rack module displacements computed by the nonlinear analysis were acceptable.

- While a finite-element analysis of the spent fuel pool structure indicated that local section loads and stresses at five points approached, but remained under, the allowable values, it was noted that these local section loads and stresses resulted from pool floor loads imposed by a 2:1 consolidation of fuel. Actual rerack loads were stated to produce pool structure stresses 40% to 68% below the analysis results. This indicated that the design margin of the spent fuel pool structure is sufficient to accommodate the loads imposed by fuel assemblies stored in high density rack modules.

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