NORTHEAST UTILITIES

THE CONNECTICUT LIGHT AND POWER COMPANY
WESTERN MASSACHUSETTS ELECTRIC COMPANY
HOLYOKE WATER POWER COMPANY
NORTHEAST UTILITIES SERVICE COMPANY
NORTHEAST NUCLEAR ENERGY COMPANY

General Offices • Selden Street, Berlin, Connecticut

P.O. BOX 270
HARTFORD, CONNECTICUT 06141-0270
(203) 665-5000

October 28, 1985

Docket No. 50-336
B11827

Director of Nuclear Reactor Regulation
Attn: Mr. Edward J. Butcher, Chief
Operating Reactors Branch No. 3
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Gentlemen:

Millstone Nuclear Power Station, Unit No. 2
Reply to Request for Additional Information on Spent Fuel Storage Capacity

In October, 1985(1) the Staff requested additional information concerning a Northeast Nuclear Energy Company (NNECO) request(2) to modify the Technical Specifications concerning the spent fuel storage capacity at Millstone Unit No. 2.

Attachment No. 1 to this letter provides the response, in a question and answer format, to the eleven (11) questions contained in the Staff's request for additional information.

We trust that the information provided is sufficient, and we remain ready to address any further questions as they arise to support expeditious processing of our pending amendment request.

Very truly yours,

NORTHEAST NUCLEAR ENERGY COMPANY

J. F. Opeka
J. F. Opeka
Senior Vice President

W. F. Fee
W. F. Fee
Executive Vice President

---

(1) E. J. Butcher letter to J. F. Opeka, "Request for Additional Information on Spent Fuel Storage Capacity Expansion for Millstone Unit No. 2," dated October 3, 1985.

(2) J. F. Opeka letter to E. J. Butcher, "Millstone Nuclear Power Station, Unit No. 2, Proposed Change to Technical Specification Modifications to Spent Fuel Storage Pool," dated July 24, 1985.

A001
1/40

Docket No. 50-336
B11827

Attachment No. 1

Millstone Nuclear Power Station, Unit No. 2

Response to Request for Additional Information on

Spent Fuel Storage Capacity

October, 1985

1. With respect to seismic loadings on the spent fuel rack modules:

a. Identify which modules were analyzed.

The following rack modules were analyzed:

i) Region I 8 x 10 module

ii) Region II 7 x 8 module

iii) Region II 7 x 9 module

iv) Region II modified 7 x 9 module

b. Provide a description of how the horizontal earthquake acceleration (time history) was oriented relative to the long and short cross-sectional dimensions of the rack modules in the non-linear displacement anlaysis.

The pool layout was arranged so that the rack modules were placed in specific locations and orientations within the spent fuel pool. Acceleration time histories were available for both the north-south and east-west directions. The acceleration time histories were applied to the rack module models in a manner consistent with their actual in-pool orientations.

c. Describe what constitutes the worst case (identifying the factors by which the worst case was identified) and how it was considered.

The worst case for shear load was a Region II 7 x 9 module, fully loaded and excited by the north-south seismic component.

The most significant factor in identifying possible worst cases is the relationship between the model natural frequencies and the acceleration response spectra for the appropriate spent fuel pool acceleration time histories. For a given response spectrum, potential worst cases may be identified by selecting cases where the model natural frequencies are near the peak of the response spectrum. There are a number of other factors, however, that have an effect on the model frequency characteristics and consequently the response loads, among these area; the natural frequency of the rack module in air, the type of fuel storage, the hydrodynamic effects between the fuel and the rack module and between the rack module and the pool structure.

Because a number of factors affect the identification of a "worst case", a number of analyses are performed, which correspond to different regions of the pool, difference size modules, difference earthquake directions and types of fuel storage.

2. Reference 4-2 was cited on page 22 of the Licensee's report in lieu of any description of the non-linear model:

   a. Provide the relationship of this reference to the analysis performed for the Licensee's report.

   The cited reference describes the general methodology used to develop a nonlinear seismic analysis model of a spent fuel rack module. The reference stresses the importance of modeling fuel assemblies as discrete structural elements and the non-linear impacting behavior between the rack module and the stored fuel. Beyond these general themes there is no specific relationship between the cited reference and the analysis performed for the Millstone 2 spent fuel racks.

SEISMIC ANALYSIS OF SPENT FUEL RACKS

R. LONGO

D. F. BAISLEY

Nuclear Power Systems
Combustion Engineering, Inc.
Windsor, Connecticut

Presented at

AMERICAN NUCLEAR SOCIETY
TOPICAL MEETING ON
OPTIONS FOR SPENT FUEL STORAGE

September 26-29, 1982
Savannah, Georgia

CE POWER SYSTEMS

ABSTRACT

The paper describes the nonlinear time history seismic analysis method used by C-E for the design and licensing of spent fuel racks. The method is applied to spent fuel racks that store both standard and consolidated fuel assemblies. The analysis is based upon a direct numerical integration of the coupled equations of motion for the fuel and the rack. The equations of motion account for the gaps, hydrodynamic coupling and impacting between the structures of the fuel and fuel rack system. A summary of representative results from nonlinear time history analyses covering a wide range of designs and seismic excitations is presented. A comparison of these results with those obtained through the use of the response spectrum analysis method is presented to demonstrate that the response spectrum method—which is unable to account for interaction effects—may lead to incorrect results. The importance of modeling the fuel as a separate structural element is established. Examples of how the fuel responds to seismic excitation at its own natural frequencies—not at that of the rack structure—are presented. The applicability of the seismic analysis method to a consolidated fuel and fuel rack design is discussed.

Additional copies of this technical paper may be obtained by writing Communications, Dept. 7021-1904, Windsor. Please refer to the number (TIS-7308) that appears in the lower right corner of the front cover.

SEISMIC ANALYSIS OF SPENT FUEL RACKS

INTRODUCTION

C-E led the industry in performing nonlinear time history seismic analyses of spent fuel racks in 1975. Since then, C-E has applied the methodology to nine spent fuel rack applications covering a wide range of designs and reactor sites. This experience is supplemented with many parameter studies using the nonlinear time-history method.

The nonlinear time-history analysis method employed by C-E is based upon a direct numerical integration of the equations of motion for the fuel and the rack. It utilizes multi-degree-of-freedom spring and lumped mass models of the fuel and the rack, and accounts for the effects of gaps and submergence in water directly in the equations of motion defined by the model. It uses the seismic excitation time-history corresponding to the spent fuel pool elevation in the auxiliary building. Figure 1 provides an example of a typical

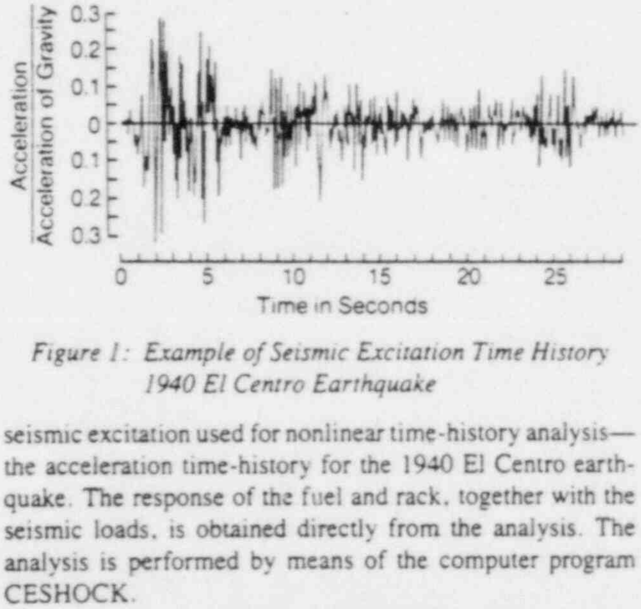


Figure 1: Example of Seismic Excitation Time History 1940 El Centro Earthquake

seismic excitation used for nonlinear time-history analysis—the acceleration time-history for the 1940 El Centro earthquake. The response of the fuel and rack, together with the seismic loads, is obtained directly from the analysis. The analysis is performed by means of the computer program CESHOCK.

To allow insertion and withdrawal of fuel, each spent fuel rack cell has a gap between the cell walls and the fuel. During seismic excitation, the fuel moves freely through the available gap and impacts the cell walls. The fuel responds to excitation at its own natural frequencies—not at that of the rack structure—since it is a separate structure and not attached to the rack. As the fuel moves within the rack and as the rack moves relative to the pool, the water between these structures is moved by them. The acceleration of the water introduces hydraulic loads on the structures which results in a lowering of natural frequencies of fuel and rack. These hydrodynamic effects are accentuated when the interacting submerged structures are in close proximity (small gaps).

The nonlinear time-history method was developed by C-E for use in spent fuel rack analyses because the linear response spectrum method does not properly characterize the fuel-to-fuel rack-to-pool interaction and, as demonstrated later in this paper, it may yield incorrect results.

THEORY

To aid in understanding the analysis method requirements corresponding to the physical problem, consider the following simplified analog of the spent fuel rack problem (see Figure 2). The three concentric cylinders represent the pool (P), the rack (R), and the fuel (F). There is water between the fuel and the rack, and between the rack and the pool. The connection (spring K_G) between the fuel and the rack represents the gap between these structures as well as the impact stiffness with which the fuel spacer grids interact

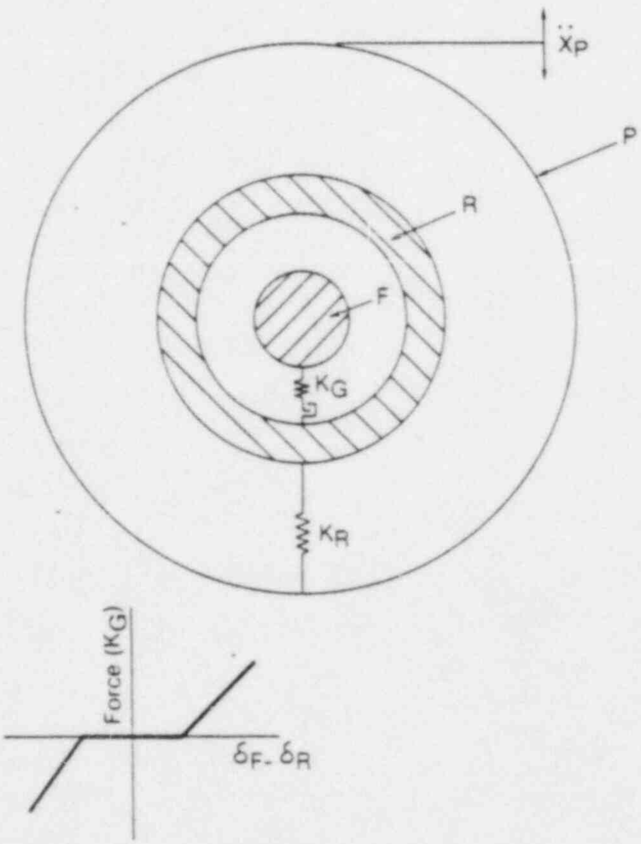


Figure 2: Simplified Analog of Spent Fuel Rack Physical Problem

with the rack when in contact. The connection (spring K_R) between the rack and the pool represents the manner in which the rack is supported by the pool. Nomenclature is as follows:

$\ddot{X}_P$ = seismic excitation (acceleration time-history) at spent fuel pool elevation
$\ddot{\delta}_R$ = acceleration of rack (relative to pool)
$\ddot{\delta}_F$ = acceleration of fuel (relative to pool)
δ_R = displacement of rack (relative to pool)
δ_F = displacement of fuel (relative to pool)
M_R = mass of rack
M_{R_O} = mass of water displaced by rack
M_{R_C} = mass of water contained within rack
M_F = mass of fuel
M_{F_O} = mass of water displaced by fuel
$F_{R_{IN}}$ = fluid force on inner boundary of rack
$F_{R_{OUT}}$ = fluid force on outer boundary of rack
$F_{F_{OUT}}$ = fluid force on outer boundary of fuel
K_R, K_G = as defined above
$\alpha_1, \alpha_2, \beta, \gamma$ = factors describing the effect of geometric proximity of hydrodynamics

With reference to the above nomenclature and Figure 2, and neglecting damping terms for purposes of simplifying discussion, the following equations of motion can be developed:

$$M_R(\ddot{X}_P + \ddot{\delta}_R) = -K_R(\delta_R) + K_G(\delta_F - \delta_R) + F_{R_{OUT}} + F_{R_{IN}}$$

$$M_F(\ddot{X}_P + \ddot{\delta}_F) = -K_G(\delta_F - \delta_G) + F_{F_{OUT}}$$

The fluid forces are given by:

$$F_{R_{OUT}} = M_{R_O}(\ddot{X}_P - \alpha_1 \ddot{\delta}_R)$$

$$F_{R_{IN}} = M_{R_C}(-\ddot{X}_P + 2\ddot{\delta}_F - \alpha_2 \ddot{\delta}_R)$$

$$F_{F_{OUT}} = M_{F_O}(\ddot{X}_P + 2\ddot{\delta}_R - \alpha_2 \ddot{\delta}_F)$$

Substitution of these expressions for fluid forces into the two equations of motion and simplification of terms yields the required coupled equations corresponding to the physical problem:

$$(M_R + \alpha_1 M_{R_O} + \alpha_2 M_{R_C})\ddot{\delta}_R - (2\beta M_{R_C})\ddot{\delta}_F + (K_R + K_G)\delta_R - K_G\delta_F = -(M_R + M_{R_C} - M_{R_O})\ddot{X}_P$$

$$-(2\gamma M_{F_O})\ddot{\delta}_R + (M_F + \alpha_2 M_{F_O})\ddot{\delta}_F - K_G\delta_R + K_G\delta_F = -(M_F - M_{F_O})\ddot{X}_P$$

The equations account for the gap between the fuel and the rack, the hydrodynamic coupling between the submerged structures and impacting between structures. The complete equations of motion (including damping) corresponding to the physical situation are modeled and solved

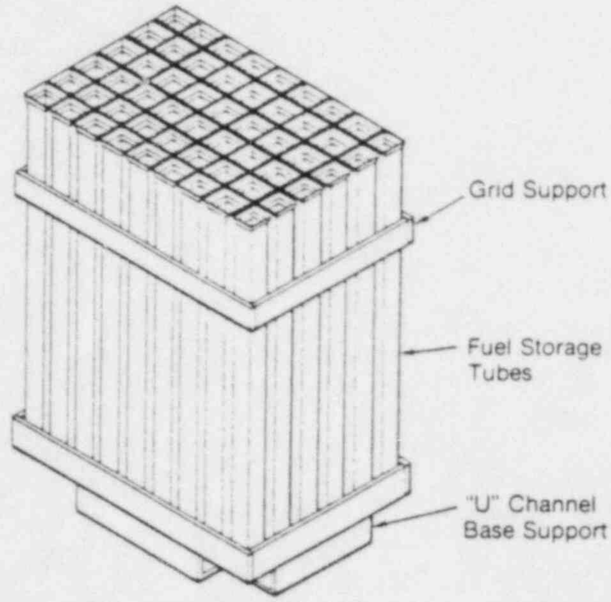


Figure 3: C-E HI-CAP Spent Fuel Rack Module

through the use of CESHOCK. In contrast to the above, the response spectrum method can accommodate only a single uncoupled equation for the response of a one-degree-of-freedom system. Modifying the response spectrum method to include an approximation of the effect of water on frequency, the analogous equation of motion for the system of Figure 3 that corresponds to the response spectrum method of analysis:

$$(M + M_C + M_D)\ddot{\delta} + K\delta = -(M + M_C)\ddot{X}_P$$

Here the representation of the system is clearly incomplete, with all sorts of approximations (of unknown effect) required to select the single values of mass, stiffness (linear only), etc., allowed. Comparison with the two equations above demonstrates the point that the response spectrum method does not model the real, physical situation. For example, it does not account for the gap between the fuel and the rack, which causes the system to have different natural frequencies (and to respond to different frequencies of excitation) and allows fuel to rack impacting to occur. Also, it does not account for the hydrodynamic coupling between the fuel and rack, with the introduction of interactive fluid forces.

RESULTS

A number of spent fuel rack seismic analyses have been performed by C-E, covering a wide range of rack designs and seismic excitations. The two basic types of spent fuel racks offered by C-E are shown in Figures 3 and 4. The High Capacity (HI-CAP) design in Figure 3 is composed of square storage cavities fabricated from stainless steel plate with each cavity capable of accepting one fuel assembly. The storage cavities are structurally connected to form modules from the use of channels, plates and chevron beams which provide the load-carrying frame and maintain spacing be-

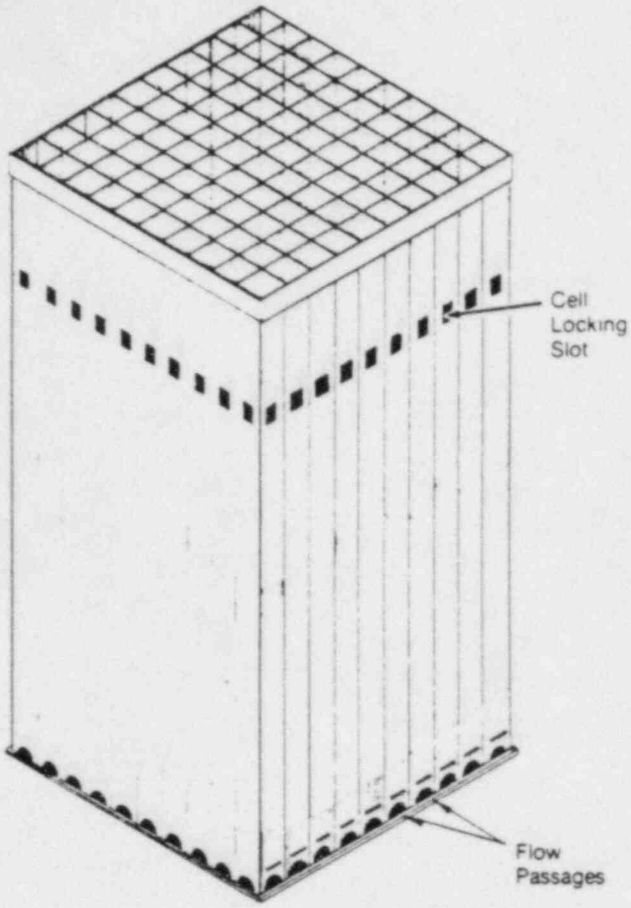


Figure 4: C-E Super HI-CAP Spent Fuel Storage Module

tween storage cavities. The C-E standard Super HI-CAP spent fuel storage rack shown in Figure 4 is a stainless steel monolithic honeycomb structure with square fuel storage locations. The fuel assembly storage cells are welded together to permit the assembled modules to be load-bearing structures as well as the storage cell enclosures. Each individual cell is a structural member and serves as a guide and retainer for a Neutron Poison Insert or a Consolidated Fuel Box. Following is a summary of representative results from nonlinear time-history analyses (utilizing CESHOCK), compared with corresponding response spectrum method analysis results.

Figure 5 shows several different seismic excitations used in obtaining the results. The response spectra are shown only to illustrate the differences in the excitations corresponding to seven sites; time-histories for these sites were used in the CESHOCK analyses.

Figures 6 and 7 represent two typical CESHOCK models. Model A corresponds to a freestanding HI-CAP design and Model B represents a freestanding Super HI-CAP design. For Model A, the fuel is modeled by masses 1 through 7 and springs K_{F1} through K_{F6}; the rack is modeled by masses 8 through 14 and springs K_{R1} through K_{R6}; the hydrodynamic coupling between the rack and the fuel and the rack and pool is represented by the couplings — H; the fuel-to-rack gaps and fuel-to-rack impact characteristics are modeled by the nonlinear springs K_{G1} through K_{G6}; the frictional restraint between the fuel and the rack and that between the rack and the pool are represented by the friction couplings $F_{F\text{-}R}$ and $F_{R\text{-}P}$, respectively. The corresponding parameters for Model B are shown in Figure 7.

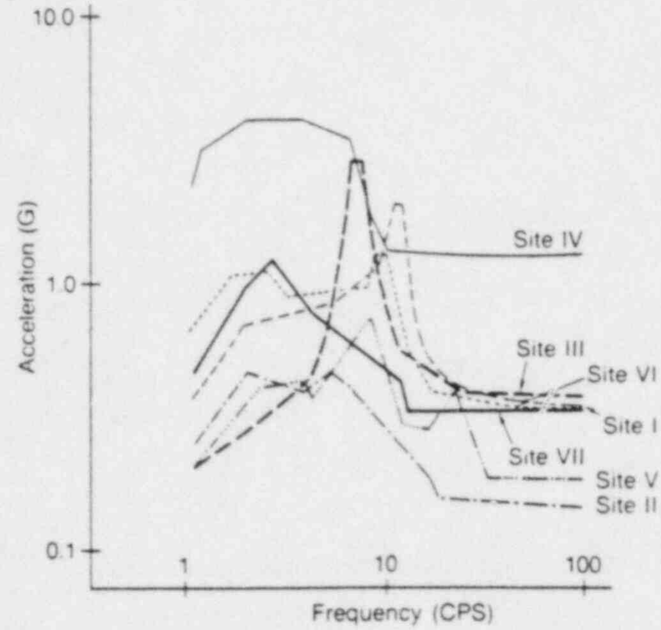


Figure 5: Spent Fuel Pools Seismic Response Spectra

Figure 8 is a brief segment of typical displacement responses (Model A) to the seismic excitation corresponding

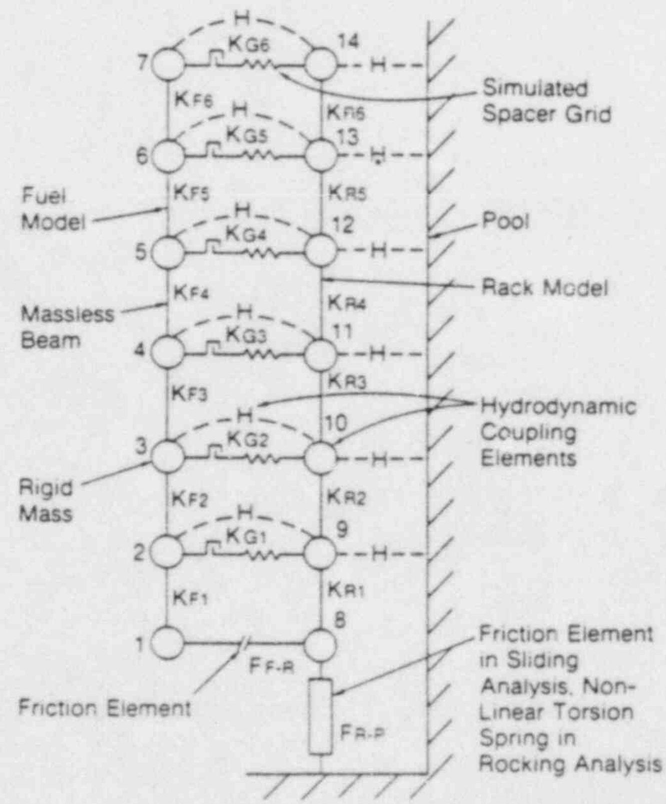


Figure 6: HI-CAP Fuel Rack Nonlinear CESHOCK Model

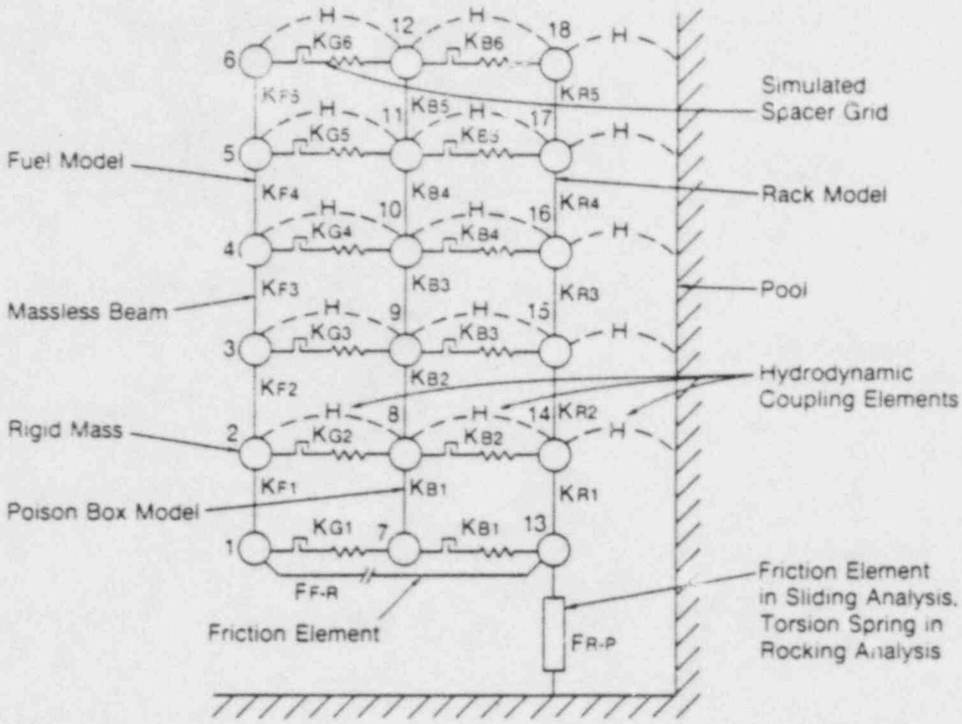


Figure 7: Super HI-CAP Fuel Rack Nonlinear CESHOCK Model

to a HI-CAP design for site III. Figure 9 provides a similar response for a Super HI-CAP design (Model B) for site VII. Note the low-amplitude, high-frequency response of the rack portion of the model in contrast to the high-amplitude, low-frequency response of the fuel. Typical fuel impact load pulses and their effect on peak base shear are seen by comparing the response quantities shown also on Figure 9. The peak base shears occurs just after the time of peak fuel impact loads.

Table 1 presents a tabulation of seismic loads developed within the rack and transmitted to the pool for a number of designs and the sites of Figure 5. The load values have been normalized. The first column identifies the site and the rack design. Four variations of a HI-CAP design (A – D) and 3 variations of a Super HI-CAP design (E – G) are presented. Four variations of HI-CAP design D are shown; the original version, a second version in which dynamic analysis parameters were changed by 10% (e.g., fuel stiffness), a third version with one-fourth the original fuel-to-rack gap, and a fourth version with an impact spring stiffness ten times that of the original. Four variations of Super HI-CAP design F are presented which include variation in gaps, impact stiffness and hydrodynamic mass representation. Design G shows results for both a stiff and a soft rack support structure. The second column presents the seismic loads obtained from the CESHOCK analyses. The third column presents the corresponding seismic loads obtained, for comparative purposes, by means of response spectrum method analyses. The last column gives the ratios of loads obtained by the two methods.

Comparison of results from nonlinear time-history analyses (fuel to rack interaction analyses) with those from response spectrum analyses (refer to Table 1) shows that the response spectrum method may give incorrect results. The results demonstrate the importance of the interaction between fuel and racks. The interaction is caused by the relative motion between the fuel and rack, through the water-filled gaps, and impacting of the fuel and rack.

TABLE 1

IDENTIFIER SITE/DESIGN			NORMALIZED REACTION LOAD PER CELL (1) TIME-HISTORY NONLINEAR ANALYSIS	(2) RESPONSE SPECTRUM METHOD	RATIO (1)/(2)
	DESIGN A (HI-CAP)		5.92	7.42	0.79
I	DESIGN B (HI-CAP)		8.17	15.66	0.52
	DESIGN C (HI-CAP)		4.21	4.08	1.03
II	DESIGN A		1.99	1.79	1.11
II			3.00	1.00	3.00
III	DESIGN D (HI-CAP)		2.73	2.56	1.07
IV			17.08	8.74	1.95
		ORIG.	3.00	1.00	3.00
II	DESIGN D	Δ 10%	2.83	1.00	2.83
		1/4 GAP	2.08	1.00	2.08
		$10xK_G$	4.27	1.00	4.27
V	DESIGN E (SUPER HI-CAP)		3.84	2.72	1.41
		ORIG.	9.26	4.06	2.28
		GAP BOX/RACK	11.83	4.06	2.91
VI	DESIGN F (SUPER HI-CAP)	GAP BOX RACK DIFF. HYDRO	6.85	4.06	1.69
		GAP BOX/RACK DIFF. HYDRO 8 FUEL GAP 25 K_B (BOX)	7.93	4.06	1.95
VII	DESIGN G	STIFF BASE	9.26	3.40	2.72
		SOFT BASE (SUPER HI-CAP)	4.19	5.31	.79

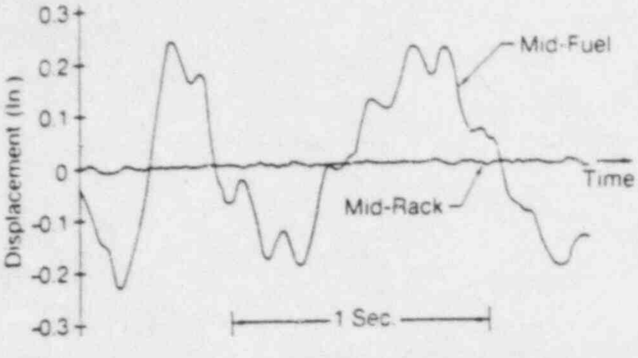


Figure 8: CESHOCK Displacement Response For HI-CAP Fuel Rack

FUEL CONSOLIDATION

Nonlinear time-history analysis is also used by C-E to analyze consolidated fuel rack designs. The consolidated fuel racks consist of the Super HI-CAP design with consolidated fuel rods in each cell. A typical consolidated fuel arrangement is shown in Figure 10. A consolidated fuel canister with a closely compacted array of fuel rods contained within it exhibits nonlinear characteristics similar to standard fuel assemblies. Separate models must be developed to represent different degrees of compaction and, for cases of less than complete compaction, fuel rod impacting must be accounted for. The hydrodynamic effects on fuel canister natural frequency and damping are also incorporated into the model. Basic modeling information concerning the dynamic interaction between the consolidated fuel and the can is provided only by testing. Because the interaction between consolidated fuel and the can is similar to standard fuel, the nonlinear time-history method is used to analyze consolidated fuel rack designs. The use of the response spectrum method for consolidated fuel rack designs may lead to incorrect results.

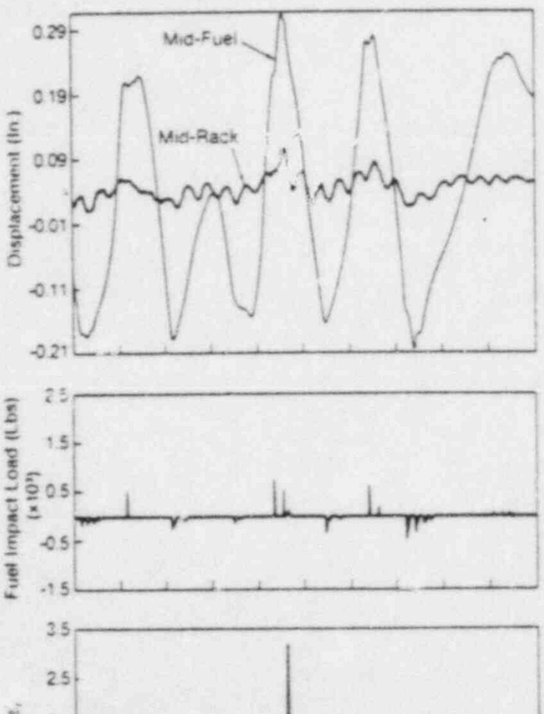


Figure 9: CESHOCK Response Parameters For Super HI-CAP Fuel Rack

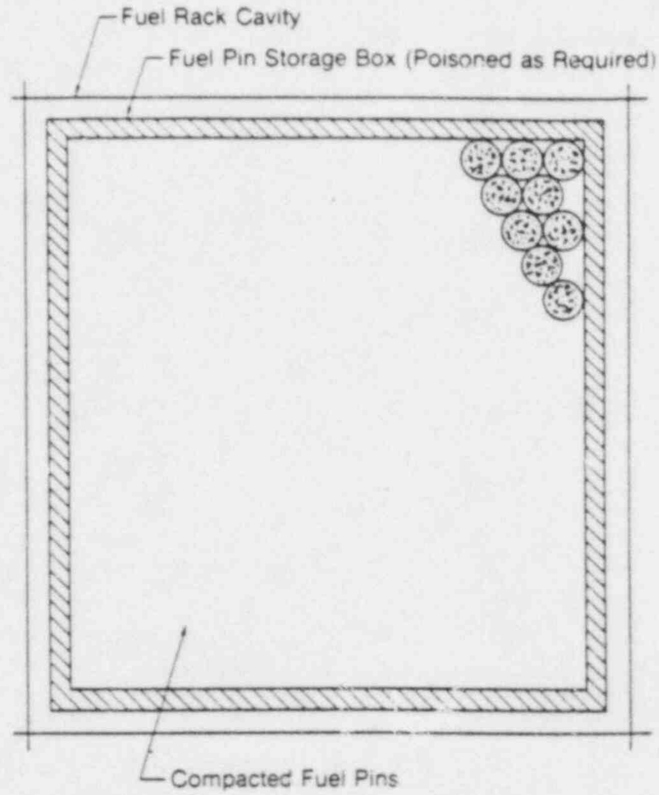


Figure 10: Consolidated Fuel Pin Arrangement

With consolidation factors of 2 or greater under consideration by many utilities, it is the job of the analyst to minimize storage pool design loads due to earthquakes. Because most pools were not designed for consolidation, they cannot readily accept higher loads. To minimize modifications to strengthen pools or to show that modifications are unnecessary, there are a number of steps the analyst can take. Some of the methods offered by C-E to obtain margin for consolidation designs are listed below:

1. Re-analyze the Auxiliary Building with Soil Structure Interaction.
2. Perform Finite Element Analysis of the Pool.
3. Couple the Fuel Rack Model to the Auxiliary Building Model.
4. Detune the Consolidated Fuel Racks from the Earthquake.

b. Describe how the analysis for the Licensee's report differed from that presented in the referenced technical paper.

The analysis for the Licensee's report differed from that presented in the refrenced paper in several respects. Most importantly, the analysis for the Licensee's report was done using models based on the Millstone 2 rack module designs and pool layout and site specific acceleration time history data. The actual Millstone 2 site specific model is described in the response to question #3.

c. Provide a copy of the reference to expedite the review.

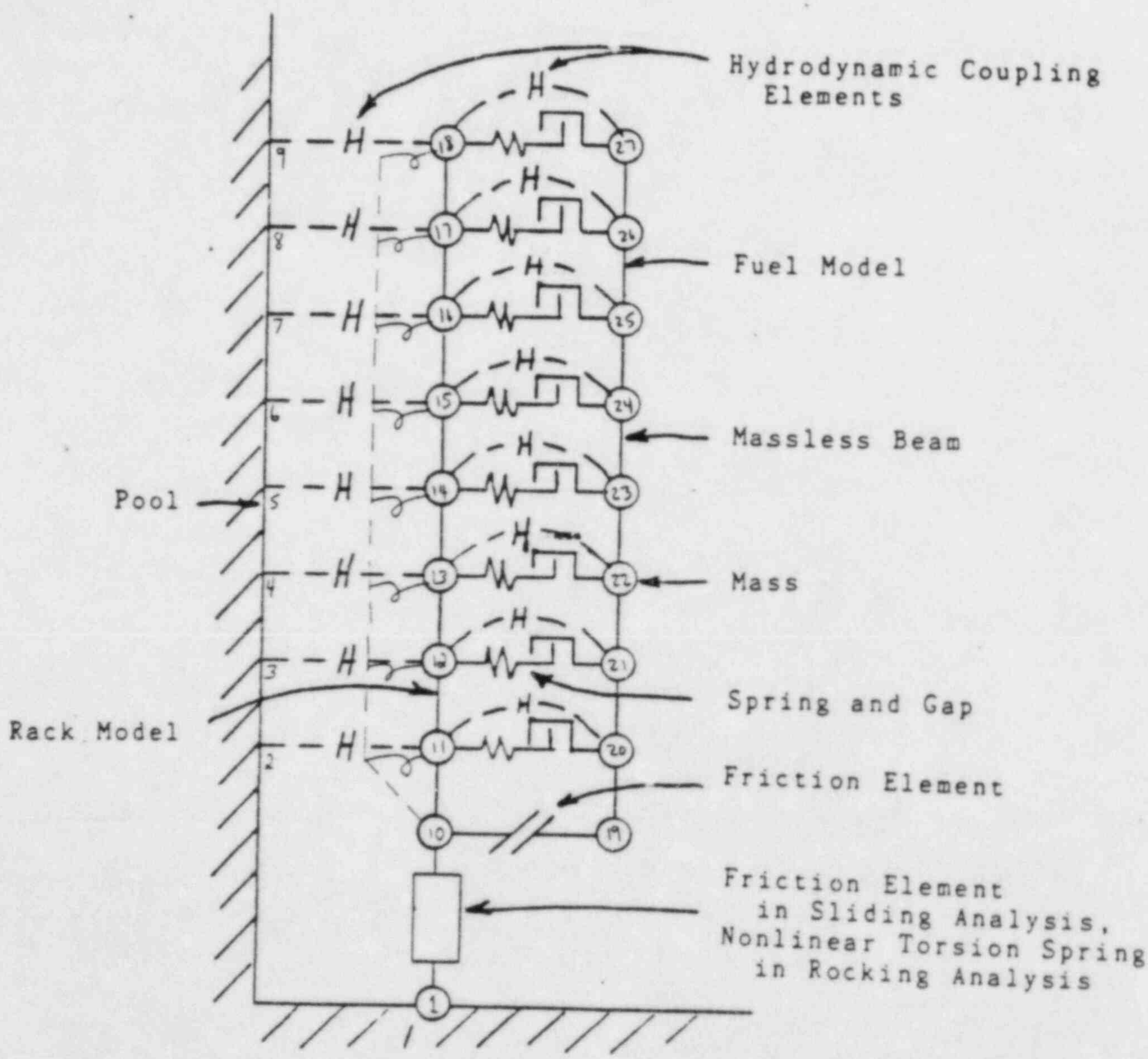
A copy of the referenced paper is attached.

3. Provide a full description of the mathetical model used for the non-linear rack module analysis.

A schematic description of the mathematical model used for the non-linear rack module analysis is shown in Figure 1. 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 wer 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 betwen the rack module nodes and the pool structure nodes, and between the fuel nodes and the rack moduel 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 interace of the module based 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.

FIGURE 1

CESHOCK Model of Millstone 2 Region II
7 X 9 Spent Fuel Rack Module



4. In addition to not providing the mathematical model for the non-linear dynamic displacement analysis, the Licensee did not indicate the relationship of the rack module analyzed to its adjacent rack modules.

The following information is required:

a. Describe and justify how in-phase and/or out-of-phase motion with adjacent rack modules was considered and implemented

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 excition, the higher rack module frequencies resulting from the in-phase node 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 hydrdynamic masses produced by out-of-phase motion.

b. Describe fully how hydro dynamic coupling to adjacent rack modules was considered and justify the use of the theoretical basis employed.

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 hdrodynamic virtual mass terms based on the odel 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-dimenstional 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 Computing Added Mass Coefficients," Argonne National Laboratory Report No. ANL-LT-78-49, September 1978.

c. Describe how the gap between adjacent rack modules was apportioned to each rack module and list the values for the racks analyzed.

A procedure of apportioning gaps between adjacent rack modules was not employed in the analysis.

d. Provide numerical comparisons of rack displacements (at the top of the rack if that is the point of maximum displacement) to the apportioned clearance.

No method of apportioning intermodule clearances was used. The peak intermodule clearances was used. The peak intermodule relative displacement, however, was determined to be 1.776 inches. This is less than the actual clearance between modules.

e. Where frequencies may be cited, please provide a copy of each reference with the response to expedite the review.

The cited references are attached.

5. With respect to the modeling of impact between the fuel assembly and a rack cell in the non-linear dynamic analysis:

a. Provide the data and structural premise upon which impact stiffness was based.

C-E uses a gap-spring element to model the impact between the fuel assembly and the rack cell in a nonlinear dynamic analysis. The spring represents the spacer grid one-sided impact stiffness with the appropriate gap. C-E determines fuel assembly one-sided impact stiffnesses using full-scale fuel assembly pluck impact tests and model-test correlations of the test data with analytical results. The value of the spacer grid impact stiffness for the Westinghouse fuel assemblies that was provided to C-E by Northeast Utilities was greater than that for a C-E fuel assembly and was conervatively used in the nonlinear dynamic analysis.

b. Provide the value of impact damping used, if greater than the nominal structural damping used in the anlaysis, and provide documentation justifying that damping value.

Impact damping was conservatively not used in the analysis.

6. The Licensee did not indicate what range of friction coefficient values was used in the non-linear displacement analysis between the rack mounting feet and the pool floor liner:

a. Provide the range of friction coefficient used and describe the procedures used to determine the friction coeficient that produces the maximum rack displacement.

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 liner until a minimum dynamic coefficient of friction of 0.28 is reached at a relative velocity of the module base with respect to the pool liner until a minimum dynamic coefficient of friction of 0.28 is reached at a relative velocity of 2.5 in/sec. For relative velocities above 2.5 in/sec., the minimum dynamic coefficient of friction applies.

b. Justify and document the validity of the range of friction coefficient used.

The friction values used are based on the following sources:

i) data from Combustion Engineering laboratory tests,

ii) 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 Pressurizeed 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 1973., and

iii) 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.

Question #7a - The Licensee did not indicate how the results from the non-linear displacement analysis was introduced to the stress analysis model.

b - Provide full description of the load selection process and how the vertical and lateral dynamic loads on each rack mounting foot, as well as rack dead weight, are considered during rack lift-off in the stress analysis model.

Answer #7a - The results of the non-linear 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 non-linear 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 Applicable	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 (Z-direction)	26.02	26.82

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

b. The analysis to determine the structural adequacy of the fuel storage module under tipping was conducted using the following technique: 1) Two loading conditions were applied to the SAP IV model these are: 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 non-linear 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.

8. Non-linear analyses, especially those involving impact of bodies as occurs between the fuel assemblies and the rack module, and between the rack mounting feet and the pool floor during lift-off, generally require additional procedures such as repeated solutions using a range of integration time steps to assure that the solution is both stable and fully converged. This is important because integration procedures that have yielded a valid solution do not necessarily remain stable for all solutions. The Licensee made no mention of this important point.

   a. Provide a description of the methods used to assure that a valid solution of the non-linear analysis was reached for all cases investigated.

   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 timestep 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 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 non-linear analysis indicates such a situation does exist.

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.)

Question #9 - At the bottom of page 22 of the Licensee's report, the Licensee stated that "The component stress on each element resulting from the application of each directional load is combined by the square root sum of the squares method". No computed stresses or allowable stresses were provided.

Answer #9a - 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.

b. The loads and load combinations used in the structural analysis of the spent fuel racks are listed below and are consistent with NRC guidance in "Review an Acceptance of Spent Fuel Storage and Handling Applications".

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^1	Faulted Condition Limits of NF 3231. 1c

The abbreviations in the table above are those used in Section 3.8.4 of the Standard Review Plan where each term is defined except for Ta which is defined as the highest temperature associated with the postulated abnormal design conditions.

c. The maximum stress values associated with the analyses performed for the Millstone II spent fuel racks are provided below. These values are based upon the SSE load condition. Except for the adjustment screw, the stresses associated with the SSE load condition are lower than the OBE allowable stress limits and therefore are acceptable for both the OBE and SSE conditions. The stress values for the adjustment screw and their allowable stress limits are provided for both OBE and SSE condition. The design margin is defined as $(\frac{\text{allowable}}{\text{actual}} - 1) \times 100\%$.

NOTE: In most cases the maximum stress is associated with SSE load condition, while the allowable stress is for the OBE condition.

Maximum Stress

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	= 13,665 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. %
Combined axial compress. plus bending	$= \frac{f_a}{F_a} + \frac{f_b}{F_b} = .736$	1	20.8%

SSE Condition	Maximum Stress	SSE 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	$= \frac{fa}{Fa} + \frac{fb}{Fb} = .828$	1	20.8%

SSE Condition	Maximum Stress	SSE Allowable Stress	
Thread shear	= 6,710 psi	11,000 psi	63.9%

Question #10 - With respect to fuel handling accidents as addressed by the Licensee on page 23 of the report:

a. Provide analysis and justification as to why a spent fuel assembly falling through a rack cell and impacting the bottom of the cell "will not affect the primary function of the racks ....".

b. Provide the approach, the assumptions, the data employed, and the results of analysis performed to assure that a fuel assembly dropped through a rack storage cell will not penetrate the bottom of the rack module, or, if it does penetrate the bottom of the rack module that it will not damage the pool liner.

c. For the case of a crane uplift accident, provide the method of analysis employed, and the criteria by which the results were judged to be acceptable, including identification and documentation of the allowable stresses.

Answer #10a

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.

b. 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.

c. 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.

QUESTION

11.a. Provide sketches and drawings of the portions of the pool and auxiliary building structures to be modeled.

Response

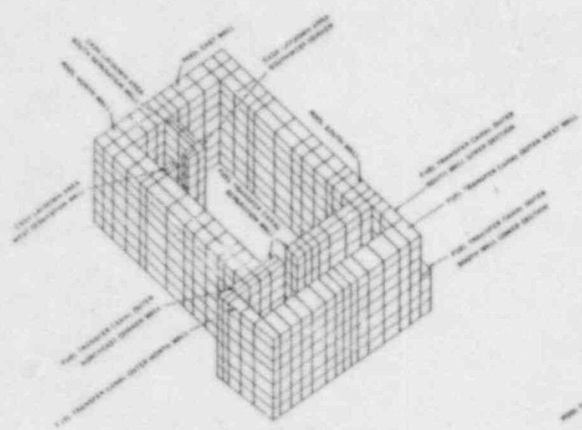
This section provides the finite element plots of the spent fuel pool, pool liner, and associated auxiliary building components covered by the analyses.

The models were derived based upon information supplied on the following NUSCO-Millstone Unit No. 2 drawings: 25203-11090 through 11099, 11104, 11106, 11107, 11112, 11126, 11127, 27016, 27018, 27019, 270122, 51044, 51045.

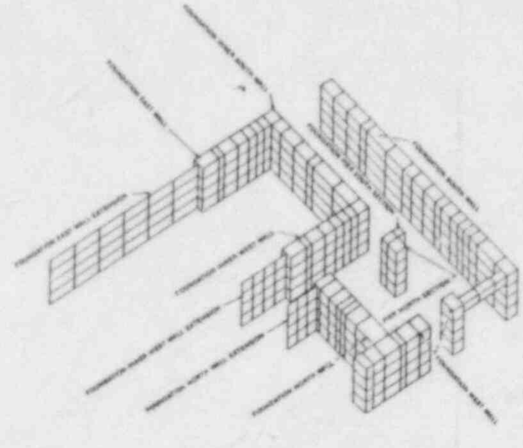
The spent fuel pool and associated auxiliary building components model contain over 9,600 degrees of freedom. Sketches are also provided of the floor liner plate model used in the analyses.

MILLSTONE POINT - UNIT 2

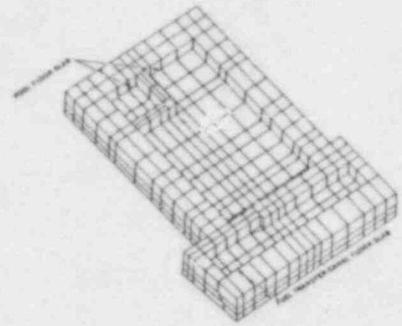
SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL



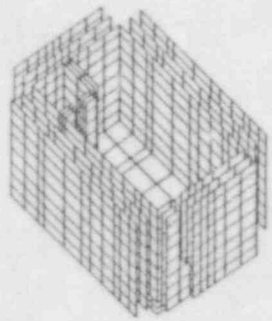
UPPER PORTION OF POOL WALLS



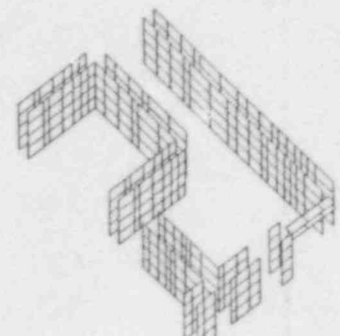
FOUNDATION WALLS



POOL FLOOR SLAB



POOL SURFACE MEMBRANES



FOUNDATION SURFACE MEMBRANES

FIGURE 1

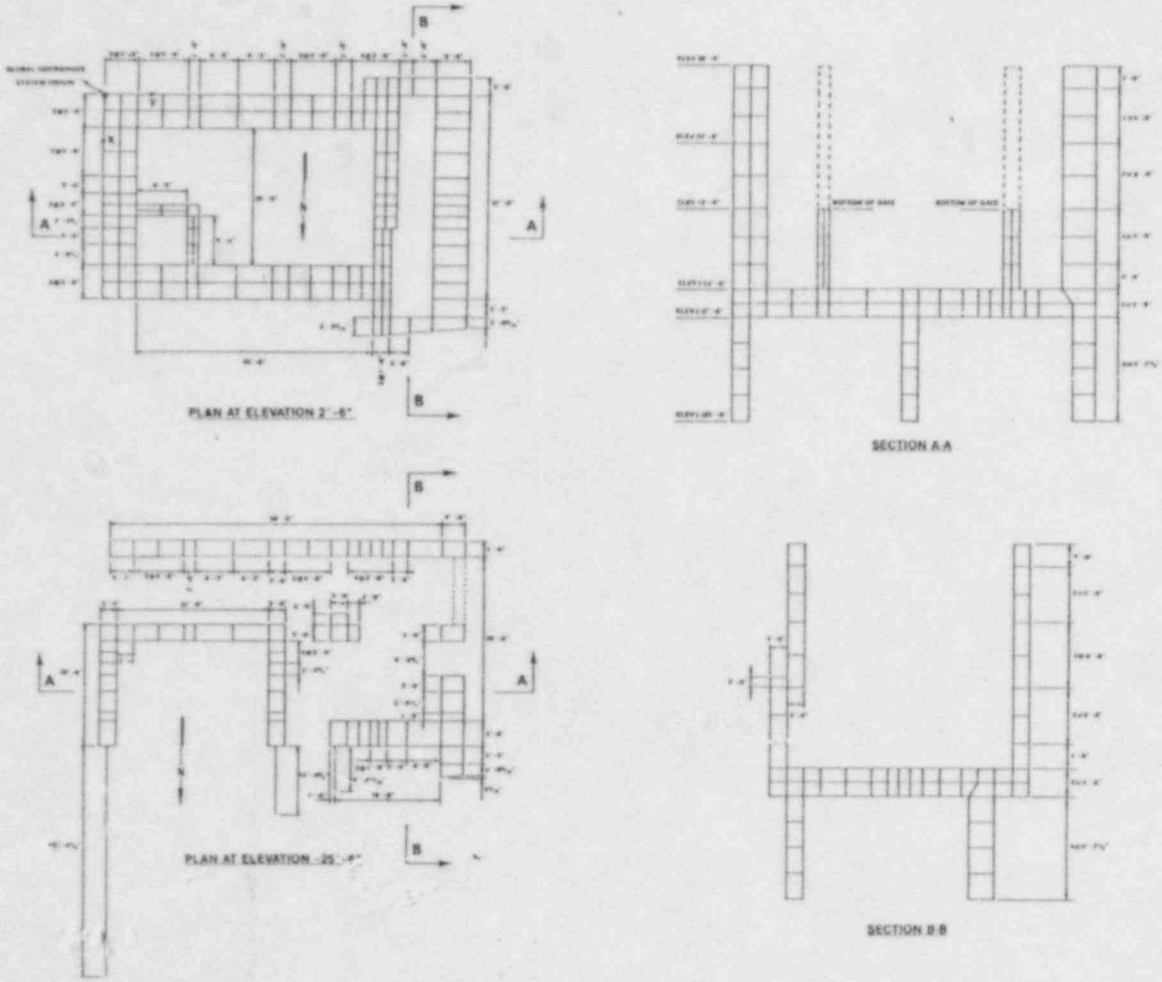
ISOMETRIC VIEWS AND KEY DIAGRAMS

NUS-01-015, REV 1
JULY 25, 1983
ENCLOSURE

SDT Structural Dynamics Technology, Inc.

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL



GENERAL NOTES

1) Reference Drawings: "Millstone Nuclear Power Station - Unit No. 2" 25203-11090 thru 11099, 11104, 11106, 11107, 11112, 11126, 11127, 27016, 27018, 27019, 27022.

2) The following deviations from the reference drawings were made in areas not considered important to model behavior:
   - The position of the lower south wall of the fuel transfer canal area was moved 3" north.
   - The length of the lower south wall of the fuel transfer canal area was reduced to provide alignment with existing nodes.
   - The curvature in the outside face of the north wall of the fuel transfer canal area was eliminated.
   - The opening in the west foundation wall was adjusted to provide alignment with existing nodes.

3) Coordinate systems:
   - Global coordinate system origin is located at the southeast outside corner of the pool at elevation 0'-0".
   - Local membrane element coordinate systems for pool and foundation walls are defined such that the positive local z axis vector points toward the outside of the walls with the positive local y axis vector up. Local membrane element coordinate systems for the pool floor are defined such that the positive local z axis vector points down with the positive local x axis vector parallel to the positive global x axis.

4) Membrane elements are provided on the surfaces of the walls that are to be evaluated to code criteria. These membranes are assigned negligible thickness and therefore do not provide additional stiffness. Stresses obtained from these membranes are used along with stresses obtained from the solid elements to calculate resultant section forces and moments.

5) Shear stiffness of auxiliary building floors and walls that frame into pool and foundation walls are represented as general matrix elements.

6) Foundation wall extensions are modeled as membrane elements with their actual thickness specified.

7) All solid elements are defined using eight nodes. Where intersections of components are made with a discontinuity of nodes, such as, the intersection of the cask laydown separation walls with the pool walls, the non-continuous node is restrained to adjacent nodes such that the plane section through the non-continuous component remains plane.

8) The cask laydown area gate opening is centered in west separation wall by relocating nodes from elevation 12'-6" to top of pool as shown in Figure 8.

9) Revision 1 to Figures 1, 2 and 11 are for the opening in the west foundation wall. Revision 1 to Figure 7 revers[illegible] the Section heading for the north and south sections of the fuel transfer canal separation wall.

NUS-01-215, REV 1
JULY 25, 1983
ENCLOSURE

FIGURE 2

PLANS, SECTIONS, DIMENSIONS AND GENERAL NOTES

SDT Structural Dynamics Technology, Inc.

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

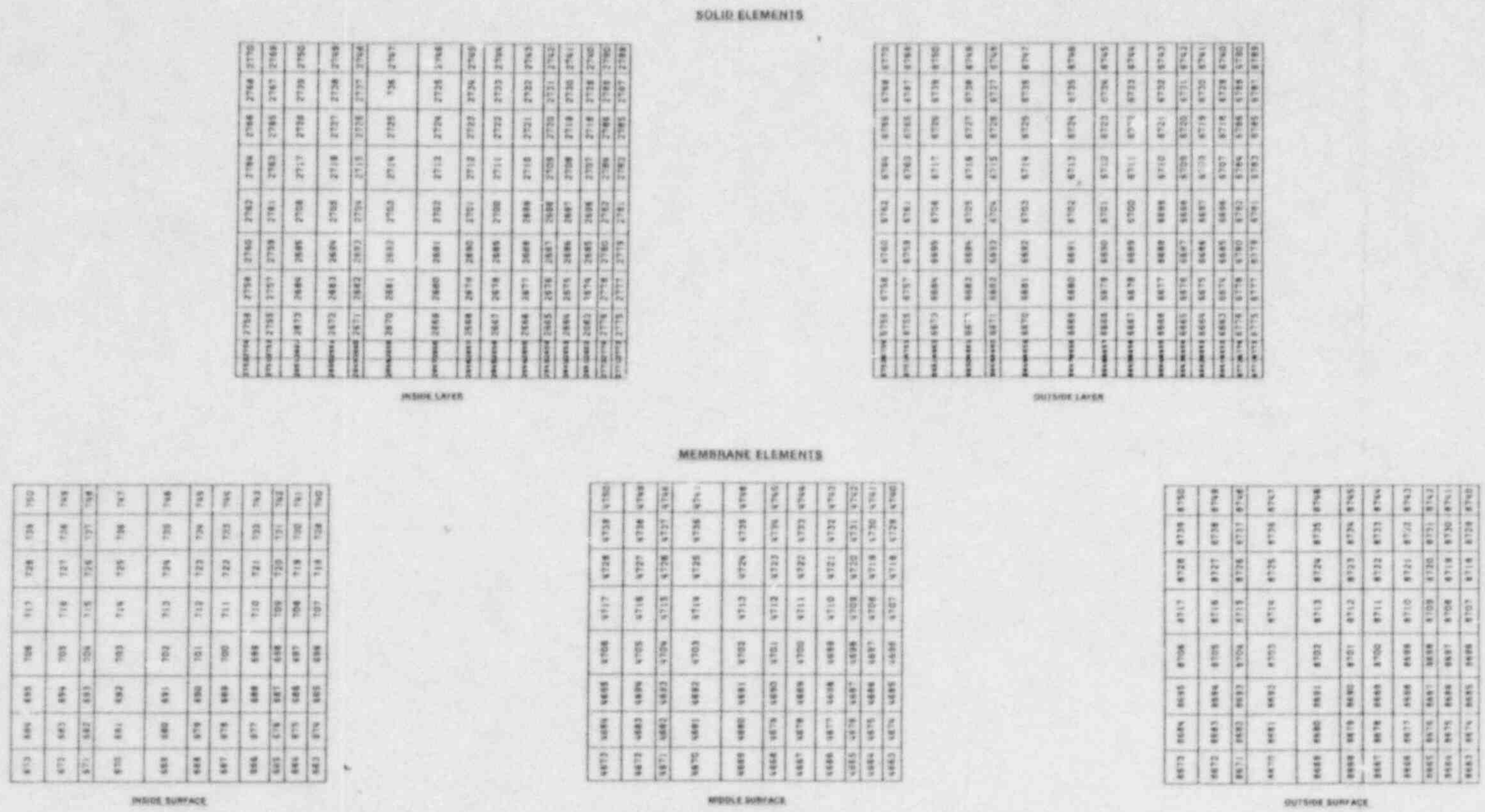


FIGURE 3

POOL SOUTH WALL ELEMENTS

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

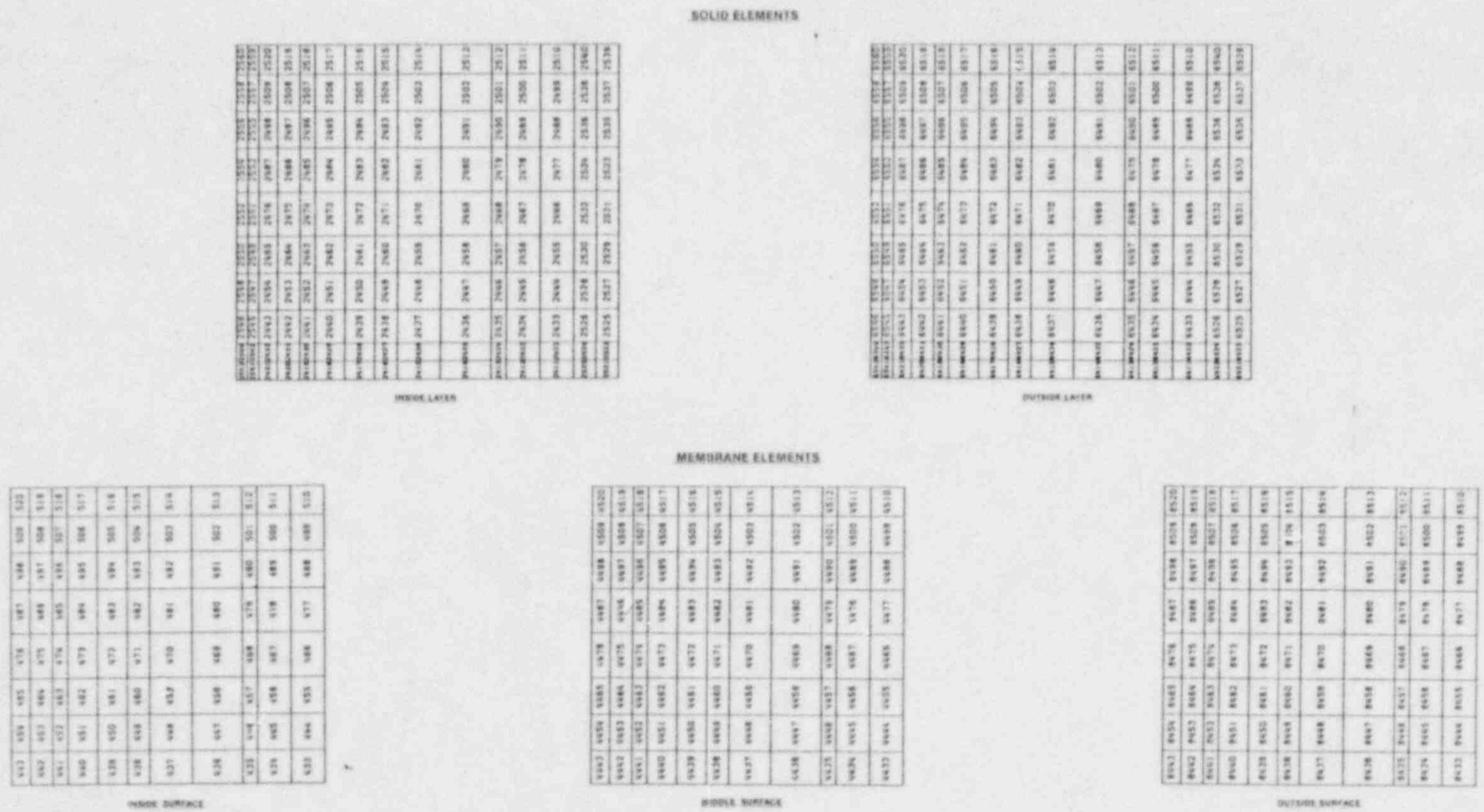


FIGURE 4

POOL NORTH WALL ELEMENTS

SDT Structural Dynamics Technology, Inc.

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

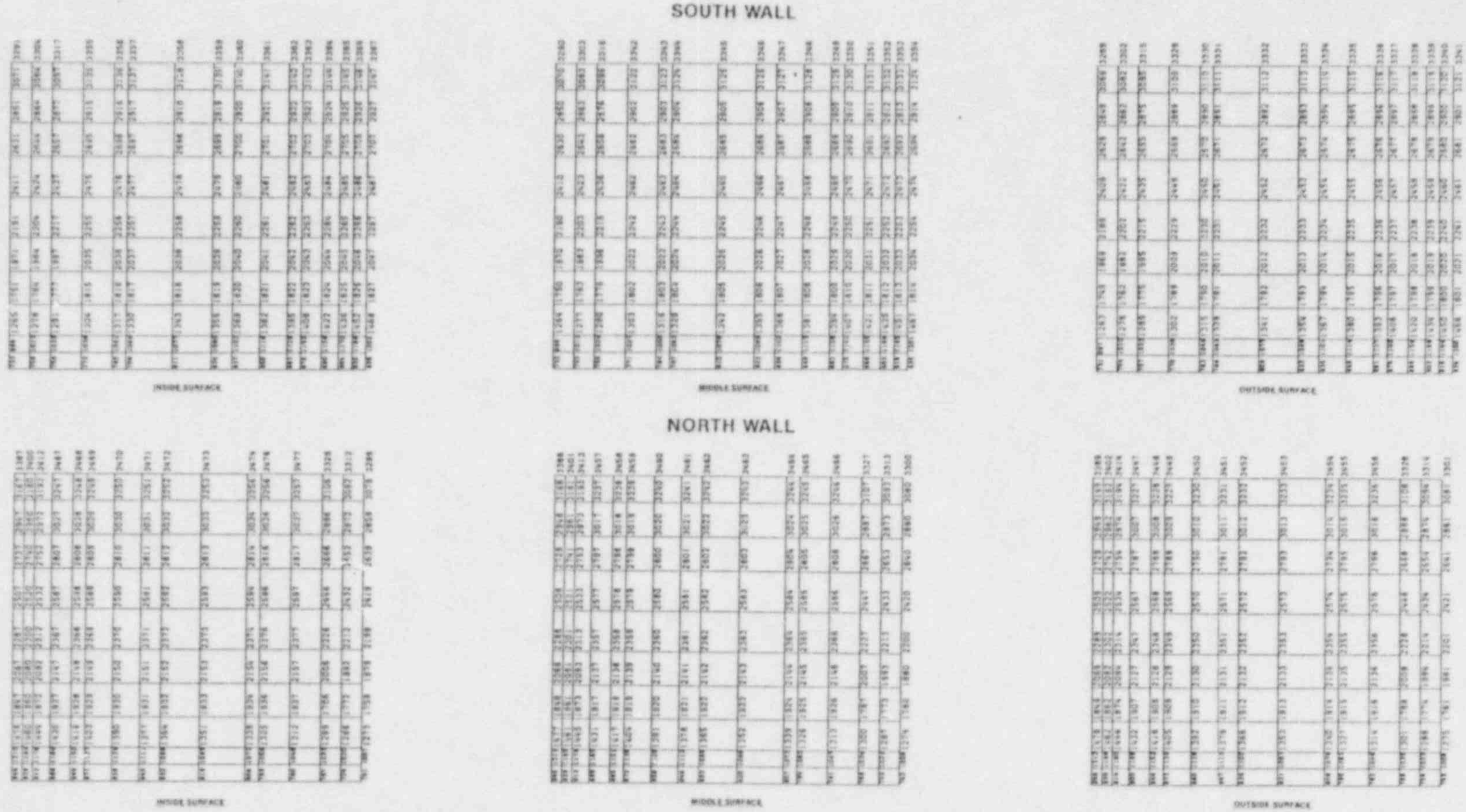


FIGURE 5

POOL SOUTH WALL AND NORTH WALL NODES

NUS-01-015
MARCH 25, 1983
ENCLOSURE

SDT Structural Dynamics Technology, Inc

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

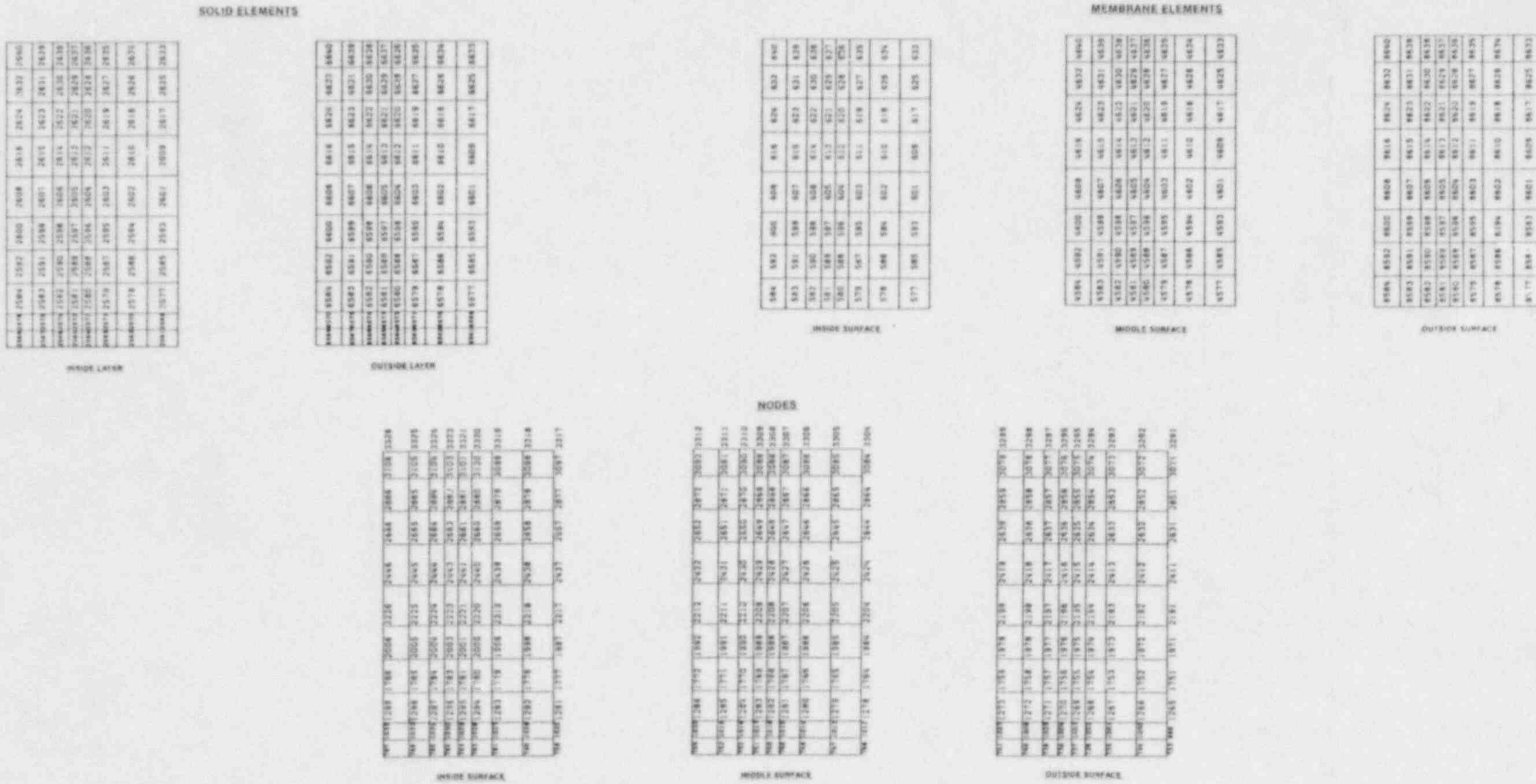


FIGURE 6

POOL EAST WALL

SDT Structural Dynamics Technology, Inc.

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

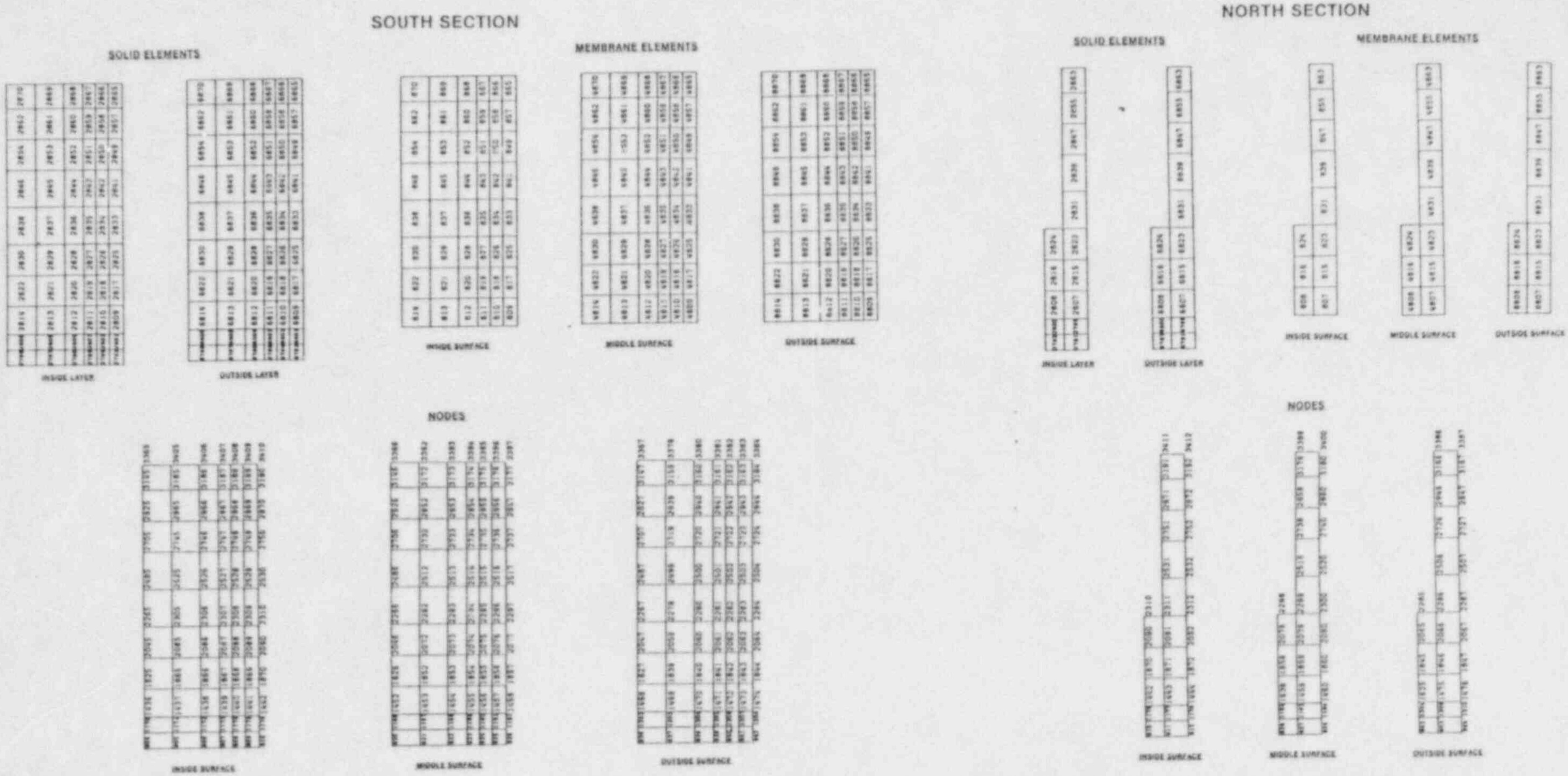


FIGURE 7

FUEL TRANSFER CANAL SEPARATION WALL

NUS-01-015, REV 1
JULY 25, 1983
ENCLOSURE

Structural Dynamics Technology, Inc.

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

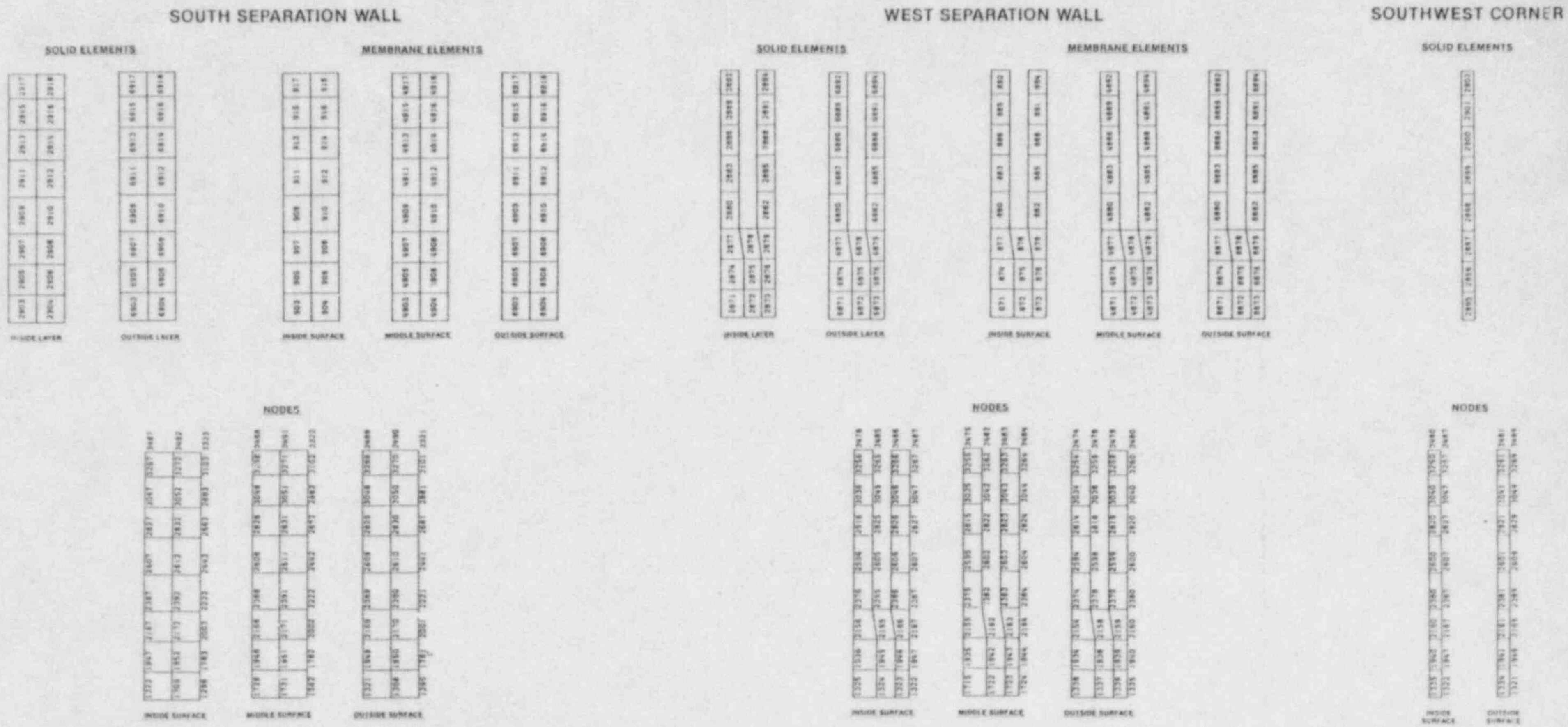


FIGURE 8

CASK LAYDOWN AREA SEPARATION WALLS

SDT Structural Dynamics Technology, Inc.

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

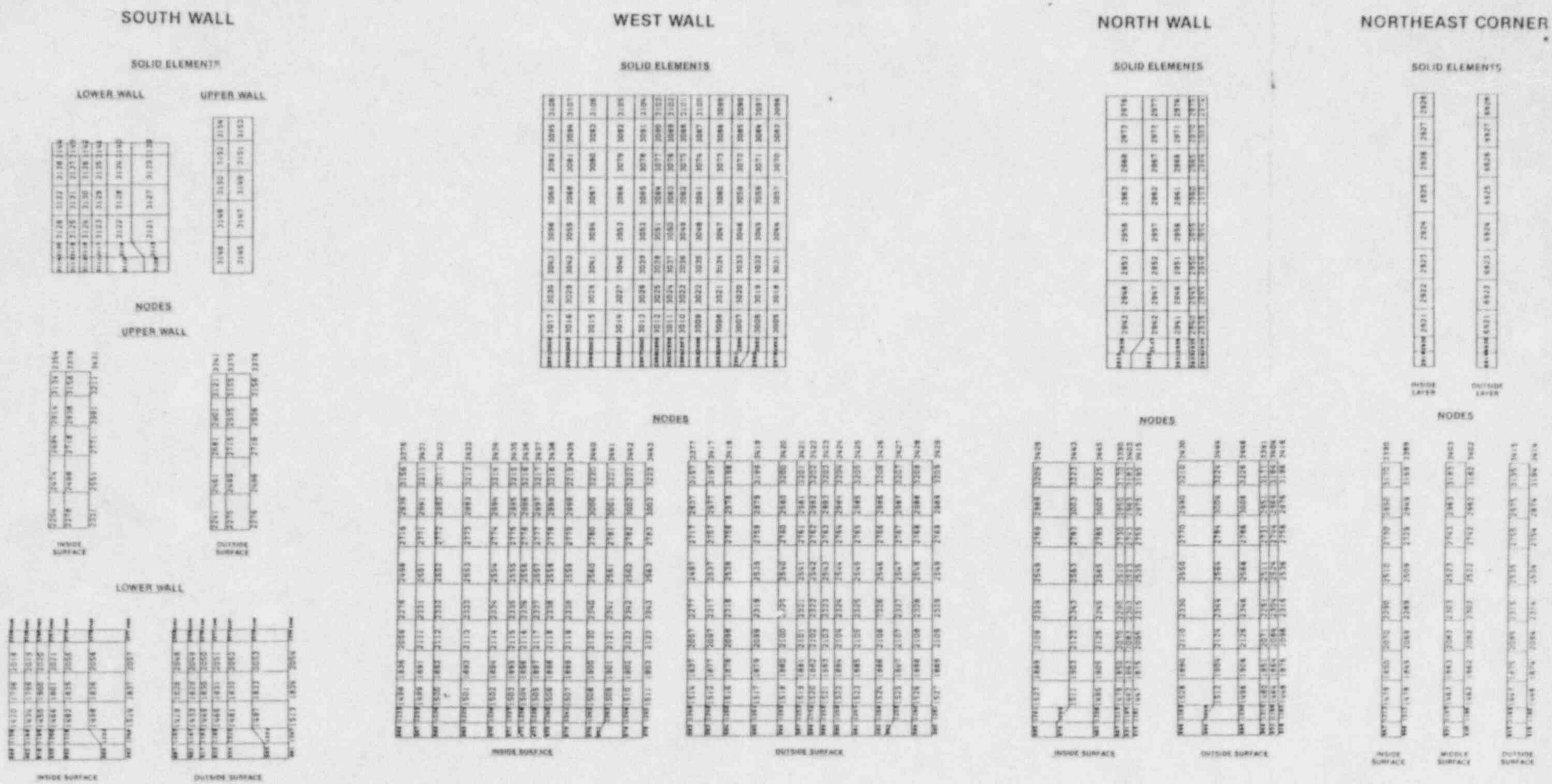


FIGURE 9

FUEL TRANSFER CANAL OUTER WALLS

NUS-01-015
MARCH 25, 1983
ENCLOSURE

Structural Dynamics Technology, Inc

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

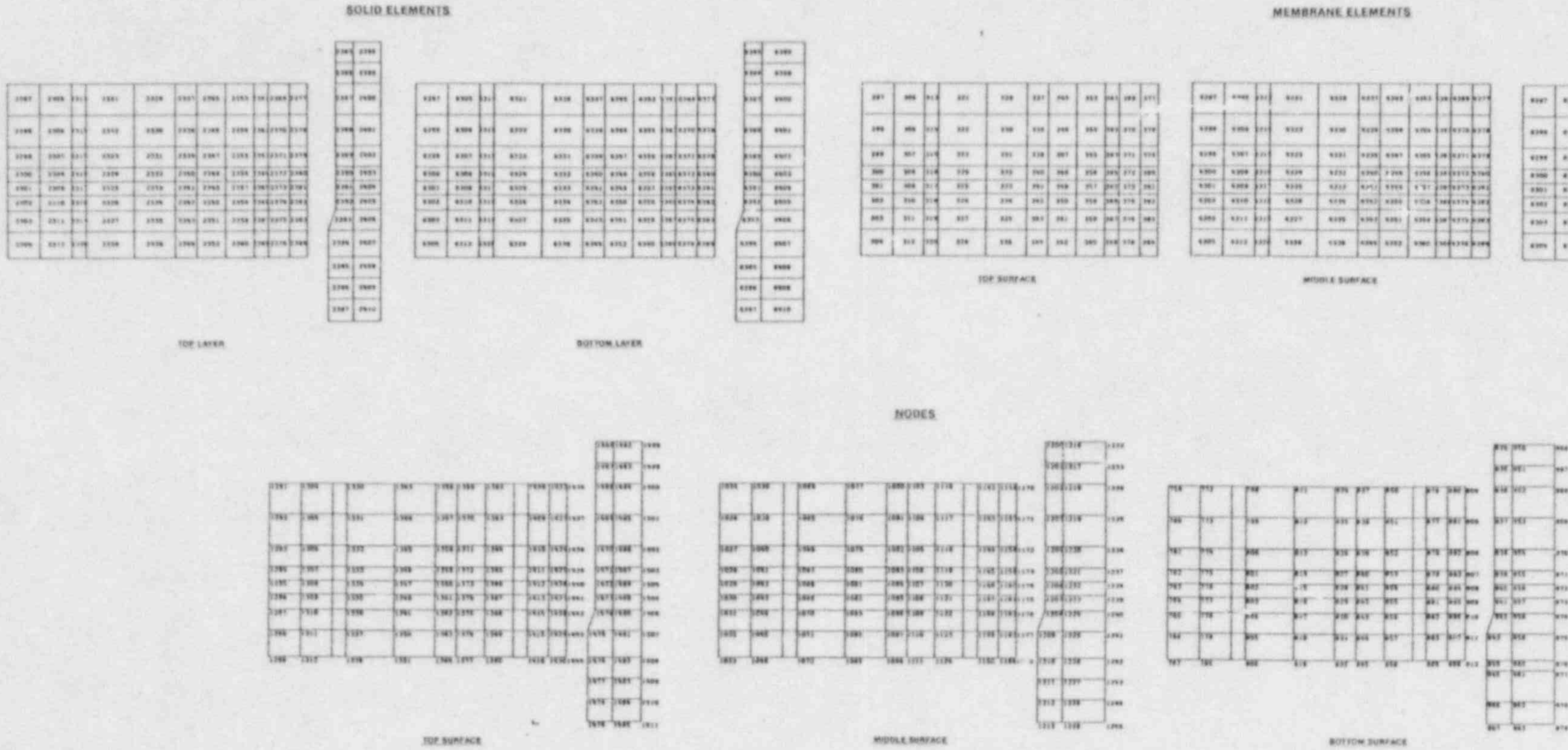


FIGURE 10

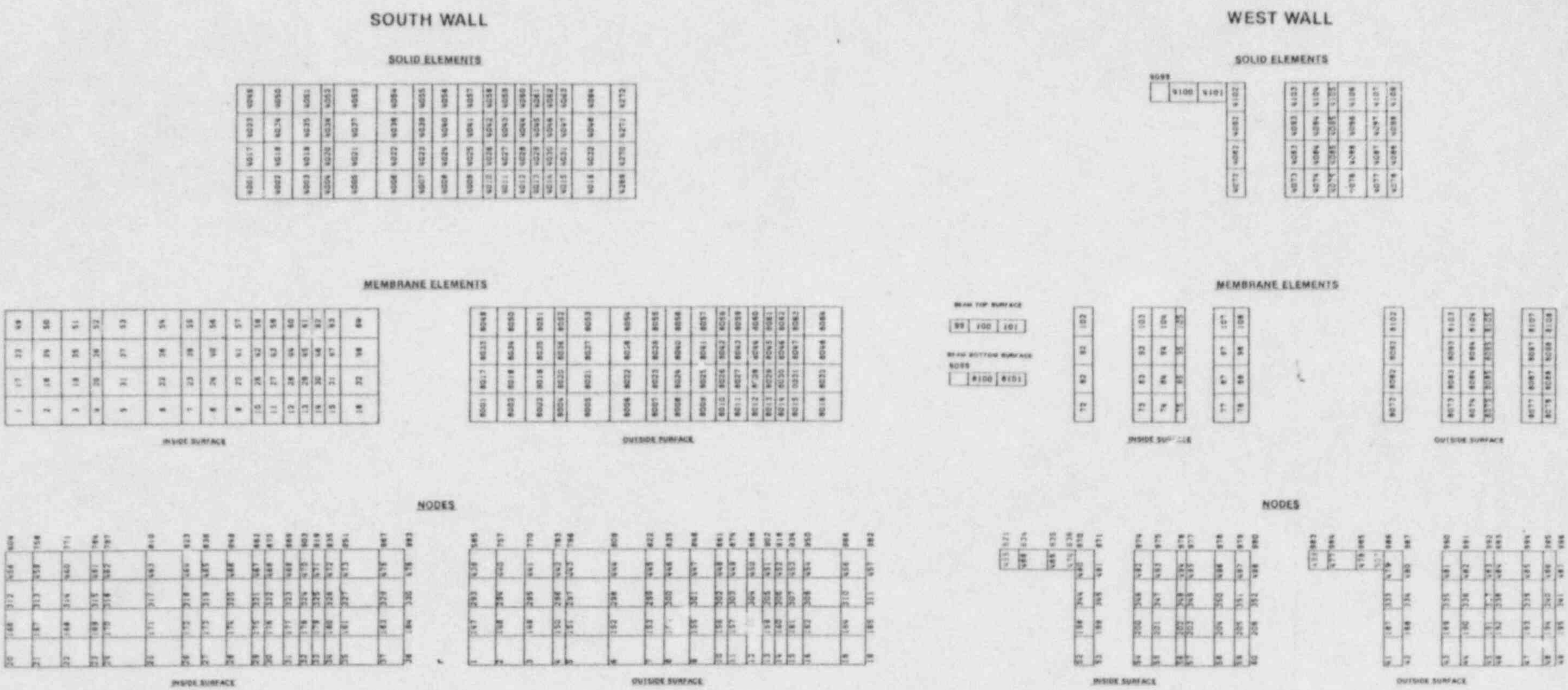
POOL FLOOR SLAB AND FUEL TRANSFER CANAL FLOOR SLAB

NUS-01-015
MARCH 25, 1983
ENCLOSURE

Structural Dynamics Technology, Inc

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL



FICURE 11

FOUNDATION SOUTH WALL AND WEST WALL

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

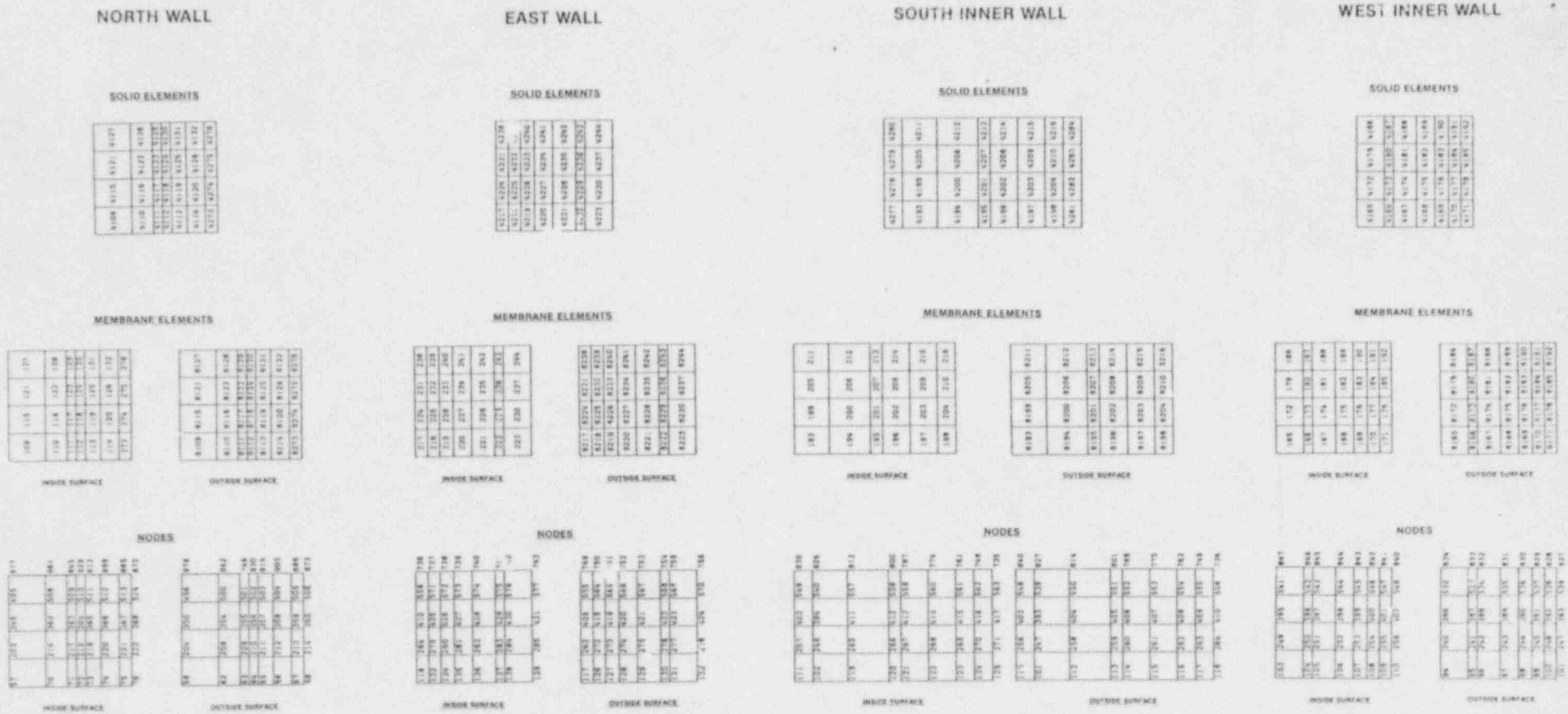


FIGURE 12

FOUNDATION NORTH WALL, EAST WALL AND INNER WALLS

NUS 01-015
MARCH 25, 1983
ENCLOSURE

Structural Dynamics Technology, Inc

MILLSTONE POINT - UNIT 2

SPENT FUEL STORAGE FACILITY FINITE ELEMENT MODEL

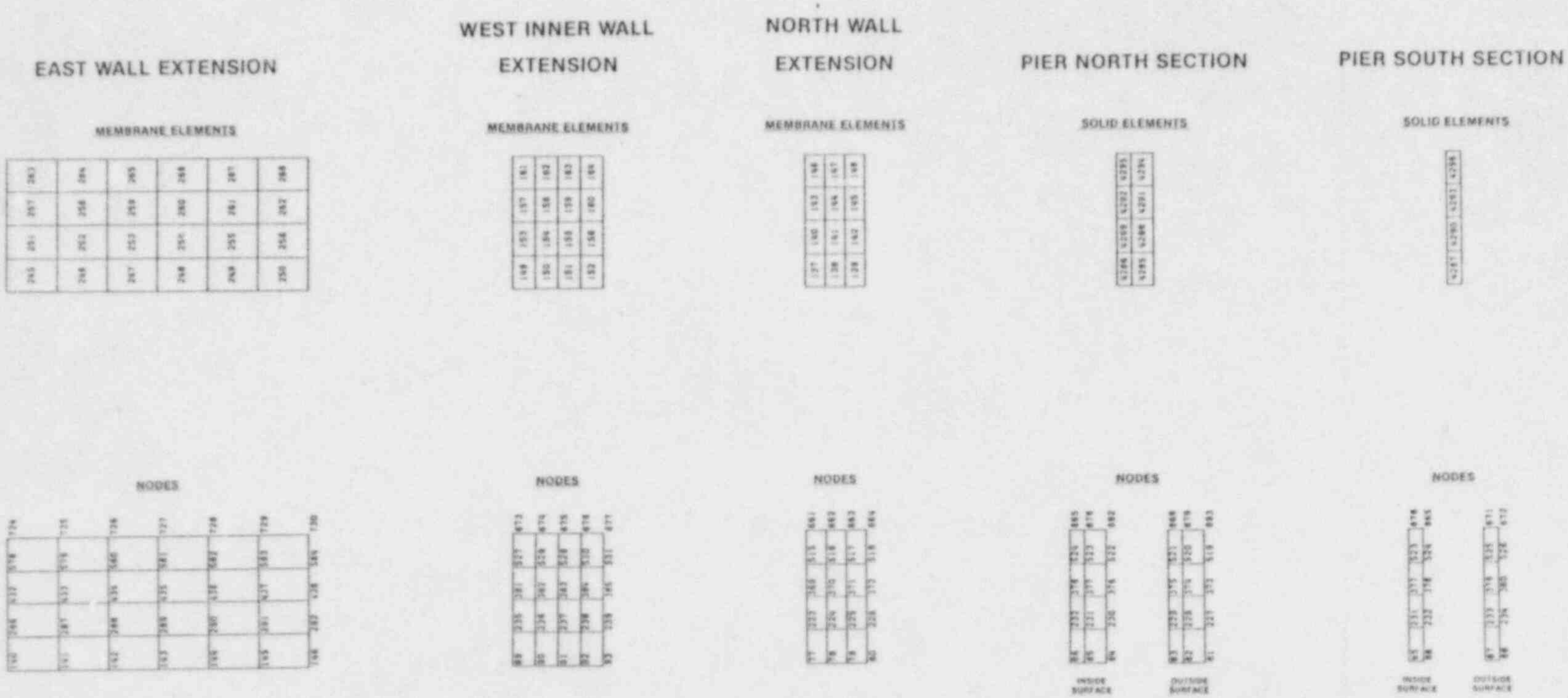


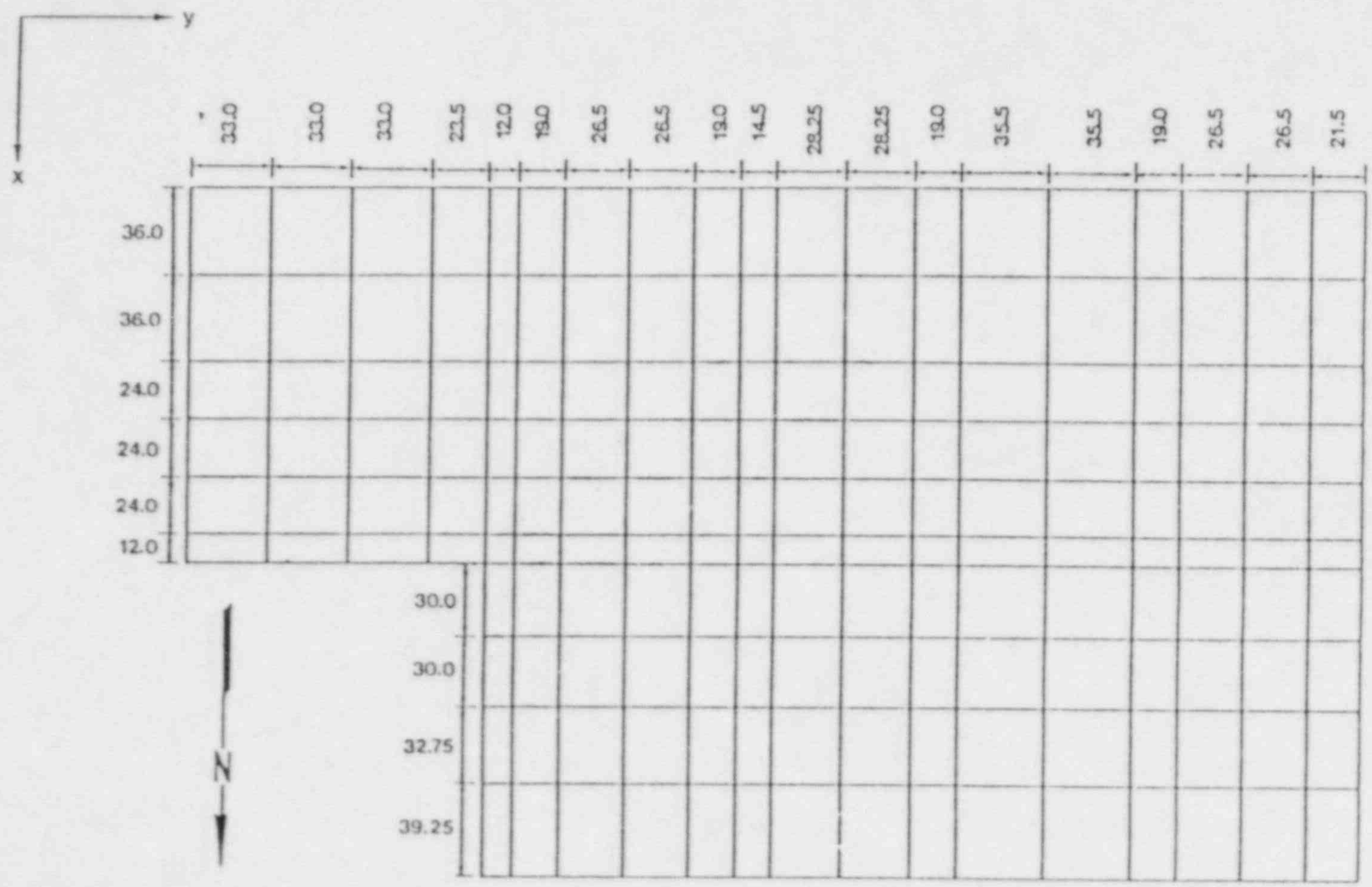
FIGURE 13

FOUNDATION PIER AND MISCELLANEOUS WALLS

NUS-01-015
MARCH 25, 1983
ENCLOSURE

SDT Structural Dynamics Technology, Inc

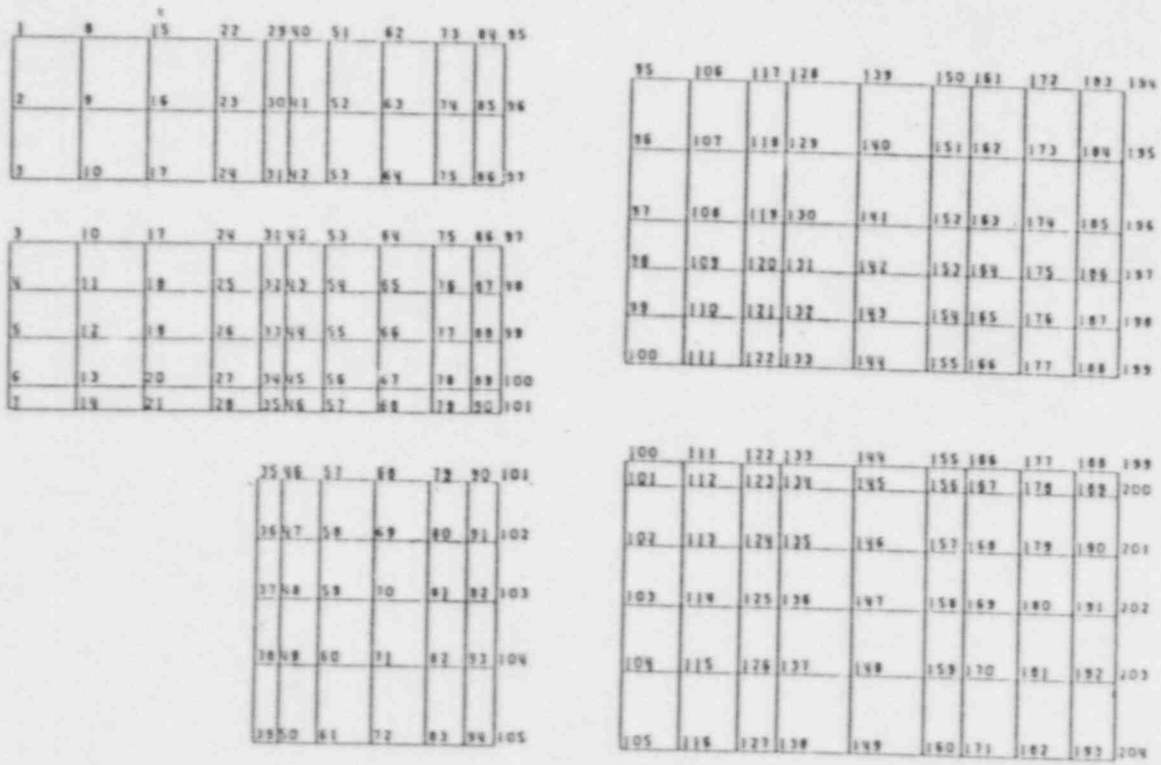
Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Floor Liner Plate Geometry



Notes:
1. Dimensions are in inches.
2. Global and local coordinate systems are coincident with the SAP6 model pool floor slab surface membranes.

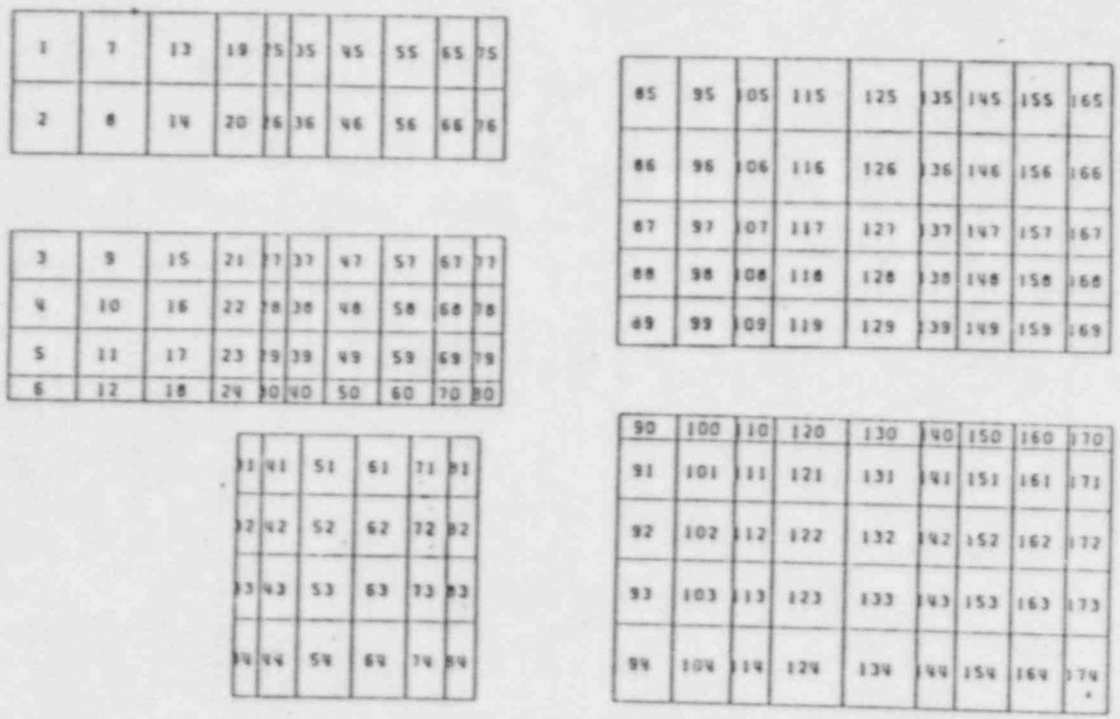
SDT Structural Dynamics Technology

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Floor Liner Plate Model Node Numbers



Structural Dynamics Technology

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Floor Liner Plate Model Element Numbers



Structural
Dynamics
Technology

QUESTION

11.b Provide a description of the mathematical model employed, including assumptions and limitations of the model.

Response

This section includes a detailed description of the finite element model used in the spent fuel pool storage facility structural evaluation along with justification of modeling assumptions which were considered important in predicting the response of the structure.

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 the auxiliary building. All walls directly adjacent 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.

The results of the finite element model were examined to insure that realistic deflections and stresses existed for each individual load case. Classical solutions were also prepared for selected components for comparison to the finite element model results. Gross force and moment reactions were calculated and resulting stresses were compared to those in the computer model. The general behavior of the model under the loads was determined to be reasonable by viewing deformed geometry plots and screening stresses at key locations.

The material properties used in the mathematical model were obtained from design criteria specifications or by NUSCO Engineering.

Concrete Material Properties

Concrete Compressive Strength	3,000	lb/in^2
Reinforcing Yield Strength	60,000	lb/in^2
Reinforcing Elastic Modulus	29.0×10^6	lb/in^2
Concrete Elastic Modulus	3.15×10^6	lb/in^2
Concrete Poisson Ratio	0.17	
Concrete Thermal Expansion Coefficient	5.5×10^{-6}	in/in/°F
Concrete Weight Density	8.68×10^{-2} ($150\ lb/ft^3$)	lb/in^3

Liner Plate Material and Anchor Properties

Plate Material	304 Stainless Steel
Plate Thickness	0.25 inches
Plate Thickness Tolerance	16%
Poissons Ratio	0.24
Coefficient of Thermal Expansion	8.82×10^{-6} in/in°F
Yield Strength	30 ksi
Weld Electrode	E308-16
Electrode Tensile Strength	90 ksi

QUESTION

11.c Describe and list the load cases used as well as the justification for these load cases.

Response

This section discusses the development and application of the loads which were applied to the finite element model. To provide flexibility for formulation of the load combinations, a static analysis was performed for the loads described in this section with the appropriate factors and permutations applied to these loads for formulation of the SRP load combinations. It should be stated that the loads applied to the mathematical model of the spent fuel pool and liner were derived based on a 2:1 consolidated fuel load. The conservatisms of this are described later in this section.

Structural Individual Load Cases

The twelve individual loads applied to the finite element model are described in Table 3.2-2. Loads which were excluded from this evaluation include fuel cask drop, crane load, rack impact and accident flood load. Fuel cask drop has been previously addressed and therefore is not considered in this analysis. The loads from the fuel handling crane were excluded since the effect on the overall pool structure was considered beneficial when considered in combination with other loads. This assumption is based upon the fact that the relatively small compressive vertical load exerted on the pool walls, due to the crane weight, aids the concrete section's ability to carry shear forces as well as other axial and moment loadings. Impacting of the rack pads due to tipping was considered a local effect and was addressed as a separate item. Accident flood load has also been eliminated from consideration since the flood gates protect the auxiliary building to the maximum probable flood height.

Dead weight of the pool structure was defined as a 1.0g vertical acceleration. Hydrostatic loading of the structure was analyzed for a pool water depth of 38'-6". The hydrostatic forces are applied to the wetted surface of the pool by computing nodal forces in the three directions as the product of the pressure at the nodal elevations by an area vector (A_x, A_y, A_z) which is computed from adjacent element areas. Membrane elements (only for the purpose of load application) were used to represent the gates in the fuel transfer canal and cask laydown areas so that the hydrostatic forces on the gates were accounted for. A resultant force was computed for this load verifying application of the load and additionally, confirming correct orientation of the elements since the nodal area vectors are based on the local coordinate systems of the membrane elements.

Individual 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.

Load cases 5 and 6 are operating and accident thermal loads, corresponding to pool water temperatures of 150°F and 212°F, respectively. The ambient (or stress free) temperature for all compartments outside the pool (including the cask laydown and fuel transfer canal areas) was defined as 55°F. These loads were applied by defining nodal temperatures for all nodes in the model based on linear interpolation of temperatures between adjacent compartments. The accident pool temperature of 212°F is justified since the pool water free surface is at atmospheric pressure. The pool bulk temperature will also be fairly uniform as a result of convection currents caused by heating of the water at lower elevations resulting in the movement of this lower density water toward the top of the pool.

Building seismic effects and the associated hydrodynamic forces due to lateral earthquake loads are included in load cases 7 and 8. The horizontal earthquake acceleration applied for these loads was calculated by taking the average of the floor zero period accelerations, determined from the auxiliary building seismic analysis for the various levels over the pool height, and applying this acceleration to the structural mass of the model. All g levels used in this analysis were taken from the "Seismic Analysis-Auxiliary Building," Millstone Nuclear Power Station, Unit No. 2, Bechtel Power Corporation, Job No. 7604-01, Revision 3, July 31, 1972.

Using the peak acceleration value from the various floor elevations over the pool height, the average peak horizontal acceleration value was found to be 0.21 g's for the 0.09 g (OBE) building base excitation. To facilitate load combinations, this seismic acceleration was expressed in terms of a nominal 1.0 g building base excitation to give a nominal 2.34 g peak acceleration at the spent fuel pool elevation. This nominal 1.0 g base excitation and resulting 2.34 g fuel pool acceleration is indicated in Table 3.2-2 for individual load cases 7 and 8.

Earthquake response of the pool water was based on the methodology outlined in TID-7024, "Nuclear Reactors and Earthquakes," which provided a basis for computing pool wall and floor pressures which result from earthquake-induced pool fluid motion. Hydrodynamic forces were calculated as the product of the pressure profiles over the wetted surfaces of the pool and their associated area vectors, similar to the application of the hydrostatic forces described previously. Gross hydrodynamic forces and moments were computed from these nodal forces, with verification by comparison to forces and moments calculated from formulas in TID-7024. These hydrodynamic responses were also normalized to a 1.0 g earthquake to facilitate load combinations.

Vertical earthquake loads were not included as individual load cases, since acceleration of the pool water mass and concrete mass are equivalent to applying appropriate load factors to their respective static load cases to account for dynamic amplification of the seismic motion.

Table 3.2-3 summarizes the load definition parameters used in evaluating the concrete structure.

Composite Load Cases

The twelve individual loads just described were combined to formulate the composite load cases applicable to this evaluation. The composite loads are shown in Table 3.2-4 and include dead load (D), live load (L), operating and accident thermal (T_o and T_a), and SSE and OBE earthquake (E and E'). Table 3.2-4 also defines the relationship between individual loads and composite loads. The Standard Review Plan load combinations which are described later in this section are formulated from these composite load cases.

Dead Loads

Dead load includes dead weight of the concrete structure, hydrostatic pressure and weight of the fuel rack modules excluding their fuel complements. The fuel module dead weight was 365 pounds per cell. Since the individual load cases for rack loads were based on nominal 1,000 psf vertical loads over Regions 1 and 2 of the pool floor slab, individual load cases 3 and 4 are factored by 0.374 and 0.607.

Live Loads

Live load consisted entirely of the submerged weight of the consolidated fuel and storage box. The weight of these two items is 2,500 pounds per cell. Based on this value, the floor slab vertical loads were computed as 2,561 pounds per square foot over Region 1 and 4,155 pounds per square foot over Region 2.

These values are based on all cells in the pool having 2:1 consolidated fuel placed in them. The actual live load for reracking in Region 1 will be 1,528 pounds per square foot or 40 percent less than analyzed for. Similarly, actual live load in Region 2 is 1,332 pounds per square foot or 68 percent less than analyzed for.

Thermal Loads

Operating and accident thermal composite loads were taken directly as their individual load cases with factors of 1.0.

Earthquake Loads

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 in 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.

As required by the Standard Review Plan, the three directions (X, Y, Z) of earthquake were applied such that all permutations of the signs were considered. Table 3.2-4 shows four of the OBE composite loads. Four additional cases not shown in Table 3.2-4 were developed by multiplying those shown in the table (E1 through E4) by -1.0. Similarly, SSE loads were formulated by multiplying the eight OBE cases by 1.8.

The service and factored load combinations were formulated according to Section 3.8.4, paragraph 3.6 of the Standard Review Plan (Reference 7). Table 3.2-5 presents the eight service load combinations and five factored load combinations from the Standard Review Plan. Eight of the SRP composite load components were not applicable to this structure and were not considered in the evaluation. These composite load components include R_o (normal operating pipe reactions), W (design wind), W_t (design tornado), R_a (pipe break reactions), P_a (accident pressure) and Y_r, Y_j, Y_m (impact and impulse from pipe break and impact). Excluding these loads, the final loads considered reduce to those shown in Table 3.2-6.

Examination of Table 3.2-6 shows load cases i.b.1 and i.b.3 to be identical, as are i.b.4 and i.b.6. Since live load is always present, the response of the structure to i.b.7 is bounded by i.b.2. Similarly, load case i.b.1 bounds i.b.8. This results in four service load combinations considered, two of which contain OBE, which has eight sub-load cases, resulting in a total of eighteen service load combinations.

The response of the structure to T_o is similar to T_a, with T_a controlling. Therefore, load case ii.b was eliminated in lieu of ii.c. For the same reason, load cases ii.a and ii.e are bounded by ii.a. This leaves two factored cases, one containing SSE, which has eight subcases, resulting in a total of nine factored load combinations.

Table 3.2-7 summarizes the coefficients applied to the composite loads for formulation of the service and factored loads previously described. Since the effect of the dead and live portions of a load combination are reduced during earthquake motion in the negative global direction, the factors on these composite loads are reduced by 10 percent. The final loads were formulated for all areas of the pool which were considered in this evaluation. Analysis was then performed for each particular concrete wall or floor for the two or three controlling load combinations.

Liner Plate Load Combination Formulation

The individual and composite load cases used for evaluation of the liner plate are identical to those presented in Tables 3.2-2 and 3.2-4, respectively, with one exception. During the liner plate evaluation, SSE horizontal rack reaction loads specified by the fuel rack vendor were reduced from 3,500 pounds per cell to 2,500 pounds per cell. This resulted in a corresponding reduction in the coefficients for individual load cases 9 through 12. The liner plate composite load cases are shown in Table 3.2-8.

The service and factored loads specified by the Standard Review Plan for plastic design methods are shown in Table 3.2-9. The same eight components for composite loads that were not considered for the liner plate analysis: including R_o (pipe break reactions), P_a (accident pressure), and Y_r, Y_j, Y_m (impact and impulse from pipe break and missile impact). Excluding these loads, the loads considered were reduced to those shown in Table 3.2-10.

From Table 3.2-10, it is evident that load cases i.b.1 and i.b.3 are identical, as are i.b.4 and i.b.6. Application of OBE in all possible locations resulted in load combination i.b.1 being bounded by i.b.2. The number of service load combinations considered was reduced to three, two of which contained OBE, which has eight subcases, resulting in seventeen possible service load combinations.

The response of structure to T_o was bounded by T_a, which resulted in elimination of ii.b.2 in lieu of ii.b.3. Similarly, load case ii.b.1 was bounded by ii.b.5. Structural response due to SSE (which is OBE factored by 1.8) results in elimination of ii.b.4 in lieu of ii.b.5. A load case of (D + L + E') was considered separately to address the effects of earthquake without thermal loads. Three factored load combinations remain, two containing SSE which (considering earthquake permutations) results in a total of 17 factored load combinations.

The final composite load case coefficients are summarized in Table 3.2.11, for the service and factored load cases previously described. Applied displacements and strains due to cracking and curvature effects were applied for the load combinations described. Concentrated loads representing the rack pad forces were not applied directly to the liner plate model at the individual load case level. It can be shown that the coefficient of friction between the rack pads and liner plate (steel-to-steel interface) is less than that between the liner plate and concrete slab. Consequently, the racks will slide before the load will be taken by the liner plate. If the rack pads stick (corresponding to a coefficient of friction of 1.0), the force provided by the cell's vertical reaction and the concrete liner plate friction is greater than the cell's horizontal reaction. In either case, the load is transmitted directly to the concrete slab which was qualified for the design loads.

Table 3.2-2

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
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)

Table 3.2-3

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
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 & Vertical	1.8 (OBE)

Structural Dynamics Technology

Table 3.2-4

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
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	.374	.607								
2	L	- Live Load			2.56	4.16								
3	T_o	- Operating Thermal					1.00							
4	T_a	- 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 3.2-5

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
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	$.75\,(1.4D + 1.7L + 1.7T_o + 1.7\,R_o)$
i.b.5	$.75\,(1.4D + 1.7L + 1.9E + 1.7T_o + 1.7R_o)$
i.b.6	$.75\,(1.4D + 1.7L + 1.7W + 1.7T_o + 1.7\,R_o)$
i.b.7	$1.2D + 1.9E$ or $.9\,(1.4D) + 1.9E$
i.b.8	$1.2D + 1.7W$ or $.9\,(1.4D) + 1.7W$
	FACTORED LOAD COMBINATIONS
ii.a	$D + L + T_o + E'$
ii.b	$D + L + T_o + R_o + W_t$
ii.c	$D + L + T_a + R_a + 1.5\,P_a$
ii.d	$D + L + T_a + R_a + 1.25\,P_a + 1.0\,Y_r + Y_j + Y_m) + 1.25\,E'$
ii.e	$D + L + T_a + R_a + 1.0\,P_a + 1.0\,(Y_r + Y_j + Y_m) + 1.0\,E'$

Structural Dynamics Technology

Table 3.2-6

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
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	.75 (1.4D + 1.7L + 1.7T_o)	
i.b.5	.75 (1.4D + 1.7L + 1.9E + 1.7T_o)	
i.b.6	.75 (1.4D + 1.7L + 1.7T_o)	(Identical to i.b.4)
i.b.7	1.2D + 1.9E or .9 (1.4D) + 1.9E	(Bounded by i.b.2)
i.b.8	1.2D or .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)

Table 3.2-7

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Final Load Combination Coefficients

Composite Load Cases LOAD COMBINATION IDENTIFIER	D	L	T_o	T_a	E_1	E_2	E_3	E_4
i.b.1	1.40	1.70						
i.b.2.1	1.40	1.70			1.90			
i.b.2.2	1.40	1.70				1.90		
i.b.2.3	1.40	1.70					1.90	
i.b.2.4	1.40	1.70						-1.80
i.b.2.5	1.26	1.53			-1.90			
i.b.2.6	1.26	1.53				-1.90		
i.b.2.7	1.26	1.53					-1.90	
i.b.2.8	1.26	1.53						1.90
i.b.4	1.05	1.28	1.28					
i.b.5.1	1.05	1.28	1.28		1.43			
i.b.5.2	1.05	1.28	1.28			1.43		
i.b.5.3	1.05	1.28	1.28				1.43	
i.b.5.4	1.05	1.28	1.28					1.43
i.b.5.5	0.95	1.15	1.28		-1.43			
i.b.5.6	0.95	1.15	1.28			-1.43		
i.b.5.7	0.95	1.15	1.28				-1.43	
i.b.5.8	0.95	1.15	1.28					-1.43
ii.c	1.00	1.00	1.00					
ii.d.1	1.00	1.00		1.00	2.25			
ii.d.2	1.00	1.00		1.00		2.25		
ii.d.3	1.00	1.00		1.00			2.25	
ii.d.4	1.00	1.00		1.00			2.25	
ii.d.5	0.90	0.90		1.00	-2.25			
ii.d.6	0.90	0.90		1.00		-2.25		
ii.d.7	0.90	0.90		1.00			-2.25	
ii.d.8	0.90	0.90		1.00				2.25

SDT Structural Dynamics Technology

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Composite Load Case Description

Individual Load Case Number:			1	2	3	4	5	6	7	8	9	10	11	12
Liner Plate														
Composite Load Case														
1	D	- Dead Load	1.00	1.00	.306	.544								
2	L	- Live Load			2.49	4.42								
3	T_o	- Operating Thermal					1.00							
4	T_a	- Accident Thermal						1.00						
5	E_1	- OBE	0.06	0.06	0.51	0.91			0.09	0.09	0.55	0.97	0.55	0.97
6	E_2	- OBE	0.06	0.06	0.51	0.91			-0.09	0.09	-0.55	-0.97	0.55	0.97
7	E_3	- OBE	0.06	-0.06	0.51	0.91			-0.09	-0.09	-0.55	-0.97	-0.55	-0.97
8	E_4	- OBE	0.06	0.06	0.51	0.91			0.09	-0.09	0.55	0.97	-0.55	-0.97

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.

SDT Structural Dynamics Technology

Table 3.2-9

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Liner Plate Standard Review Plan Load Combination Summary

Load Combination Number	Description
	SERVICE LOAD COMBINATIONS - LINER PLATE
i.b.1	$1.7D + 1.7L$
i.b.2	$1.7D + 1.7L + 1.7E$
i.b.3	$1.7D + 1.7L + 1.7W$
i.b.4	$1.3 (D + L + T_o + R_o)$
i.b.5	$1.3 (D + L + E + T_o + R_o)$
i.b.6	$1.3 (D + L + W + T_o + R_o)$
	FACTORED LOAD COMBINATIONS - LINER PLATE
ii.b.1	$D + L + T_o + R_o + E'$
ii.b.2	$D + L + T_o + R_o + W_t$
ii.b.3	$D + L + T_a + R_a + 1.5 P_a$
ii.b.4	$D + L + T_a + R_a + 1.25 P_a + 1.0 (Y_r + Y_j + Y_m) + 1.25 E$
ii.b.5	$D + L + T_a + R_a + 1.0 P_a + 1.0 (Y_r + Y_j + Y_m) + E'$

Structural Dynamics Technology

Table 3.2-10

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Applicable Liner Plate Standard Review Plan Load Combinations

Load Combination Number	Description	
	SERVICE LOAD COMBINATIONS - LINER PLATE	
i.b.1	1.7D + 1.7L	(Bounded by i.b.2)
i.b.2	1.7D + 1.7 L + 1.7E	
i.b.3	1.7D + 1.7L	(Identical to i.b.1)
i.b.4	$1.3 (D + L + T_o)$	
i.b.5	$1.3 (D + L + E + T_o)$	
i.b.6	$1.3 (D + L + T_o)$	(Identical to i.b.4)
	FACTORED LOAD COMBINATIONS - LINER PLATE	
ii.b.1	$D + L + T_o + E'$	(Bounded by ii.b.5)
ii.b.2	$D + L + T_o$	(Bounded by ii.b.3)
ii.b.3	$D + L + T_a$	
ii.b.4	$D + L + T_a + 1.25E$	(Bounded by ii.b.5)
ii.b.5	$D + L + T_a + E'$	

Table 3.2-11

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Final Load Combination Coefficients

Service Composite Load Cases - Liner Plate LOAD COMBINATION IDENTIFIER	D	L	T_o	T_a	E_1	E_2	E_3	E_4
i.b.2.1	1.70	1.70			1.70			
i.b.2.2	1.70	1.70				1.70		
i.b.2.3	1.70	1.70					1.70	
i.b.2.4	1.70	1.70						1.70
i.b.2.5	1.53	1.53			-1.70			
i.b.2.6	1.53	1.53				-1.70		
i.b.2.7	1.53	1.53					-1.70	
i.b.2.8	1.53	1.53						-1.70
i.b.4	1.30	1.30	1.30					
i.b.5.1	1.30	1.30	1.30		1.30			
i.b.5.2	1.30	1.30	1.30			1.[illegible]		
i.b.5.3	1.30	1.30	1.30				[illegible]	
i.b.5.4	1.30	1.30	1.30					[illegible]
i.b.5.5	1.17	1.17	1.30		-1.30			
i.b.5.6	1.17	1.17	1.30			-1.30		
i.b.5.7	1.17	1.17	1.30				[illegible]	
i.b.5.8	1.17	1.17	1.30					[illegible]

Structural Dynamics Technology

Table 3.2-11 (continued)

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Final Load Combination Coefficients

Factored Composite Load Cases - Liner Plate	D	L	T_o	T_a	E_1	E_2	E_3	E_4
LOAD COMBINATION IDENTIFIER								
ii.b.1.1	1.00	1.00			1.80			
ii.b.1.2	1.00	1.00				1.80		
ii.b.1.3	1.00	1.00					1.80	
ii.b.1.4	1.00	1.00						1.80
ii.b.1.5	0.90	0.90			-1.80			
ii.b.1.6	0.90	0.90				-1.80		
ii.b.1.7	0.90	0.90					-1.80	
ii.b.1.8	0.90	0.90						-1.80
ii.b.3	1.00	1.00		1.00				
ii.b.5.1	1.00	1.00		1.00	1.80			
ii.b.5.2	1.00	1.00				1.80		
ii.b.5.3	1.00	1.00					1.80	
ii.b.5.4	1.00	1.00						1.80
ii.b.5.5	0.90	0.90		1.00	-1.80			
ii.b.5.6	0.90	0.90		1.00		-1.80		
ii.b.5.7	0.90	0.90		1.00			-1.80	
ii.b.5.8	0.90	0.90		1.00				-1.80

Structural Dynamics Technology

11d. Describe how the dynamic interaction between the pool structure and the rack modules was considered, including the value of any associated dynamic amplification factors. Include all assumptions made regarding the summation and phase of all rack loads.

The dynamic interaction between the pool structure and the rack modules was accounted for by considering the mass of fully loaded rack modules in the dynamic analysis model of the auxiliary building. Motions of the spent fuel pool from a time-history analysis of the auxiliary building were then used as input for a nonlinear seismic time-history analysis of the spent fuel rack modules. The nonlinear time-history analysis of the rack modules produced seismic loads which are transmitted to the pool floor. These seismic loads consisted of horizontal shear loads and vertical loads including impacting of the rack module on the pool floor.

The total horizontal loads on the pool floor are obtained by combining the loads due to the North-South & East-West earthquake directions in accordance with Reg. Guide 1.92. The total vertical loads are obtained by combining the vertical seismic load and the tipping impact load in accordance with Reg. Guide 1.92 and adding the deadweight load. The evaluation of the local loading under the rack feet and the total pool load should be provided by Northeast Utilities. As far as phasing of racks, the nonlinear seismic analysis of the racks assumes all the rack modules move in phase. CE recommends that loads be applied to the pool floor in accordance with this assumption.

QUESTION

11.e Provide analysis of the adequance of the pool floor and liner under the local maximum rack module dynamic mounting foot loads.

Response

An analysis was performed which investigated the local effects on the pool floor slab due to rack module impact loads. The analysis considered two adjacent rack mounting feet impacting the slab simultaneously. The concrete being impacted was considered to be fully cracked. Therefore, only the residual reinforcing bar strength was accounted for. The controlling load combination for this analysis was 1.7 (D + L + E). It was determined that the residual shear strength for the section is 3,565 kips. The required residual shear strength capacity is 239.4 kips.

The analysis therefore shows that the structural integrity of the pool floor is maintained when subjected to the local maximum rack module dynamic mounting foot loads.

QUESTION

11.f Provide identification of the most critical regions of the pool structure. List the stresses and their comparison to allowable values, where the source and justification of their use of that allowable is also documented.

Response

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 3.1-1, a stress allowable criteria is used in evaluating the anchors for nonthermal loads versus a displacement criteria for thermal load combinations.

The following tables identifies the critical spent fuel pool and liner stresses and their comparison to allowable values based upon the previously described criteria. As described previously, these stresses are based on fully consolidated fuel loads.

By review of these tables, it can be shown that all stresses/ strains remain within the stated code allowables.

Table 3.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Liner Plate Criteria Summary

Liner Plate Allowables[1]

Membrane Strains

sc = .005 in/in
st = .003 in/in

Membrane Plus Bending Strains

sc = 0.014 in/in
st = 0.010 in/in

Liner Anchor Allowables[2]

Load Combinations Without Thermal

Non-Factored Load Combinations $Fa = 0.5 F_u$
Factored Load Combinations $Fa = 0.85 F_u$

Load Combinations with Thermal

a = 0.5 u

Fu and u are based on an ultimate displacement of 0.2 inches.

Notes:

1) These allowables are consistent with those specified by ASME Section II, Subsection CC for containment liner plate when ultimate strength is the basis, i.e., factored load combinations.

2) These allowables are consistent with AISC, Specification for Steel Structures, Part 2; ASME Section III Subsection CC for containment liner anchors and formulas from References 13 and 14.

Table 4.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Controlling Section Resultant Moments

Location	Controlling Load Case	Section Axial Force	Section(2) Resultant Moment	Section(3) Allowable Moment	Section Code Ratio
Pool North Wall					
Horizontal Section Lower Portion of Wall - East End Elements 444-445-446-447 (MFPSTAIAI-05B)	$(D+L+T_a+1.25E3')$	6.686	76.97	388.2	0.20
Vertical Section Lower Portion of Wall Mid-Span Element 437 (MFPSTAIAI-05)	$(D'+L'+T_a-1.25E3')$	-22.11	710.9	1325.0	0.54
Horizontal Section Upper Portion of Wall - East End Elements 477-478-479-480 (MFPSTAIAI-05B)	$(D+L+T_a+1.25E3')$	1.794	44.35	545.8	0.08
Vertical Section Upper Portion, Mid-Span Elements 482-493-504-515 (MFPSTAIAI-05A)	$(D+L+T_a+1.25E3')$	10.42	272.5	598.6	0.46
Pool South Wall					
Horizontal Section Lower Portion, West End of Pool Element 685 (MFPSTAIAI-06)	$(D'+L'+T_a-1.25E4')$	-30.32	810.1	1367.0	0.59

Units: Forces are in kips/in.
Moments are in kip in/in.

Notes:
1) Positive moment causes tension on outside surface of walls and lower surface of floor slab.
2) T_a moments are relieved, maintaining equilibrium and curvature of section.
3) Allowable moment is based on strength design method per ACI 349/80.

SDT Structural Dynamics Technology

Table 4.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Controlling Section Resultant Moments
(Continued)

Location	Controlling Load Case	Section Axial Force	Section(2) Resultant Moment	Section(3) Allowable Moment	Section Code Ratio
Pool South Wall (Continued)					
Vertical Section Lower Portion, Mid-Span Element 668 (MFPSTAIAI-06)	$(D'+L'+T_a-1.25E4')$	-33.12	813.1	1516.0	0.54
Horizontal Section Upper Portion, West End of Pool Element 707 (MFPSTAIAI-06)	$(D'+L'-T_a-1.25E4')$	-23.27	685.6	1142.0	0.60
Vertical Section Upper Portion, Mid-Span Elements 712-723-734-745 (MFPSTAIAI-06A)	$(D+L+T_a+1.25E4')$	11.99	177.3	545.7	0.32
Pool East Wall					
Horizontal Section Bottom of Wall Elements 577-578-579-580-581-582-583-584 (MFPSTAIAI-07B)	$(D'+L'+T_a-1.25E2')$	7.807	109.3	339.1	0.32
Vertical Section Lower Portion of Wall - South End Element 578 (MFPSTAIAI-07)	$(D'+L'+T_a-1.25E3')$	-18.52	669.0	1332.0	0.50

Units: Forces are in kips/in.
Moments are in kip in/in.

Notes:
1) Positive moment causes tension on outside surface of walls and lower surface of floor slab.
2) T_a moments are relieved, maintaining equilibrium and curvature of section.
3) Allowable moment is based on strength design method per ACI 349/80.

Table 4.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Controlling Section Resultant Moments
(Continued)

Location	Controlling Load Case	Section Axial Force	Section[2] Resultant Moment	Section[3] Allowable Moment	Section Code Ratio
Pool East Wall (Continued)					
Horizontal Section Upper Portion of Wall Elements 609-610-611-612-613-614-615-616 (MFPSTAIAI-07B)	$(D'+L'+T_a-1.25E3')$	-0.821	133.0	612.8	0.22
Vertical Section Top of Wall - South End Elements 609-617-625-633 (MFPSTAIAI-07A)	$(D'+L'+T_a-1.25E3')$	7.527	19.77	695.6	0.03
Fuel Transfer Canal Separation Wall					
South (4 ft.) Portion of Wall (MFPSTAIAI-08)					
Horizontal Section Mid-Span (Element 844)	$(D'+L'+T_a-1.25E3')$	15.30	58.27	60.56	0.96
Vertical Section South End of Wall Lower Portion (Element 829)	$(D'+L'+T_a-1.25E4')$	-15.82	366.4	749.0	0.49
Horizontal Section South End of Wall Lower Portion (Element 829)	$(D'+L'T_a-1.25E4')$	-18.12	345.6	640.0	0.54

Units: Forces are in kips/in.
Moments are in kip in/in.

Notes:
1) Positive moment causes tension on outside surface of walls and lower surface of floor slab.
2) T_a moments are relieved, maintaining equilibrium and curvature of section.
3) Allowable moment is based on strength design method per ACI 349/80.

Structural Dynamics Technology

Table 4.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Controlling Section Resultant Moments
(Continued)

Location	Controlling Load Case	Section Axial Force	Section(2) Resultant Moment	Section(3) Allowable Moment	Section Code Ratio
Fuel Transfer Canal Separation Wall (Continued)					
Vertical Section Mid-Span (Element 843)	$(D'+L'+T_a-1.25E4')$	-8.915	268.1	684.5	0.39
North (3 ft.) Portion of Wall (MFPSTAIAI-08)					
Vertical Section Below Elevation of Bottom of Gate Opening (Element 823)	$(D'+L'+T_a-1.25E4')$	-23.64	363.6	581.5	0.63
Horizontal Section Below Elevation of Bottom of Gate Opening (Element 823)	$(D'+L'+T_a-1.25E3')$	-14.47	304.6	591.9	0.51
Vertical Section Above Elevation of Bottom of Gate Opening (Element 839)	$(D+L+T_a+1.25E4')$	-11.11	196.1	473.9	0.41
Horizontal Section Above Elevation of Bottom of Gate Opening (Element 839)	$(D'+L'+T_a-1.25E4')$	-7.476	192.5	332.1	0.58

Units: Forces are in kips/in.
Moments are in kip in/in.

Notes:
1) Positive moment causes tension on outside surface of walls and lower surface of floor slab.
2) T_a moments are relieved, maintaining equilibrium and curvature of section.
3) Allowable moment is based on strength design method per ACI 349/80.

Table 4.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Controlling Section Resultant Moments
(Continued)

Location	Controlling Load Case	Section Axial Force	Section Resultant Moment (2)	Section Allowable Moment (3)	Section Code Ratio
Cask Laydown Area West Separation Wall (MFPSTAIAI-10)					
Vertical Section Below Elevation of Bottom of Gate (Element 874)	$(D'+L'+T_a-1.25E2')$	-10.26	-134.7	-232.3	0.58
Horizontal Section at Bottom of Wall (Element 872)	$(D'+L'+T_a-1.25E1')$	-7.759	-91.34	-184.4	0.50
Vertical Section Above Elevation of Bottom of Gate Opening (Element 880)	$(D'+L'+T_a-1.25E2')$	-5.537	-84.16	-351.2	0.24
Horizontal Section Above Elevation of Bottom of Gate Opening (Element 880)	$(D'+L'+T_a-1.25E2')$	-10.92	-91.31	-203.6	0.45

Units: Forces are in kips/in.
Moments are in kip in/in.

Notes:
1) Positive moment causes tension on outside surface of walls and lower surface of floor slab.
2) T_a moments are relieved, maintaining equilibrium and curvature of section.
3) Allowable moment is based on strength design method per ACI 349/80.

Structural Dynamics Technology

Table 4.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Controlling Section Resultant Moments
(Continued)

Location	Controlling Load Case	Section Axial Force	Section(2) Resultant Moment	Section(3) Allowable Moment	Section Code Ratio
Cask Laydown Area South Separation Wall (MFPSTAIAI-10)					
Vertical Section Below Elevation of Bottom of Gate Opening (Element 906)	$(D'+L'+T_a-1.25E2')$	-5.087	-104.0	-203.6	0.51
Horizontal Section at Bottom of Wall (Element 903)	$(D'+L'+T_a-1.25E2')$	-7.573	-88.94	-183.2	0.49
Vertical Section Above Elevation of Bottom of Gate Opening (Element 910)	$(D'+L'+T_a-1.25E2')$	1.031	-118.2	-355.4	0.33
Horizontal Section Above Elevation of Bottom of Gate Opening (Element 910)	$(D'+L'+T_a-1.25E1')$	-9.703	-85.72	-196.6	0.44
Pool Floor Slab (MFPSTAIAI-09)					
North-South Section at South End of Pool Mid-Span (Element 338)	$(D+L+T_a+1.25E4')$	-0.417	537.5	759.8	0.71

Units: Forces are in kips/in.
Moments are in kip in/in.

Notes:
1) Positive moment causes tension on outside surface of walls and lower surface of floor slab.
2) T_a moments are relieved, maintaining equilibrium and curvature of section.
3) Allowable moment is based on strength design method per ACI 349/80.

SDT Structural Dynamics Technology

Table 4.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Controlling Section Resultant Moments
(Continued)

Location	Controlling Load Case	Section Axial Force	Section(2) Resultant Moment	Section(3) Allowable Moment	Section Code Ratio
Pool Floor Slab (Continued)					
East-West Section at South End of Pool Mid-Span (Element 346)	$(D+L+T_a+1.25E4')$	-25.36	644.0	1121.	0.57
North-South Section in Cask Laydown Area Elements 302-303-304 (MFPSTAIAI-09B)	$(D+L+T_a+1.25E1')$	17.01	-33.76	-259.3	0.13
East-West Section in Cask Laydown Area Elements 303-311-319-327 (MFPSTAIAI-09A)	$(D+L+T_a+1.25E1')$	3.843	129.0	646.6	0.20
Foundation West Wall Beam					
Horizontal Section at South End of Beam Element 99 (MFPSTAIAI-17)	$(D+L+T_a+1.25E3')$	-1.283	-39.59	-237.6	0.17

Units: Forces are in kips/in.
Moments are in kip in/in.

Notes:
1) Positive moment causes tension on outside surface of walls and lower surface of floor slab.
2) T_a moments are relieved, maintaining equilibrium and curvature of section.
3) Allowable moment is based on strength design method per ACI 349/80.

Table 4.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Controlling Section Resultant Moments
(Continued)

Location	Controlling Load Case	Section Axial Force	Section[2] Resultant Moment	Section[3] Allowable Moment	Section Code Ratio
Foundation West Wall Column					
Horizontal Section at Top of Column Element 102 (MFPSTAIAI-18)	$(D+L+T_a+1.25E4')$	-28.12	277.0	865.3	0.32
South Foundation Wall					
Vertical Section East Portion East End of Wall at Bottom Elements 1-2-3-4-5 (MFPSTAIAI-11B-1)	$(D'+L'+T_a-1.25E2')$	-3.954	-102.5	-312.6	0.33
Vertical Section West Portion West End of Wall at Bottom Elements 10-11-12-13-14-15-16 (MFPSTAIAI-11B)	$(D'+L'+T_a-1.25E4')$	10.22	54.0	54.92	0.98
Inner West Foundation Wall					
Vertical Section at Bottom Elements 165-166-167-168-169-170-171 (MFPSTAIAI-15B)	$(D'+L'+T_a-1.25E2')$	-0.994	58.02	289.7	0.20

Units: Forces are in kips/in.
Moments are in kip in/in.

Notes:
1) Positive moment causes tension on outside surface of walls and lower surface of floor slab.
2) T_a moments are relieved, maintaining equilibrium and curvature of section.
3) Allowable moment is based on strength design method per ACI 349/80.

Table 4.1-1

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaulation
Tabulation of Controlling Section Resultant Moments
(Continued)

Location	Controlling Load Case	Section Axial Force	Section[2] Resultant Moment	Section[3] Allowable Moment	Section Code Ratio
Inner South Foundation Wall					
Vertical Section at Bottom Elements 193-194-195-196 (MFPSTAIAI-13B)	$(D'+L'+T_a-1.25E4')$	1.553	-89.81	-128.8	0.70

Units: Forces are in kips/in.
Moments are in kip in/in.

Notes:
1) Positive moment causes tension on outside surface of walls and lower surface of floor slab.
2) T_a moments are relieved, maintaining equilibrium and curvature of section.
3) Allowable moment is based on strength design method per ACI 349/80.

Structural Dynamics Technology

Table 4.1-2

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant Transverse Shear Forces

Location	Controlling Load Case	Section(2) Shear	Allowable(3) Section Shear	Code Shear Ratio
Pool North Wall				
Vertical Section at West End of Wall Elements 443-454-465-476-487-498-509	$(D+L+T_a+1.25E3')$	3.062	6.377	0.48
Vertical Section at West End of Wall at Top Element 520	$(D+L+T_a+1.25E3')$	8.881	27.77	0.32
Vertical Section at Intersection with Cask Laydown Area West Wall at Top Element 512	$(D'+L'+T_a-1.25E4')$	14.50	28.93	0.50
Vertical Section at Intersection with Cask Laydown Area West Wall Elements 435-446-457-468-479-501	$(D'+L'+T_a-1.25E4')$	11.27	31.21	0.36
Horizontal Section at Bottom of Wall Elements 433-434-435-436-437-438-439-440-441-442-443	$(D+L+T_a+1.25E3')$	1.805	6.167	0.29

Units: Kips/inch

Notes:
1) Data from MFPSTAIAI-04
2) Shear forces are linearly interpolated to the distance from the face of the effective support equal to the distance from the section compressive face to the centroid of the tensile steel where applicable.
3) Allowable shear is based on strength design per ACI 349/80.

Table 4.1-2

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant Transverse Shear Forces
(Continued)

Location	Controlling Load Case	Section(2) Shear	Allowable(3) Section Shear	Code Shear Ratio
Pool South Wall				
Vertical Section at West End at Top of Wall Element 740	$(D+L+T_a+1.25E4')$	10.18	25.89	0.39
Vertical Section at West End of Wall Elements 663-674-685-696-707-718-729	$(D+L+T_a+1.25E4')$	1.087	6.234	0.17
Horizontal Section at Top of Wall Elements 740-741-742-743-744-745-746-747-748-749-750	$(D'+L'+T_a-1.25E4')$	5.397	7.827	0.69
Pool East Wall				
Vertical Section at South End of Wall at Top Element 633	$(D+L+T_a+1.25E3')$	3.876	25.88	0.15
Vertical Section at South End of Wall Elements 577-585-593-601-609-617-625	$(D+L+T_a+1.25E3')$	3.018	6.362	0.47

Units: Kips/inch

Notes:
1) Data from MFPSTAIAI-04
2) Shear forces are linearly interpolated to the distance from the face of the effective support equal to the distance from the section compressive face to the centroid of the tensile steel where applicable.
3) Allowable shear is based on strength design per ACI 349/80.

Structural Dynamics Technology

Table 4.1-2

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant Transverse Shear Forces
(Continued)

Location	Controlling Load Case	Section(2) Shear	Allowable(3) Section Shear	Code Shear Ratio
Pool East Wall (Continued)				
Vertical Section at Intersection with Cask Laydown Area South Wall at Top Element 637	$(D+L+T_a+1.25E2')$	8.720	26.26	0.33
Vertical Section at Intersection with Cask Laydown Area South Wall Elements 581-589-597-605-613-621-629	$(D+L+T_a+1.25E2')$	14.55	31.18	0.47
Horizontal Section at Top of Wall Elements 625-626-627-628-629-630-631-632	$(D'+L'+T_a+1.25E2')$	5.573	5.922	0.94
Fuel Transfer Canal Separation Wall				
Vertical Section at South End of Wall (4 ft. portion) at Top Element 870	$(D+L+T_a+1.25E3')$	11.73	19.05	0.62

Units: Kips/inch

Notes:
1) Data from MFPSTAIAI-04
2) Shear forces are linearly interpolated to the distance from the face of the effective support equal to the distance from the section compressive face to the centroid of the tensile steel where applicable.
3) Allowable shear is based on strength design per ACI 349/80.

Structural Dynamics Technology

Table 4.1-2

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant Transverse Shear Forces
(Continued)

Location	Controlling Load Case	Section[(2)] Shear	Allowable[(3)] Section Shear	Code Shear Ratio
Fuel Transfer Canal Separation Wall (Continued)				
Vertical Section at South End of Wall (4 ft. portion) Elements 814-822-830-838-846-854-862	$(D+L+T_a+1.25E3')$	1.849	4.837	0.38
Horizontal Section at Mid Height of South (4 ft.) Portion Elements 833-834-835-836-837-838	$(D+L+T_a+1.25E3')$	4.130	4.346	0.95
Vertical Section Below Gate Opening North (3 ft.) Portion Elements 808-816-824	$(D+L+T_a+1.25E3')$	0.718	3.307	0.22
Horizontal Section at Bottom of Wall Elements 807-808-809-810-811-812-813-814	$(D+L+T_a+1.25E3')$	2.910	4.041	0.72

Units: Kips/inch

Notes:
1) Data from MFPSTAIAI-04
2) Shear forces are linearly interpolated to the distance from the face of the effective support equal to the distance from the section compressive face to the centroid of the tensile steel where applicable.
3) Allowable shear is based on strength design per ACI 349/80.

Structural Dynamics Technology

Table 4.1-2

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant Transverse Shear Forces
(Continued)

Location	Controlling Load Case	Section[(2)] Shear	Allowable[(3)] Section Shear	Code Shear Ratio
Cask Laydown Area South Separation Wall				
Vertical Section at Intersection with Pool East Wall Elements 903-905-907-909-911-913-915-917	$(D+L+T_a+1.25E4')$	2.533	3.325	0.76
Horizontal Section at Bottom of Wall Elements 903-904	$(D'+L'+T_a-1.25E3')$	1.594[(4)]	2.084	0.76
Cask Laydown Area West Separation Wall				
Vertical Section at Intersection with Cask Laydown Area South Wall Elements 873-876-879-882-885-888-891-894	$(D'+L'+T_a-1.25E2')$	1.887	4.079	0.46
Horizontal Section at Bottom of Wall Elements 871-872-873	$(D'+L'+T_a-1.25E1')$	1.691	1.943	0.87

Units: Kips/inch

Notes:
1) Data from MFPSTAIAI-04
2) Shear forces are linearly interpolated to the distance from the face of the effective support equal to the distance from the section compressive face to the centroid of the tensile steel where applicable.
3) Allowable shear is based on strength design per ACI 349/80.
4) Transverse shear adjusted based upon cracked section equilibrium moment gradient.

Structural Dynamics Technology

Table 4.1-2

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant Transverse Shear Forces
(Continued)

Location	Controlling Load Case	Section[(2)] Shear	Allowable[(3)] Section Shear	Code Shear Ratio
Pool Floor Slab				
East-West Section at Mid-Span Elements 301-309-317-325-333-341-349 357-365-373-381	$(D'+L'+T_a-1.25E1')$	4.323	5.622	0.77
North-South Section Beneath Cask Laydown Area West Separation Wall Elements 313-314-315-316-317-318-319-320	$(D+L+T_a+1.25E1')$	11.23	13.07	0.86
North-South Section at Mid-Span Elements 321-322-323-324-325-326-327-328	$(D+L+T_a+1.25E1')$	2.996	8.491	0.35
Foundation South Wall				
West Portion Horizontal Section at Top Elements 58-59-60-61-62-63-64	$(D'+L'+T_a-1.25E1')$	2.141	7.581	0.28
East Portion Horizontal Section at Top Elements 49-50-51-52-53-54-55-56-57	$(D+L+T_a+1.25E1')$	2.446	7.064	0.35

Units: Kips/inch

Notes:
1) Data from MFPSTA1A1-04
2) Shear forces are linearly interpolated to the distance from the face of the effective support equal to the distance from the section compressive face to the centroid of the tensile steel where applicable.
3) Allowable shear is based on strength design per ACI 349/80.

Structural Dynamics Technology

Table 4.1-2

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant Transverse Shear Forces
(Continued)

Location	Controlling Load Case	Section(2) Shear	Allowable(3) Section Shear	Code Shear Ratio
Foundation East Wall				
Horizontal Section at Top Elements 238-239-240-241-242-243-244	$(D+L+T_a+1.25E1')$	2.976	6.949	0.43
Foundation Inner South Wall				
Horizontal Section at Bottom Elements 193-194-195-196-197-198	$(D'+L'+T_a-1.25E4')$	1.848	3.316	0.56
Foundation Inner West Wall				
Horizontal Section at Bottom Elements 165-166-167-168-169-170-171	$(D'+L'+T_a-1.25E3')$	1.848	2.920	0.63
Foundation North Wall				
Horizontal Section at Bottom Elements 109-110-111-112-113-114	$(D+L+T_a+1.25E2')$	5.803	10.46	0.55
Foundation West Wall				
North Portion Horizontal Section at Bottom Elements 77-78	$(D+L+T_a+1.25E4')$	3.001	11.79	0.25

Units: Kips/inch

Notes:
1) Data from MFPSTAIAI-04
2) Shear forces are linearly interpolated to the distance from the face of the effective support equal to the distance from the section compressive face to the centroid of the tensile steel where applicable.
3) Allowable shear is based on strength design per ACI 349/80.

Table 4.1-2

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant Transverse Shear Forces
(Continued)

Location	Controlling Load Case	Section(2) Shear	Allowable(3) Section Shear	Code Shear Ratio
Foundation West Wall				
South Portion Horizontal Section at Bottom Elements 83-84-85	$(D+L+T_a+1.25E3')$	6.140	12.91	0.48

Notes: 1) Data from MFPSTAIAI-04
2) Shear forces are linearly interpolated to the distance from the face of the effective support equal to the distance from the section compressive face to the centroid of the tensile steel where applicable.
3) Allowable shear is based on strength design per ACI 349/80.

Table 4.1-3

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant In-Plane Shear Forces

Location	Controlling Load Case	Section[1] Shear	Allowable Section Shear	Code Shear Ratio
Pool North Wall				
Horizontal Section at Top of Wall Elements 510-511-512-513-514-515-516-517-518-519-520	$(D'+L'+T_a-1.25E3')$	0.774	25.4	0.03
Pool South Wall				
Horizontal Section at Bottom of Wall Elements 663-664-665-666-667-668-669-670-671-672-673	$(D+L+T_a+1.25E3')$	3.032	25.4	0.12
Pool East Wall				
Horizontal Section at Bottom of Wall Elements 577-578-579-580-581-582-583-584	$(D+L+T_a+1.25E2')$	9.206	26.58	0.35
Fuel Transfer Canal Separation Wall				
South (4 ft.) Portion Horizontal Section at Bottom of Wall Elements 817-818-819-820-821-822	$(D+L+T_a+1.25E3')$	8.670	24.79	0.35

Units: Kips/inch

Notes: 1) Allowable shear is based on strength design per ACI 349/80.

Structural Dynamics Technology

Table 4.1-3

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Tabulation of Resultant In-Plane Shear Forces
(Continued)

Location	Controlling Load Case	Section[1] Shear	Allowable Section Shear	Code Shear Ratio
Fuel Transfer Canal Separation Wall (Continued)				
Horizontal Section at Bottom of North (3 ft.) Portion Elements 807-808	$(D+L+T_a+1.25E3')$	14.29	23.90	0.60
Cask Laydown Area South Separation Wall				
Horizontal Section in Upper Portion of Wall Elements 913-914	$(D+L+T_a+1.25E2')$	5.566	30.35	0.18
Cask Laydown Area West Separation Wall				
Horizontal Section at Bottom of Wall Elements 871-872-873	$(D+L+T_a-1.25E3')$	6.770	12.80	0.53
Pool Floor Slab				
North-South Section Near East End of Pool Elements 313-314-315-316-317-318-319-320	$(D+L+T_a+1.25E1')$	14.14	24.87	0.57

Units: Kips/inch

Notes: 1) Allowable shear is based on strength design per ACI 349/80.

Structural Dynamics Technology

Table 4.1-4

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Pool Floor Liner Plate Analysis Summary

Controlling Non-Thermal Load Combination 1.7 (D + L + E2) i.b.2.2

	Element	Strain (in/in)x10^{-3} s	Allowable Strain (in/in)x10^{-3} a	Ratio s/a
Membrane Strains				
Tensile	31	0.201	3.0	0.07
Compressive	45	-0.051	-5.0	0.01
Membrane plus Bending Strains				
Tensile	84	0.444	10.0	0.04

	Node(s)	Weld Stress s	Allowable Stress(ksi) a	Ratio s/a
Weld Stress	105	2.69	20.4	0.13

Data from MFPSTA2A1-12

Controlling Thermal Load Combination (D + L + Ta + E2') ii.b.5.2

	Element	Strain (in/in)x10^{-3} s	Allowable Strain (in/in)x10^{-3} a	Ratio s/a
Membrane Strains				
Compressive	6	-0.639	-5.0	0.13
Membrane plus Bending Strains				
Compressive	6	-2.83	-14.0	0.20

	Node(s)	Weld Stress (ksi) s	Allowable Stress (ksi) a	Ratio s/a
Weld Stress	195-198 by 1	20.2	20.4	0.99

Data from MFPSTA2A1-12

Structural Dynamics Technology

Table 4.1-4

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Pool Floor Liner Plate Analysis Summary
(Continued)

Controlling Non-Thermal Load Combination 1.7 (D + L + E2) i.b.2.2

Node (Ancnor Location)	Displacement (inches)	Allowable Displacement (inches)	(Ratio)
204	0.074	0.10	0.74

Data from MFPSTA2A1-09

Controlling Thermal Load Combination (D + L + Ta + E2') i.b.5.2

Node (Ancnor Location)	Displacement (incnes)	Allowable Displacement (inches)	(Ratio)
22	0.013	0.10	0.10

Seam Embedded Angle

Node-DOF	Shear Stress-F_s (ksi)	Allowable Stress - F_{sa} (ksi)	F_s/F_{sa} (Ratio)
68	5.192	16.5	0.31

Date from MFPSTA2A1-10

Structural Dynamics Technology

Table 4.1-5

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Wall Liner Plate Strains
Membrane Tensile Strains

Location - Description (Analysis Identifier)		Load Combination	Nominal Strain (in/in) x 10^{-3}	Allowable Strain (in/in) x 10^{-3}	Ratio E_s/E_a
North & South Walls (MFPSTA1A2-11)	Element 518 - X Section North Wall at Top	$(D'+L'+T_o-1.25E4')$ II.B.5.8	1.118	3.0	0.37
East Wall (MFPSTA1A2-12)	Element 601 - X Section Mid-Height of Wall	1.7(D'+L'+E2) I.B.2.2	0.438	3.0	0.15
Fuel Transfer Canal Wall 3 Foot Portion (MFPSTA1A2-13)	Element 863 - X Section at Top of Wall	1.7(D+L+E4) I.B.2.4	0.820	3.0	0.27
Fuel Transfer Canal Wall 4 Foot Portion (MFPSTA1A2-13)	Element 844 - Y Section Mid-Height of Wall	$(D'+L'+T_o-1.25E4')$ II.B.5.8	0.694	3.0	0.23
Cask Laydown Area South Wall (MFPSTA1A2-14)	Element 871 - Y Section West Separation Wall at Bottom	1.7(D+L+E2) I.B.2.2	0.197	3.0	0.07

Table 4.1-5 (Continued)

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Wall Liner Plate Strains
Membrane Compressive Strains

Location - Description (Analysis Identifier)		Load Combination	Nominal Strain (in/in) $\times 10^{-3}$	Allowable Strain (in/in) $\times 10^{-3}$	Ratio E_s/E_a
North & South Walls (MFPSTA1A2-11)	Element 668 - X Section South Wall at Bottom	$(D'+L'+T_o-1.25E4')$ II.B.5.8	-0.623	-5.0	0.12
East Wall (MFPSTA1A2-12)	Element 612 - Y Section Mid-Span of Wall	$(D'+L'+T_o-1.25E3')$ II.B.5.7	-0.597	-5.0	0.12
Fuel Transfer Canal Wall 3 Foot Thick Portion (MFPSTA1A2-13)	Element 823- X Section Mid-Height of Wall	$(D'+L'+T_o-1.25E4')$ II.B.5.8	-0.949	-5.0	0.19
Fuel Transfer Canal Wall 4 Foot Thick Portion (MFPSTA1A2-13)	Element 822 - X Section South End at Bottom	$(D'+L'+T_o-1.25E4')$ II.B.5.8	-0.587	-5.0	0.12
Cask Laydown Area Walls (MFPSTA1A2-14)	Element 878 - X Section West Separation Wall Below Gate	$(D'+L'+T_o-1.25E2')$ II.B.5.6	-0.911	-5.0	0.18

Table 4.1-5 (Continued)

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Wall Liner Plate Strains
Membrane + Bending Tensile Strains

Location - Description (Analysis Identifier)		Load Combination	Nominal Strain (in/in) x 10^{-3}	Membrane Bending Strain (in/in) x 10^{-3}	Allowable Strain (in/in) x 10^{-3}	Ratio E_s/E_a
North and South Walls (MFPSTA1A2-11)	Element 512 - X Section North Wall at Top	(D'+L'+T_o-1.25E4') II.B.5.8	1.111	4.444	10.0	0.44
East Wall (MFPSTA1A2-12)	Element 601 - X Section North Wall Adjacent CLA South Wall	(D'+L'+T_o-1.25E3') II.B.5.7	0.438	1.751	10.0	0.18
Fuel Transfer Canal Wall 3 Foot Thick Portion (MFPSTA1A2-13)	Element 863 - X Section Top of Wall	1.7(D+L+E4) I.B.2.4	0.820	3.280	10.0	0.33
Fuel Transfer Canal Wall 4 Foot Thick Portion (MFPSTA1A2-13)	Element 870 - X Section Top at South End of Wall	(D'+L'+T_o-1.25E4') II.B.5.8	0.571	2.284	10.0	0.23
Cask Laydown Area Walls (MFPSTA1A2-14)	Element 871 - Y Section West Separation Wall at Bottom	1.7(D+L+E2) I.B.2.2	0.197	0.768	10.0	0.79

Table 4.1-5 (Continued)

Northeast Utilities Service Company
Millstone Point Unit 2 Spent Fuel Pool Evaluation
Wall Liner Plate Strains
Membrane + Bending Compressive Strains

Location - Description (Analysis Identifier)		Load Combination	Nominal Strain (in/in) $\times 10^{-3}$	Membrane Bending Strain (in/in) $\times 10^{-3}$	Allowable Strain (in/in) $\times 10^{-3}$	Ratio E_s/E_a
North and South Walls (MFPSTA1A2-11)	North Wall - Element 443 Y Section, Bottom at West End of Wall	(D'+L'+T_o-1.25E4') II.B.5.8	-0.544	-2.176	-14.0	0.16
East Wall (MFPSTA1A2-13)	Element 580 - Y Section Bottom of Wall at Mid-Span	(D'+L'+T_o-1.25E3') II.B.5.7	-0.561	-2.245	-14.0	0.16
Fuel Transfer Canal Wall 3 Foot Thick Portion (MFPSTA1A2-13)	Element 823 - X Section Mid-Height of Wall	(D'+L'+T_o-1.25E4') II.B.5.8	-0.949	-3.796	-14.0	0.27
Fuel Transfer Canal Wall 4 Foot Thick Portion (MFPSTA1A2-13)	Element 822 - X Section South End at Bottom	(D'+L'+T_o-1.25E4') II.B.5.8	-0.587	-2.348	-14.0	0.17
Cask Laydown Area Walls (MFPSTA1A2-14)	Element 877 - X Section West Separation Wall Below Gate	(D'+L'+T_o-1.25E2') II.B.5.6	-0.762	-3.050	-14.0	0.22