NES 81A0522

STRUCTURAL ANALYSIS DESIGN REPORT
for the
CRYSTAL RIVER UNIT 3
HIGH DENSITY FUEL STORAGE RACKS

Prepared Under Project 5127
for the
FLORIDA POWER CORPORATION

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SUMMARY

The Crystal River Power Station Unit 3 high density fuel storage racks have been designed to meet the requirements for Seismic Category I structures. Detailed structural and seismic analyses of the high density storage racks have been performed to verify the adequacy of the design to withstand the loadings encountered during installation, normal operation, the severe and extreme environmental conditions of the operating basis and safe shutdown earthquakes and the abnormal loading conditions of an accidental fuel assembly drop event.

The response of the rack structure to specified static and dynamic loading conditions has been evaluated by means of linear elastic analysis using the finite element method. Non-linear analysis and energy balance techniques have been used to evaluate structural damage resulting from accidental fuel assembly drop. For the specified loading conditions, the maximum stresses and deflections of the rack structure have been calculated and shown to be less than the allowable values.

It has been concluded from the results of the structural and seismic analyses that the storage racks are adequately designed to withstand the loadings associated with normal operating and abnormal conditions.

2. INTRODUCTION

Nuclear Energy Services, Inc. (NES) has designed high density spent fuel storage racks as shown in the drawings listed in Reference 1, to be installed in the Crystal River Unit 3 fuel pools. The racks are designed to provide storage locations for up to 1158 fuel assemblies; 549 assemblies in Pool A and 609 assemblies in Pool B and are designed to maintain the stored fuel, having an equivalent uranium enrichment of 3.3 weight percent U-235 in UO_2, in a safe, coolable, and subcritical configuration during normal and abnormal conditions.

This report presents the results of the structural analysis performed by NES to evaluate the design adequacy of the high density spent fuel racks to withstand the loadings which could occur during installation, normal operating and abnormal conditions, including the dynamic loadings resulting from the specified Operating Basis and Safe Shutdown Earthquakes and the effect of fuel assembly impact on the fuel storage cells during earthquake events. The seismic response has been calculated by response spectrum modal superposition techniques using seismic response spectra for 1.0 percent equipment damping for the Operating Basis and Safe Shutdown Earthquakes. The combination of modes and spatial earthquake components in the seismic response analysis are based on NRC Regulatory Guide 1.92 (Reference 2). The impact effect has been considered by applying appropriate impact factors to the loads/stresses developed by the seismic analysis.

3. DESCRIPTION OF THE STORAGE RACKS

The arrangement of the storage racks in Pool A and in Pool B are shown in Figures 3.1 and 3.2 respectively. The fuel storage rack configuration in Pool A contains a variety of storage arrays including five (5) storage racks with a 6x6 array of fuel storage locations, ten (10) storage racks with a 6x5 array of fuel storage locations, one (1) storage rack with a 4x5 array of fuel storage locations, one (1) storage rack with a 4x6 array of fuel storage locations plus four (4) failed fuel storage locations and one (1) storage rack with a 4x5 array of fuel storage locations plus two (2) failed fuel storage locations. Pool B has twenty-five (25) storage racks with a 5x5 array of fuel storage locations.

Each storage rack consists of an assembly of fuel storage cells spaced 10.5 inches on center and welded to a base grid structure. Each storage cell is a double wall Type 304 stainless steel box (8.9375 inches I.D.). The double wall construction provides four (4) compartments in which poison elements can be placed. The top opening of the storage cell is flared to facilitate insertion of the fuel assembly; the bottom member of the storage cell provides the level support surface required for the fuel assembly and contains the cooling flow orifice.

The bottom member of each storage cell sits on and is welded to the rack base unit which is basically a grid structure constructed from Type 304 stainless steel wide flange and box beam members. A schematic drawing of a 6x6 rack base unit a single storage assembly is shown in Figure 3.3. Continuous space bars are provided at the middle and top of the storage cells to ensure that the required pitch (10.5 inches) is maintained between storage cells in both directions (north/south and east/west). The spacer bars which are intermittantly welded to the storage cells also maintain the vertical alignment of the cells. Support feet attached to the bottom of the rack base raise the rack above the pool floor to the height required to provide an adequately sized cooling water supply plenum (for natural circulation). Each support foot contains a remotely adjustable jackscrew to permit the rack to be leveled following wet installation.

The horizontal seismic loads are transmitted from the rack structure to the spent fuel pool walls at the rack base elevation through the bumpers between the racks and the lateral seismic bracing between the pool walls and the peripheral racks. No shear loads are transmitted to the pool floor. The vertical dead-weight and seismic loads are transmitted directly to the pool floor by the support feet. The seismic bracing is fabricated from Type 304 stainless steel and is provided with restraint clips to facilitate wet installation and subsequent adjustment.

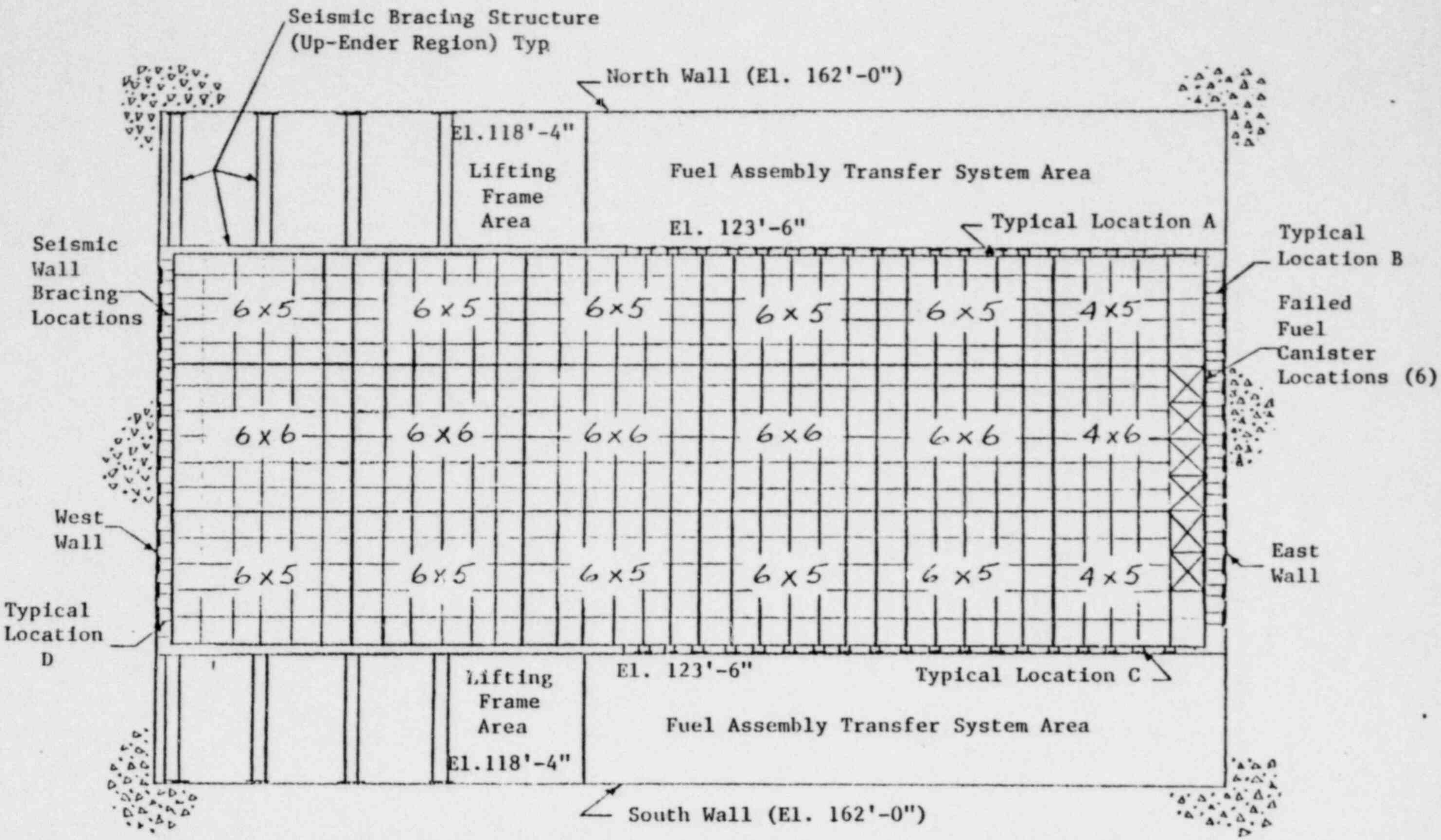


FIGURE 3.1

SPENT FUEL STORAGE RACK ARRANGEMENT PLAN
"Pool A"

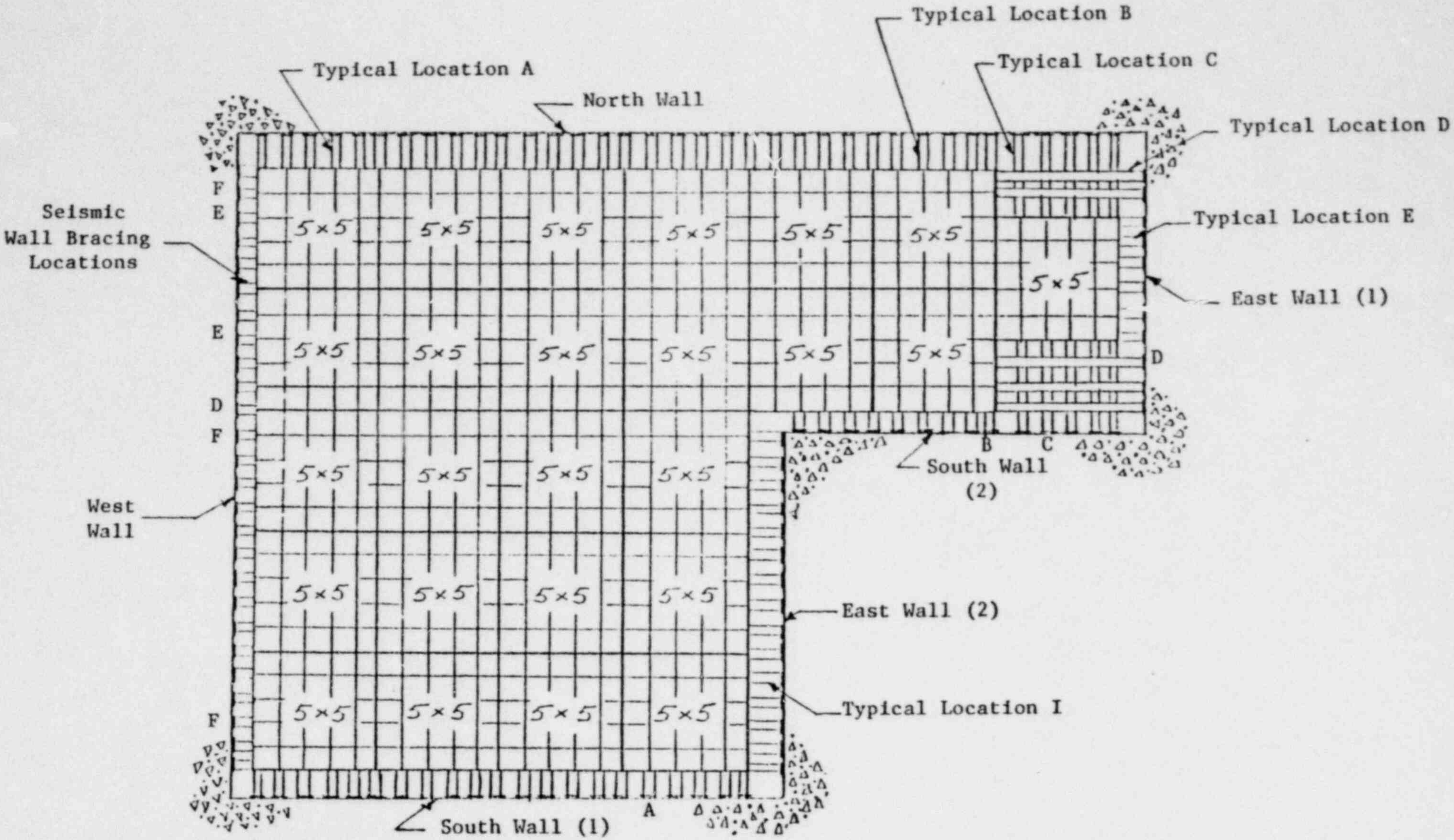


FIGURE 3.2

SPENT FUEL STORAGE RACK ARRANGEMENT PLAN
"Pool B"

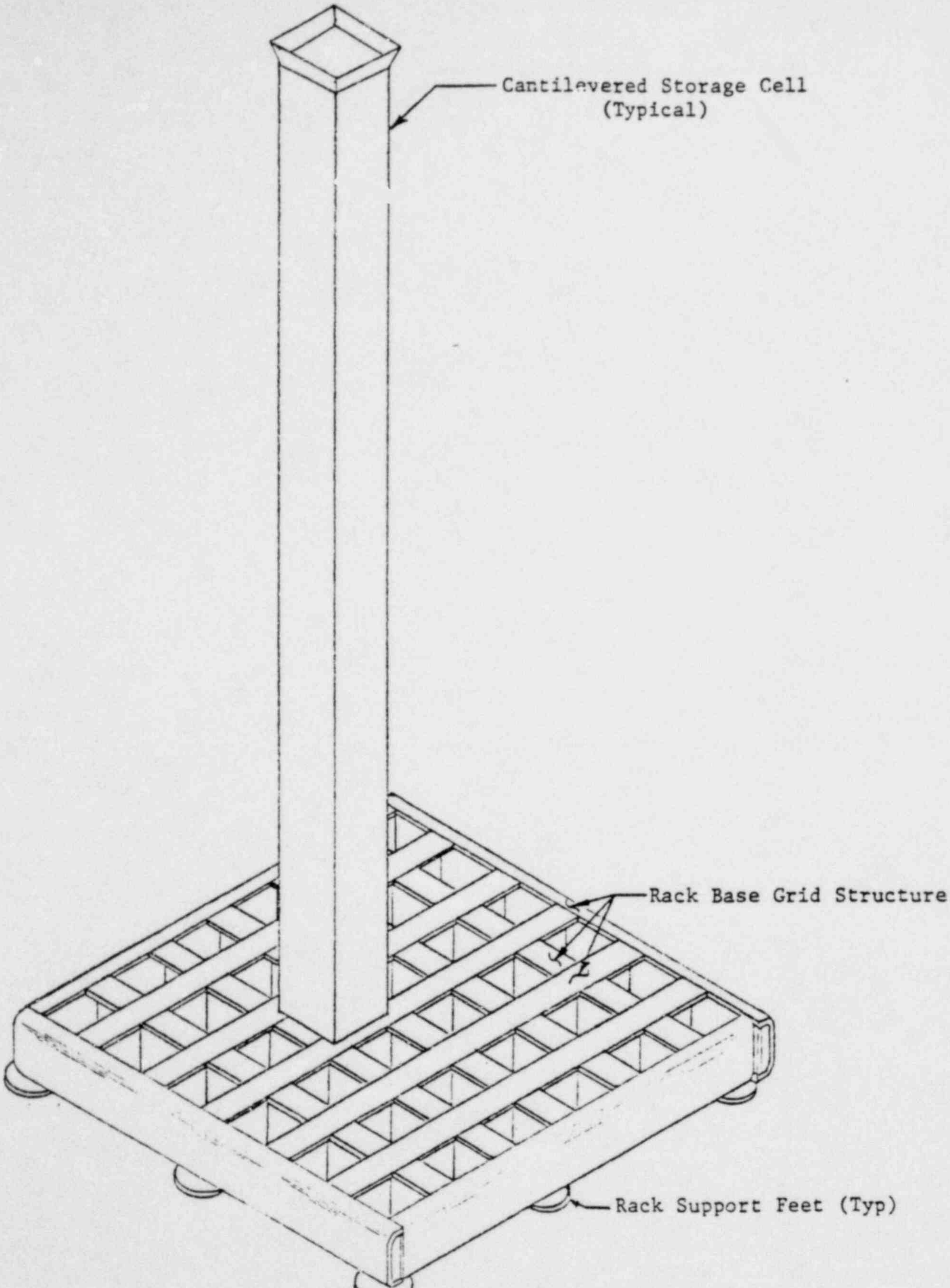


FIGURE 3.3

SPENT FUEL STORAGE RACKS SCHEMATIC (6x6)
CRYSTAL RIVER UNIT #3

4. APPLICABLE CODES, STANDARDS AND SPECIFICATIONS

The following design codes and regulatory guides have been used in the design/analysis of spent fuel storage racks.

1. A.I.S.C. Manual of Steel Construction, Seventh Edition, 1970.

2. USNRC Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants", October 1973.

3. USNRC Regulatory Guide 1.92, "Combination of Modes and Spatial Components in Seismic Response Analysis", Rev. 1, February 1976.

4. USNRC Standard Review Plan, Section 3.8.4.

5. Specification for High Density Spent Fuel Storage Racks, Crystal River - Unit No. 3, Gilbert Associates, Inc., Specification No. SP-6486, February 25, 1977.

6. "USNRC Proposed Position for Review and Acceptance of Spent Fuel Storage and Handling Application."

5. LOADING CONDITIONS

The following load cases and load combinations have been considered in the analysis in accordance with the requirements of USNRC Standard Review Plan, Section 3.8.4 (Reference 3).

5.1 Load Cases

Load Case 1 - Dead Weight of Rack, D + L (Normal Load)

Under normal operating conditions, the rack is subjected to the dead weight loading of the rack structure itself plus the loads resulting from the storage cells and fuel assemblies stored in the cells. Dead weight analysis considered two storage conditions: a fully-loaded rack (36 assemblies) and a partially-loaded rack (9 assemblies).

Load Case 2 - Dead Weight of Rack Plus 1 G. Vertical Installation Load, D + I.L. (Normal Load)

During installation the rack is subjected to the loading resulting from its own structural weight, weight of the empty storage cells plus a 1 G. vertical load resulting from a suddenly applied crane load.

Load Case 3 - Dead Weight of Rack Plus Uplifting Load, (D + U.L.) (Abnormal Load)

The possibility of the fuel handling bridge fuel hoist grapple getting hooked on a fuel storage cell was considered. The axial upward or downward force considered for this load case was 350 pounds.

Load Case 4 - Operating Basis Earthquake, E (Severe Environmental Load)

The rack, fuel assemblies, and virtual water mass react to the simultaneous loading of the horizontal and vertical components of the seismic response acceleration spectra specified for the Operating Basis Earthquake in the Crystal River Unit 3 Seismic Design Specifications (Reference 4) and presented in Table 5.1. The seismic loading is considered for two storage conditions: a fully-loaded and partially-loaded rack (36 and 9 assemblies, respectively).

Load Case 5 - Safe Shutdown Earthquake, E'
(Extreme Environmental Load)

Same as Load Case 4 except that the seismic response acceleration spectra corresponding to the Safe Shutdown Earthquake was used in the analysis (see Table 5.1).

Load Case 6 - Assembly Drop Impact Load,
(Abnormal Load)

The possibility of dropping a fuel assembly on the rack from the highest possible elevation during spent fuel handling was considered. A 1700 pound weight was postulated to drop on the rack from a height of 34 inches. Three cases were considered: 1) a direct drop on top of a single storage can, 2) a subsequent tipping of the assembly onto surrounding storage cans, and 3) a straight drop through the storage can and impact onto the rack grid structure.

Thermal Loading, T (Normal Load)

The stresses and reaction loads due to thermal loadings are insignificant since clearances are provided between racks to allow unrestrained growth of the racks for the maximum expected temperature differential based on a maximum pool temperature of 205°F (see Appendix B).

5.2 Load Combinations

(a) For service load conditions, the following load combinations are considered using elastic working stress design methods of AISC (Reference 5):

(1) D + L (1a) D + L + T

(2) D + I.L.

(3) D + L + E (3a) D + L + T + E

(b) For factored load conditions, the following load combinations are considered using elastic working stress design methods of AISC (Reference 5):

(4) D + L + T + E'

(5) D + T + U.L.

TABLE 5.1

SEISMIC RESPONSE ACCELERATION SPECTRA

Earthquake Direction	Mode Number	Frequency CPS	Seismic Acceleration OBE Event 1% Damping	Seismic Acceleration SSE Event 1% Damping
		Case 1 - Fully Loaded Rack (36 Assemblies)		
X-1 (N-S)	1	4.37	0.15	0.30
	2	11.74	0.41	0.82
	3	17.57	0.25	0.50
	4	18.68	0.21	0.41
	5	19.71	0.17	0.34
	6	20.22	0.16	0.32
	7	20.46	0.11	0.22
	8	23.48	0.09	0.18
	9	26.20	0.07	0.14
	10	28.34	0.065	0.13
	11	31.70	0.065	0.13
X-2 (E-W)	1	4.35	0.15	0.30
	2	11.73	0.41	0.82
	3	12.48	0.415	0.83
	4	13.06	0.415	0.83
	5	13.55	0.415	0.83
	6	13.78	0.41	0.82
	7	13.90	0.41	0.82
	8	19.93	0.16	0.32
	9	20.18	0.16	0.32
	10	23.46	0.095	0.19
	11	26.18	0.07	0.14
X-3 (Vertical)*	1	56.3	0.033	0.067
	2	58.8	0.033	0.067
	3	58.8	0.033	0.067

* G value equal 2/3 horizontal spectra

TABLE 5.1 (cont.)

SEISMIC RESPONSE ACCELERATION SPECTRA

Earthquake Direction	Mode Number	Frequency CPS	Seismic Acceleration OBE Event 1% Damping	SSE Event 1% Damping
		Case 2 - Partially-Loaded Rack (9 Assemblies)		
X-1 (N-S)	1	6.30	0.16	0.32
	2	16.94	0.28	0.56
	3	17.59	0.24	0.48
	4	18.6[illegible]	0.21	0.42
	5	19.72	0.16	0.32
	6	20.22	0.16	0.32
	7	20.46	0.15	0.30
	8	28.32	0.07	0.14
	9	28.86	0.07	0.14
	10	33.89	0.06	0.12
X-2 (E-W)	1	6.28	0.16	0.32
	2	12.49	0.415	0.83
	3	13.06	0.415	0.83
	4	13.55	0.415	0.83
	5	13.78	0.415	0.83
	6	13.90	0.415	0.83
	7	16.93	0.28	0.56
	8	19.93	0.16	0.32
	9	20.19	0.16	0.32
	10	33.86	0.06	0.12
X-3 (Vertical)*	1	65.26	0.0267	0.053
	2	95.33	0.0267	0.053
	3	106.02	0.0267	0.053
	4	121.85	0.0267	0.053

* G value equal 2/3 horizontal spectra

6. STRUCTURAL ACCEPTANCE CRITERIA

The following allowable limits constitute the structural acceptance criteria used for each of the loading combinations presented in Section 5.2.

Load Combinations	Limit
1, 2, 3	S
1a, 3a	1.5S
4, 5	1.6S

Where S is the required section strength based on the elastic design methods and the allowable stresses defined in Part 1 of the AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings", February 12, 1969. The yield stress value for stainless steel is taken as 30.0 ksi.

The acceptance criteria for Load Case 6, the accidental fuel assembly drop onto the rack, is that the resulting impact will not adversely affect the leak-tightness integrity of the fuel pool floor and liner plate and that the deformation of the impacted storage cells will not adversely affect the value of k_{eff}.

7. METHOD OF ANALYSIS

7.1 Rack Structural Analysis

7.1.1 Mathematical Model

A 6x6 rack module, assumed to be located at the north-west corner of the spent fuel pool B (Figure 3.2) has been analyzed in detail. This module conservatively represents the controlling structural case since it has the larger member span and it will be axially loaded by the lateral seismic loads from the maximum number of fuel assemblies located adjacent to the rack. Analyses have been performed for two conditions of fuel loading. In the first case, the analysis assumed the 6x6 module (36 fuel assemblies) and the complete pool to be fully loaded with fuel assemblies. In the second case, the analysis assumed the 6x6 module to be partially loaded (9 fuel assemblies) and the adjacent racks in the pool to be fully loaded with fuel assemblies.

In order to perform static, dynamic, and stress analyses of the fuel storage rack structure, the rack has been mathematically modeled as a three-dimensional finite-element structure consisting of discrete three-dimensional elastic beam and plate elements interconnected at a finite number of nodal points as shown in Figures 7.1.a through 7.1.c. Stiffness characteristics of the structural members are related to the cross sectional area, effective shear area and moment of inertia of the element sections. Six degrees of freedom (three translations and three rotations) are permitted at each nodal point.

Appropriate boundary conditions are assumed for both static and dynamic analysis. For the static dead weight analysis, the distributed masses of the structural elements are lumped at the system nodal points. For the rack installation load analysis, loads corresponding to the 36 empty cells are applied at the appropriate system nodal points. Similarly for Load Case 3, a vertical load of 350 pounds is applied at the worst location on the rack base structure. Applicable boundary conditions for the static load cases are shown in Figures 7.2 through 7.4 and 7.7 of this report.

For the horizontal seismic analysis, the distributed masses corresponding to the individual storage cells, fuel assemblies and contained water mass are lumped at appropriate nodal points. Additional lumped masses, located at appropriate points around the rack, account for the interaction between adjacent racks. The resulting lumped mass multi-degree-of-freedom model best represents the dynamic characteristics of the racks base structure acting in unison. The boundary conditions and lumped mass locations for the seismic analysis are shown in Figures 7.5, 7.6, 7.8, and 7.9.

7.1.2 Mathematical Formulation of the Static Analysis

The static analysis of the finite element model has been performed using the direct stiffness methods of structural analysis. If the force displacement relationship of each of the discrete structural elements is known (the element stiffness matrix) then the force-displacement relationship for the entire structure can be assembled using standard matrix methods as shown below.

For each element

$$k\ u = f \qquad (1)$$

where:

k = Element stiffness matrix
u = Element nodal displacement vector
f = Element nodal force vector

For the idealized system the equation of equilibrium may be written, in matrix form, as follows:

$$K\ U = F \qquad (2)$$

where:

K = Assembled stiffness matrix for the system

$$= \sum_{i=1}^{n} k$$

U = Nodal displacement vector for the system
F = External nodal point force vector

If sufficient boundary conditions are specified on U to guarantee a unique solution, Equation (2) can be solved for the nodal point displacements at each node in the structure, knowing the system stiffness matrix and external force matrix. From the displacement response of the system, the internal forces and stresses in each structural element can be calculated.

7.1.3 Mathematical Formulation of the Dynamic Analysis

Eigenvalue Analysis

The eigenvalues (natural frequencies) and the eigenvectors (mode shapes) for each of the natural modes of vibration are calculated by solving the following frequency equation:

$$\left[K - \omega_n^2 M\right] \left\{\phi_n\right\} = \left\{0\right\} \qquad (3)$$

where:

ω_n = Natural angular frequency for the n^{th} node

M = System mass matrix

ϕ_n = Mode shape vector for the n^{th} mode

0 = Null vector

The eigenvalue/eigenvector extraction is performed using the Householder-QR technique.

<u>Dynamic (Seismic) Load Analysis</u>

Considering only translational degrees of freedom and assuming viscous (velocity proportional) form of damping, the equation of motion in matrix form can be expressed as follows:

$$M(\ddot{U}_t + \ddot{U}_{gt}) + C\dot{U}_t + KU_t = 0 \qquad (4)$$

where:

$\ddot{U}_t$ = Relative acceleration time history vector

$\ddot{U}_{gt}$ = Ground acceleration time history vector

C = Damping matrix

$\dot{U}_t$ = Velocity time history vector

U_t = Relative displacement time history vector

Rearranging equation (4)

$$M\ddot{U}_t + C\dot{U}_t + KU_t = -M\ddot{U}_{gt} = P_{eff} \qquad (5)$$

To uncouple equation (5), assume

$$U = \Phi Y_t$$

where:

Φ = Characteristic free vibration mode shapes matrix

Y_t = Generalized coordinate displacement time history vector

Applying the above coordinate transformation and multiplying equation (5) by the transpose of Φ and using orthogonality conditions, the following uncoupled equations of motion are obtained:

$$\ddot{Y}_{nt} + 2\omega_n\lambda_n \dot{Y}_{nt} + \omega_n^2 Y_{nt} = M_n^{*-1} R_n \ddot{U}_{gt}$$

where:

Y_{nt} = Generalized displacement coordinate time history for n^{th} mode

λ_n = Damping ratio for the n^{th} mode expressed as percent of critical damping

M_n^* = Generalized mass for the n^{th} mode

$$= \Phi_n^T M\Phi_n = \sum M_i \phi_{in}^2$$

The mode shape Φ_n is normalized such that $M_n^* = 1$

R_n = Participation factor for the n^{th} mode

$$= \Phi_n^T MI = M_i\phi_{in}$$

I = Column vector whose elements are generally unity

The solution for the differential equation (6) is given by the Duhamel Integral

$$\ddot{Y}_{nt} = \frac{R_n}{M^*_n \omega_n} \int_o^t \ddot{U}_{gt}\, e^{-\lambda_n\omega_n (t-T)} \sin \omega_n (t-T)\, dT$$

Using the response spectrum method of analysis, the maximum values of the generalized response for each mode is given by:

$$\ddot{Y}_{n\,max} = \frac{R_n S_{an}}{M^*_n} \tag{7}$$

where:

$\ddot{Y}_n$ max = Maximum generalized coordinate acceleration response for the n^{th} mode.

S_{an} = Maximum spectral acceleration value for the n^{th} mode (from the applicable response spectrum curve, considering a ±10% variation in the calculated natural frequency for the n^{th} mode).

From the maximum generalized coordinate response the maximum acceleration ($\ddot{U}_{n\,max}$) and maximum inertia forces ($F_{n\,max}$) at each mass point are given by:

$$\ddot{U}_{n\,max} = \ddot{Y}_{n\,max}\, \phi_{in}$$

$$F_{n\,max} = M_n \ddot{U}_{n\,max}$$

The inertia forces ($F_{n\ max}$) for each of the system natural modes are applied as external static forces, and system response (displacements, member internal forces and stresses) are calculated using the procedure described in Section 7.1.2. Total system response is then obtained by combining the individual modal response values in accordance with Regulatory Guide 1.92 (Reference 2); lower modes having large contribution to the response are considered and higher modes with negligible participation are neglected.

The combined seismic response of the three spatial components of the earthquake has been obtained by taking the square root of the sum of the squares of the corresponding maximum response values due to the three components calculated independently (Regulatory Guide 1.92, Reference 2).

7.1.4 Stress Analysis

From the internal forces at the ends of each structural member, the stresses are calculated using the following equations:

$$f_a = \frac{Fx1}{A}$$

$$f_{bx2} = \frac{Mx2}{I_2} \times C3$$

$$f_{bx3} = \frac{Mx3}{I_2} \times C2$$

$$f_{max} = f_a + f_{bx2} + f_{bx3}$$

$$f_{min} = f_a + f_{bx2} - f_{bx3}$$

where:

f_a = Axial Stress

f_{bx2} = Bending stress due to moment about member X2 axis

f_{bx3} = Bending stress due to moment about member X3 axis

f_{max} = Maximum tensile stress due to axial load plus biaxial bending moments

f_{min} = Maximum compression stress due to axial load plus biaxial bending moments

Fx1 = Member axial force in member x1 direction

Mx1, Mx2, Mx3 = Moment about member X1, X2, X3 axes respectively

A = Member cross sectional area

I2, I3 = Moment of inertia about member X2 and X3 axes respectively

C2, C3 = Edge distance of the structural section from neutral axes in X2 and X3 direction of member coordinate system.

The structural analysis and stress analysis calculations are performed using the STARDYNE computer program (Reference 6).

7.2 Water Sloshing Effects

The sloshing effects of water on the fuel racks have been evaluated using the analytical methods given in ASCE's "Structural Analysis and Design of Nuclear Plant Facilities", Reference 7. These calculations are presented in Appendix C.

7.3 Fuel Assembly Impact Loads

The "rattling" effects of the fuel assembly inside the cell have been accounted for by using suitable impact factors as described in Appendix D.

7.4 Protection Against Overturning

A stability analysis of the rack structure under seismic loading has been performed by means of energy balance techniques using maximum rack velocities generated in the seismic analysis described in Section 7.1. This analysis is presented in Appendix E.

7.4 Accidental Fuel Assembly Drop Analysis

Linear and non-linear analysis techniques using energy balance methods as indicated in Appendix F are used to assess the structural damage resulting from a fuel assembly dropping on the rack from a height of 34 inches.

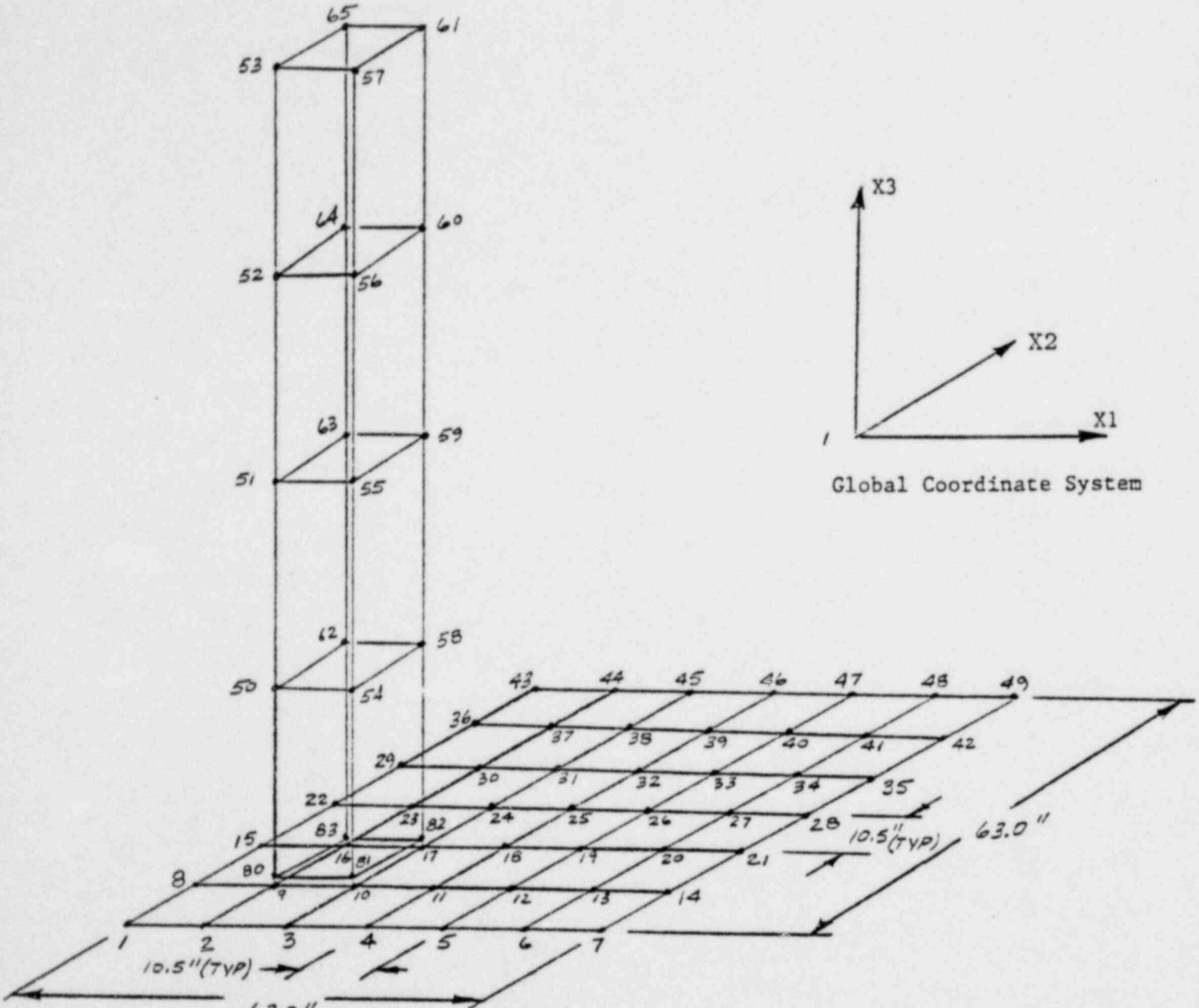


FIGURE 7.1.a

FINITE ELEMENT MODEL - DIMENSIONS AND NODE NUMBERS

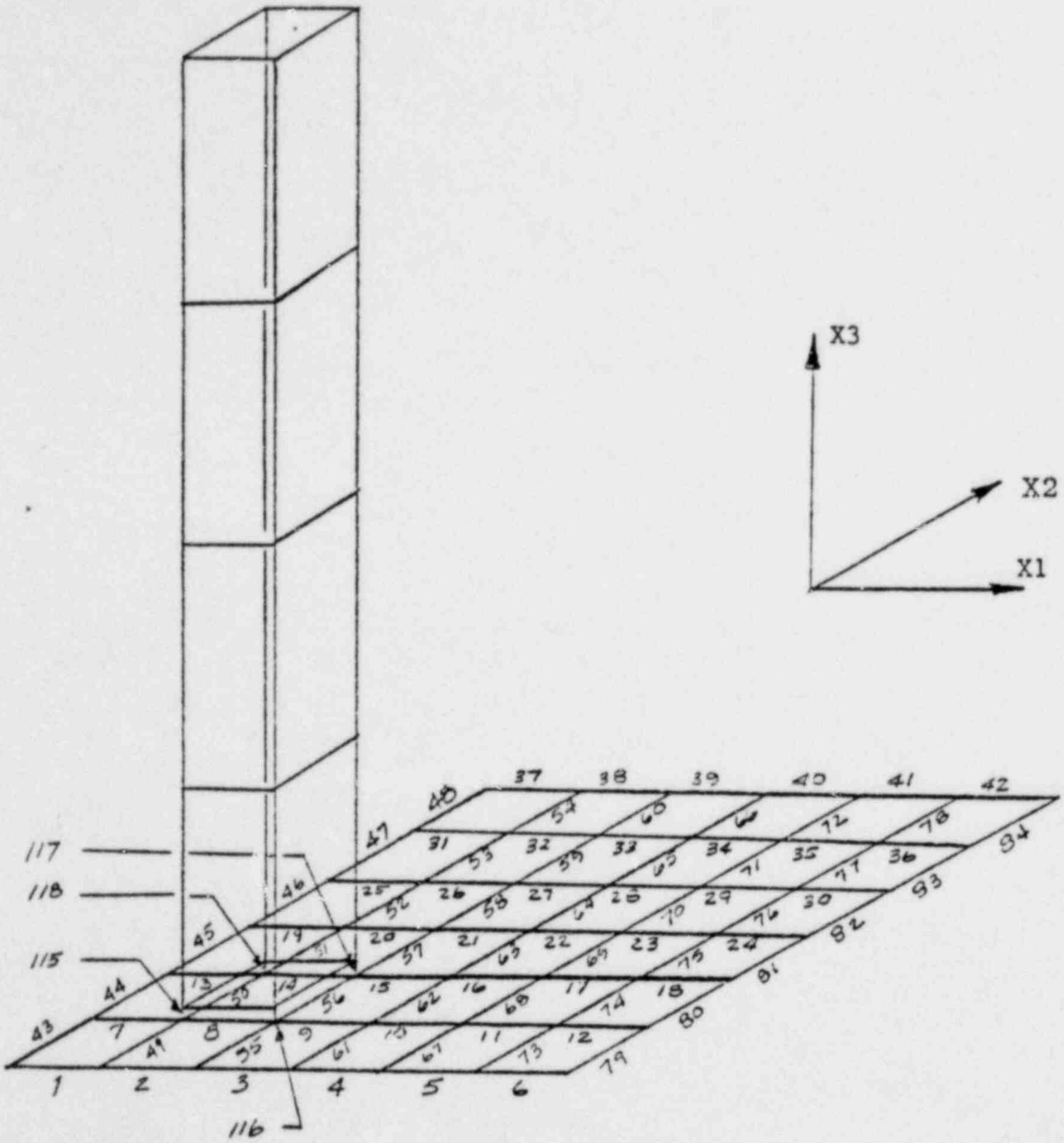


FIGURE 7.1.b

FINITE ELEMENT MODEL - BEAM ELEMENT NUMBERS

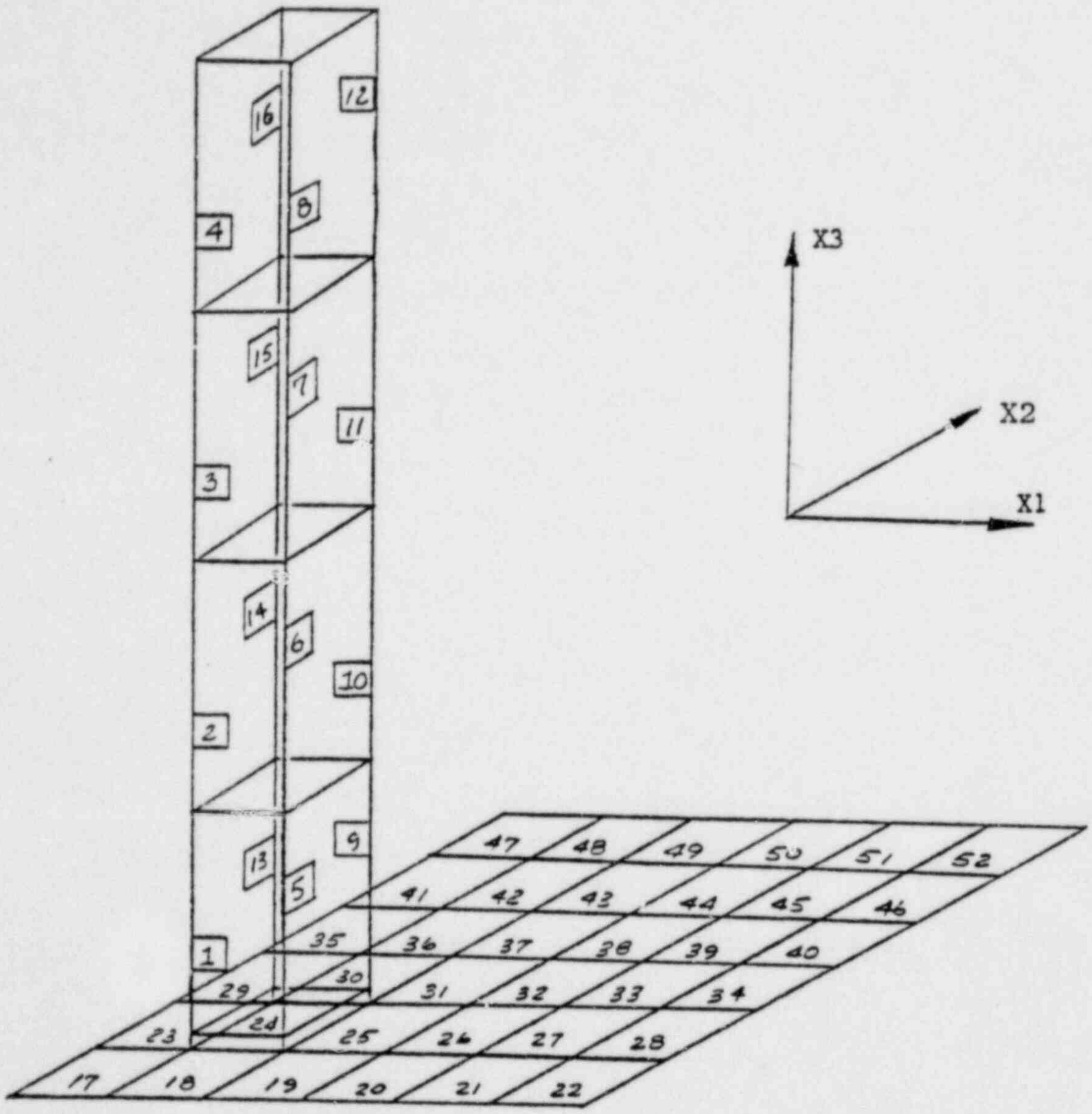


FIGURE 7.1.c

FINITE ELEMENT MODEL - PLATE ELEMENT NUMBERING

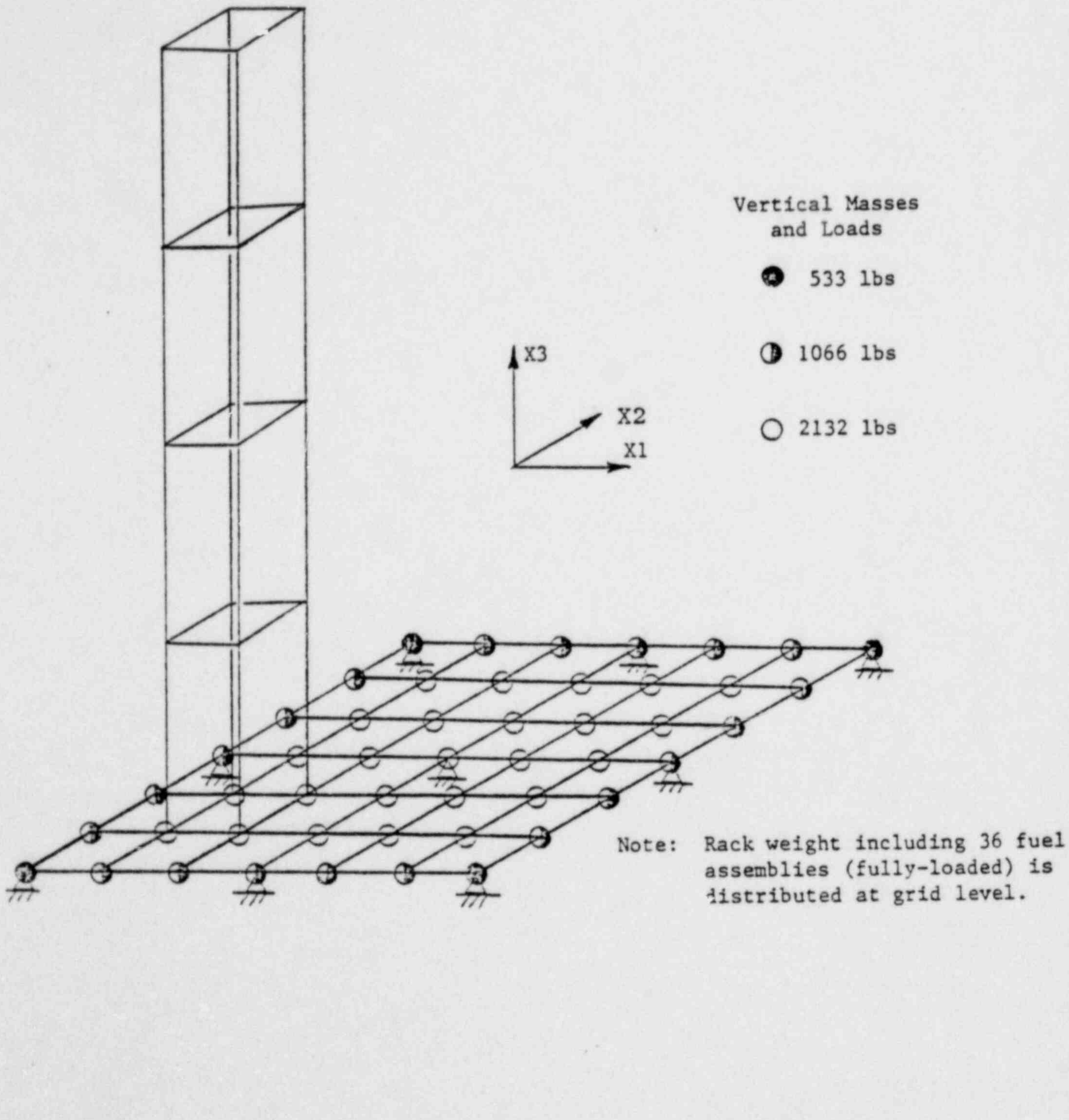
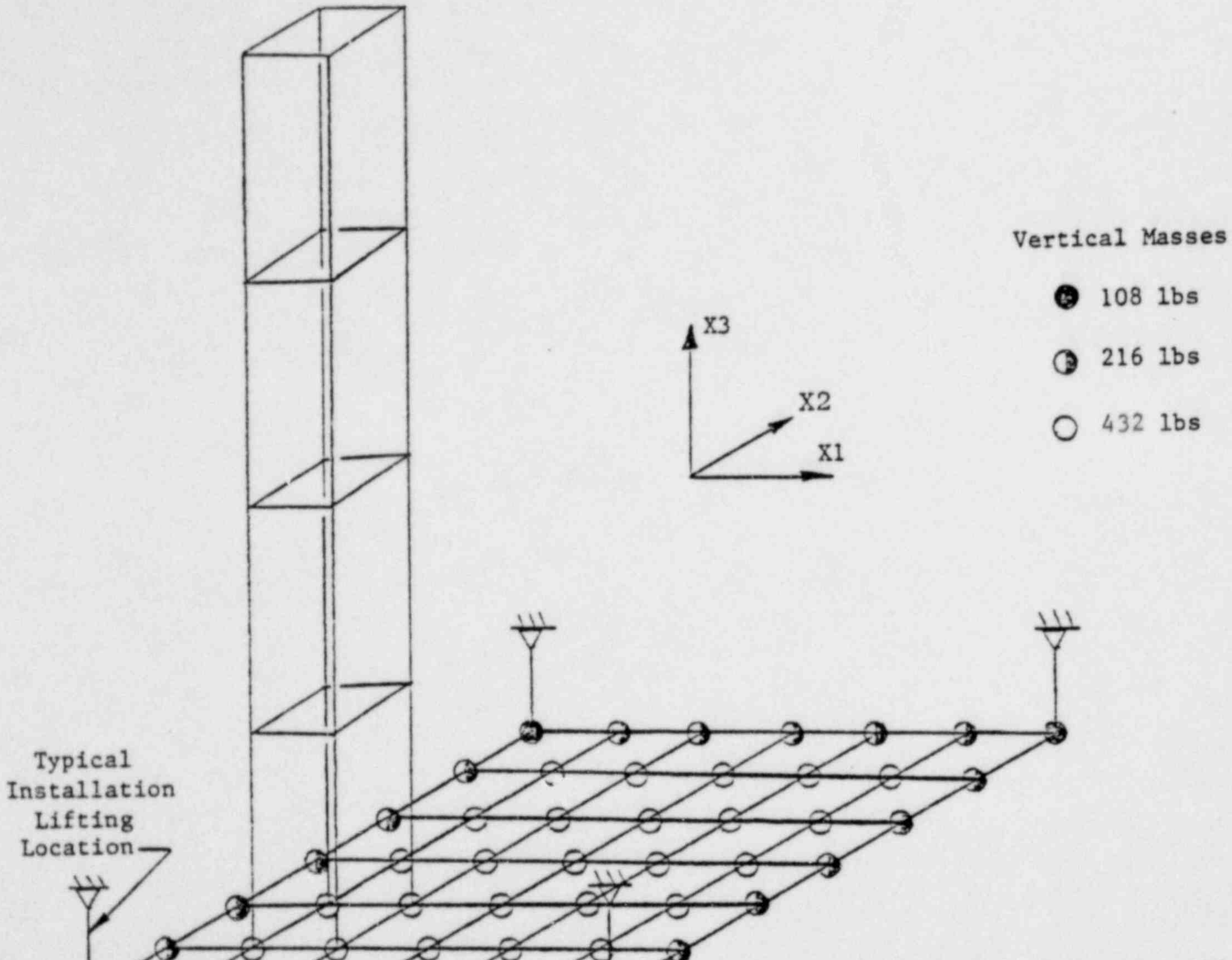


FIGURE 7.2

APPLIED LOADS AND BOUNDARY CONDITIONS - LOAD CASE 1
DEAD WEIGHT OF FULLY-LOADED RACK (36 ASSEMBLIES), D+L



Note: A load equal to 2 G. is applied vertically downward to empty rack to simulate an installation impact load.

FIGURE 7.3

APPLIED LOADS AND BOUNDARY CONDITIONS - LOAD CASE 2
EMPTY RACK DEAD WEIGHT PLUS INSTALLATION LOAD ANALYSIS (D+I.L)

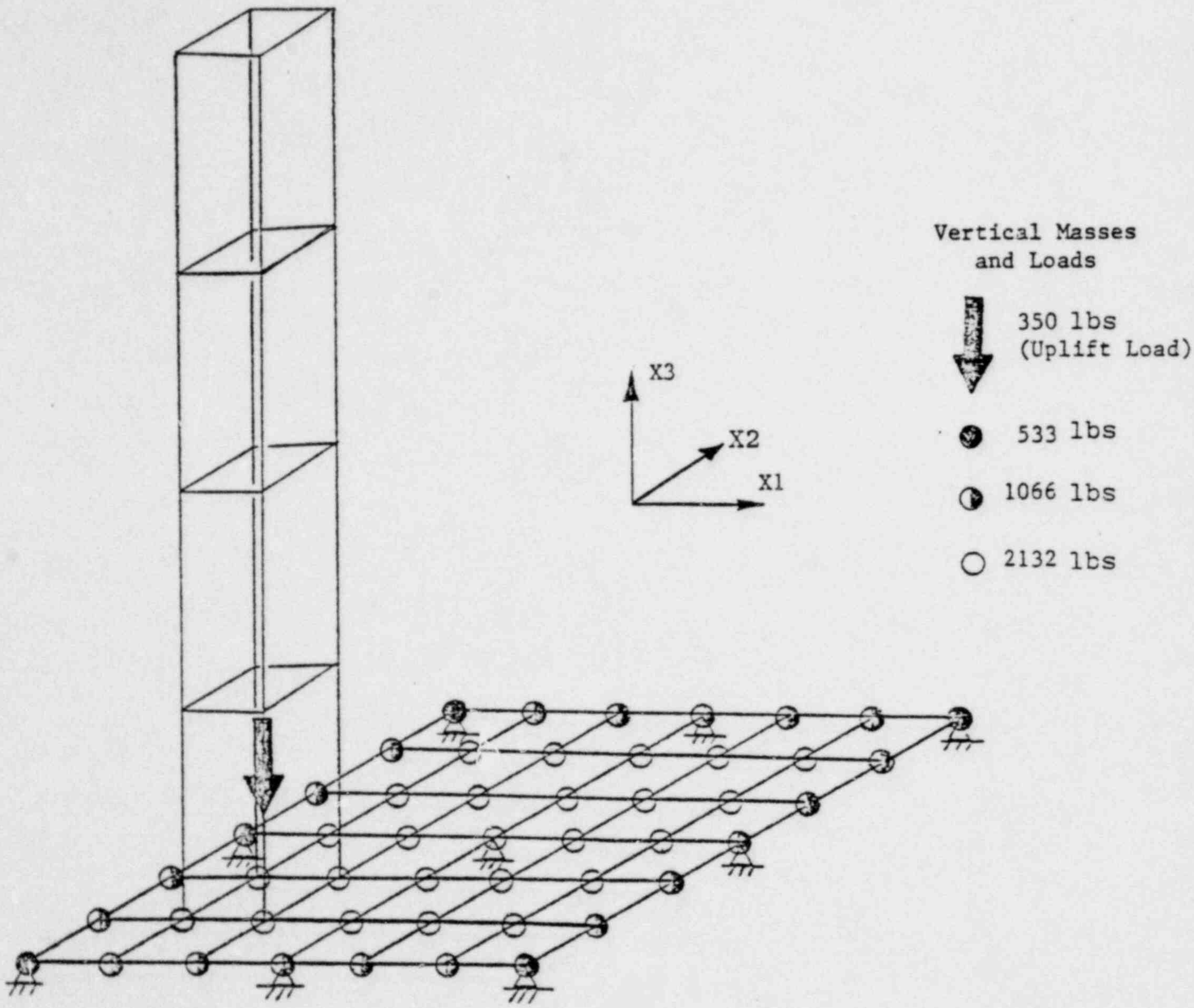


FIGURE 7.4

APPLIED LOADS AND BOUNDARY CONDITIONS - LOAD CASE 3
ACCIDENTAL UPLIFT LOAD (D+U.L)

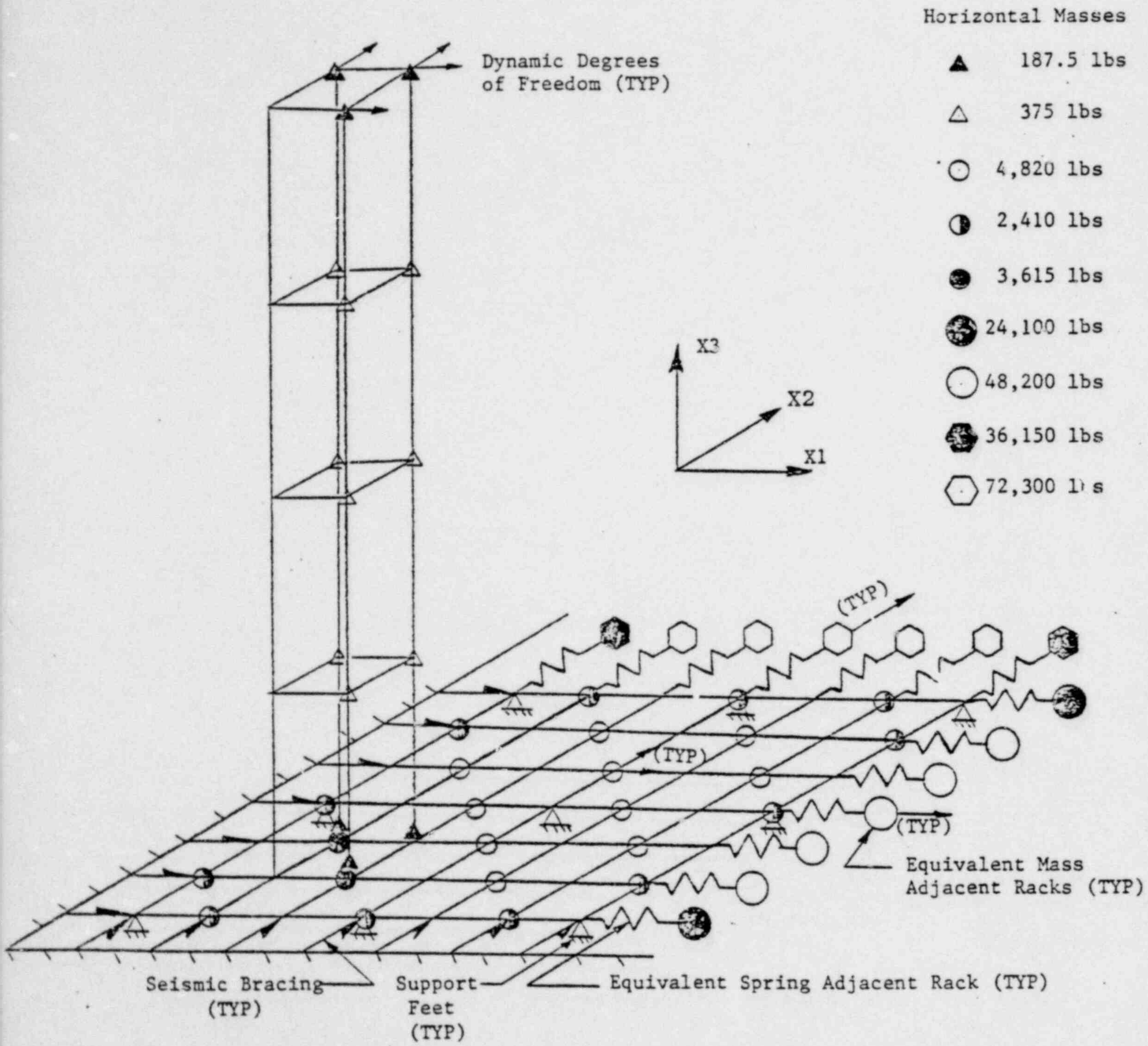


FIGURE 7.5

HORIZONTAL LUMPED MASSES AND BOUNDARY CONDITIONS - LOAD CASES 4 AND 5
SEISMIC ANALYSIS OF FULLY-LOADED RACK (36 ASSEMBLIES) (E AND E')

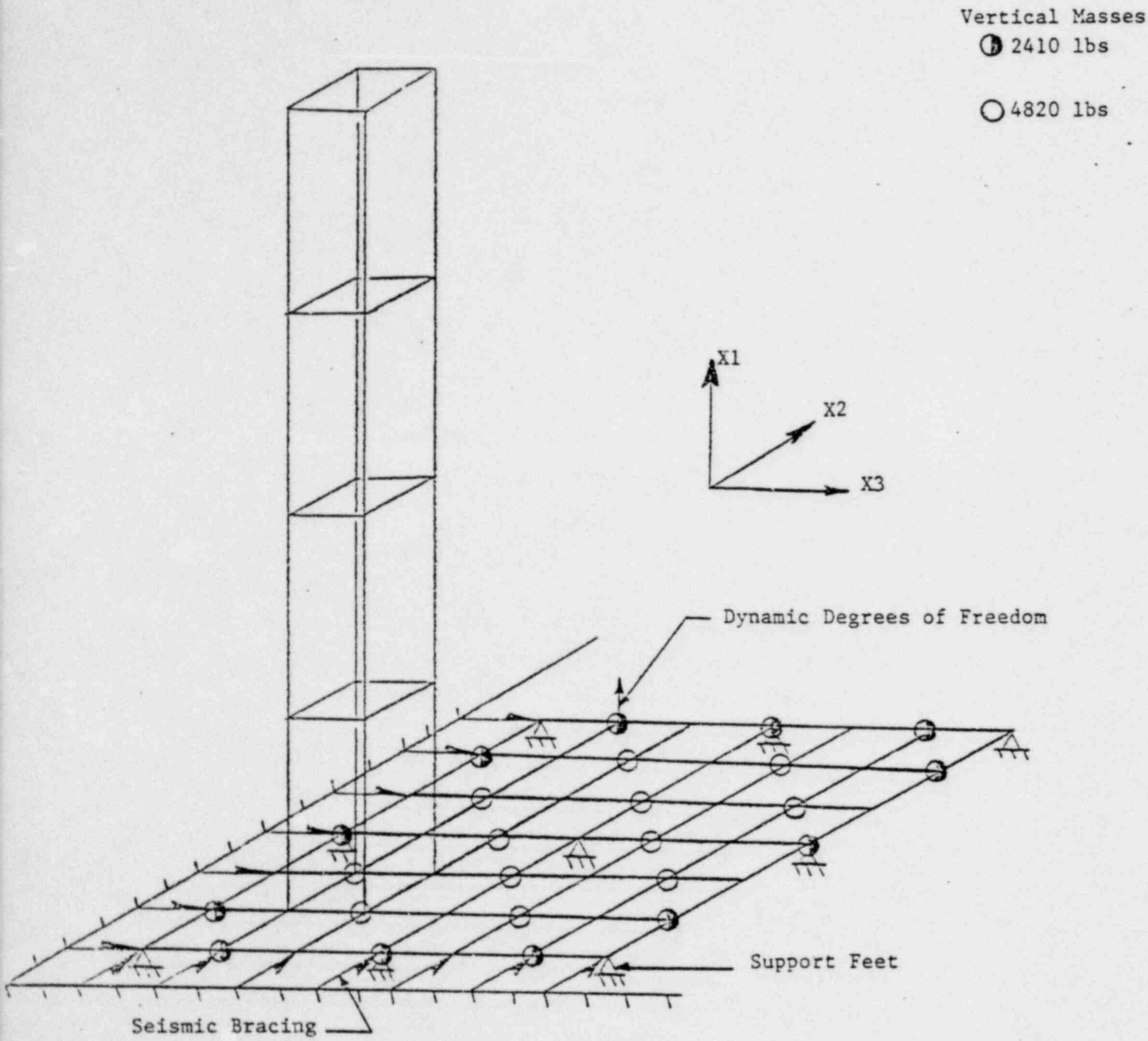


FIGURE 7.6

VERTICAL LUMPED MASSES AND BOUNDARY CONDITIONS - LOAD CASES 4 AND 5
SEISMIC ANALYSIS OF FULLY-LOADED MODEL (36 ASSEMBLIES) (E AND E')

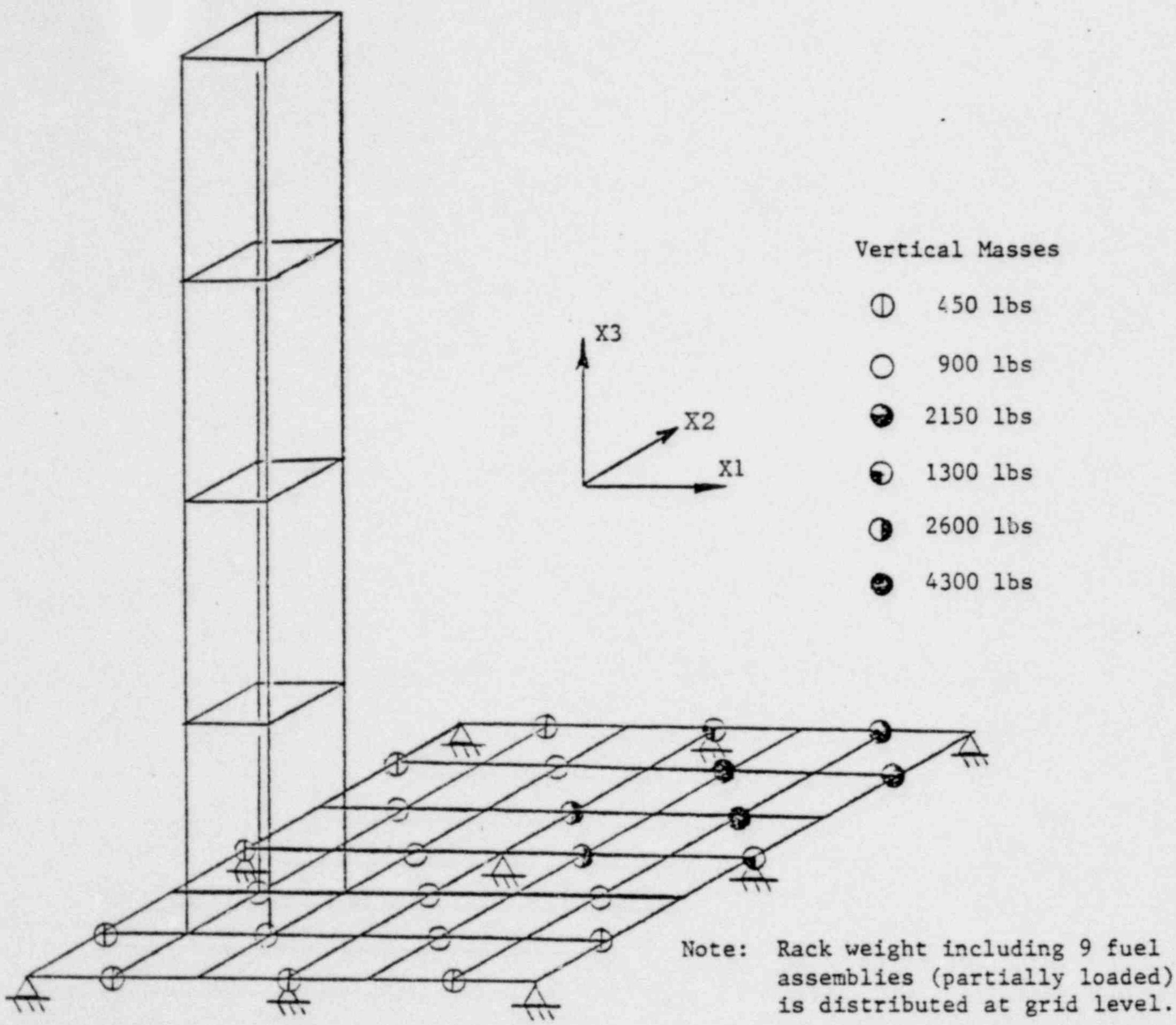


FIGURE 7.7

APPLIED LOADS AND BOUNDARY CONDITIONS - LOAD CASE 1
DEAD WEIGHT OF PARTIALLY-LOADED RACK (9 ASSEMBLIES) D+L'

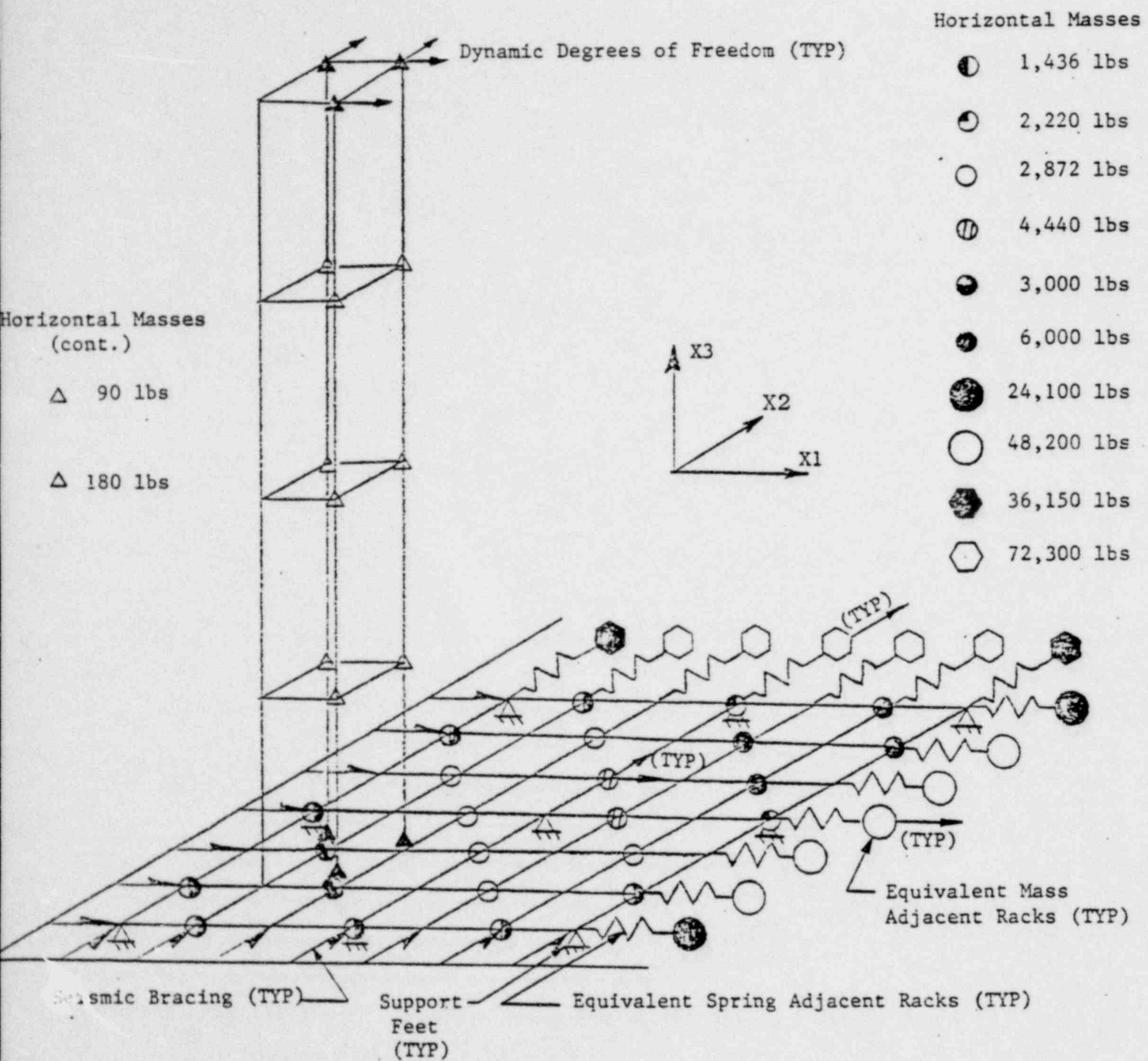


FIGURE 7.8

HORIZONTAL LUMPED MASSES AND BOUNDARY CONDITIONS - LOAD CASES 4 AND 5
SEISMIC ANALYSIS OF PARTIALLY-LOADED MODEL (9 ASSEMBLIES) (E AND E')

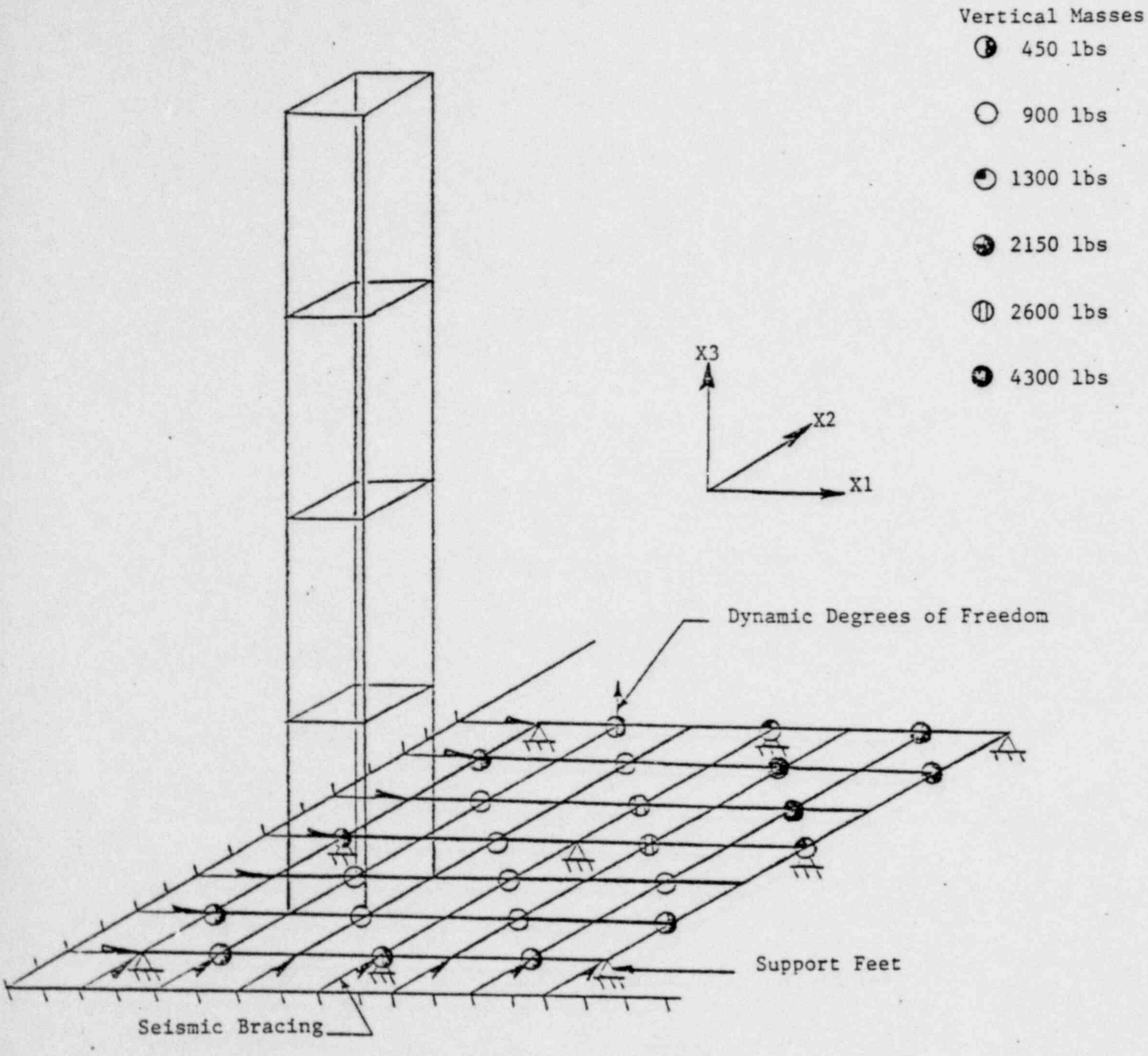


FIGURE 7.9

VERTICAL LUMPED MASSES AND BOUNDARY CONDITIONS - LOAD CASES 4 AND 5
SEISMIC ANALYSIS OF PARTIALLY-LOADED MODEL (9 ASSEMBLIES) (E AND E')

8. THE RESULTS OF THE ANALYSIS

The results of static and seismic structural/stress analysis of the Crystal River Unit No. 3 high density fuel storage racks performed with the STARDYNE computer code are contained in Reference 8.

Appendices A through H contain the beam properties table used in the rack analysis, the thermal expansion analysis, the water sloshing analysis, the effective fuel assembly impact factor calculation, the rack stability analysis, the accidental fuel assembly drop analysis, and the seismic wall bracing and failed fuel canister rack stress analysis.

8.1 Rack Structural/Stress Analysis

The natural frequencies of vibration of the representative 6x6 fuel storage rack are given in Table 8.1 along with their corresponding modal participation factors. For the fully-loaded rack, the first mode frequencies of 4.37 and 4.35 cycles per second represent the combined first mode frequency of the rack base structure and the fundamental storage cell frequency in the North-South and East-West directions, respectively. Similarly, the combined first mode frequencies for the partially-loaded rack are 6.30 and 6.28 cycles per second. The frequencies for the partially-loaded rack are higher than that for the fully-loaded rack due to its reduced weight.

The results of the rack structural/stress analysis, which includes fuel assembly impact, are summarized in Tables 8.2.a for the fully-loaded rack and 8.2.b for the partially-loaded rack. These tables present the maximum stresses and deflections in each type of rack structural member for the various load combinations developed in accordance with the NRC Standard Review Plan, Section 3.8.4 and compares them with the allowable values as specified in the acceptance criteria of Section 6. From these tables it can be seen that the maximum stresses and deflections in various structural members of the racks are nominal and within the allowable limits.

8.2 Spent Fuel Pool Wall and Floor Loads

The maximum reaction loads transmitted to the pool walls and floor of both spent fuel pools A and B resulting from the dead weight and seismic accelerations are presented in Table 8.3. These maximum reaction loads are calculated by considering each pool to be fully-loaded with spent fuel storage racks, failed fuel canisters, and the full compliment of spent fuel assemblies including control rods as specified in the design criteria (Reference 3).

8.3 Water Sloshing Effects

Detail calculations to evaluate the effects of sloshing water on the fuel storage racks indicate that the upper 13.68 feet of pool water for the north/south earthquake and the upper 13.68 feet of pool water for the east/west earthquake will be subject to sloshing. The remaining pool water will move as a constrained solid mass in rigid contact with the pool walls. Consequently, there will be 10.51 feet and 10.10 feet of contained water for the north/south and east/west earthquake respectively between the top of the fuel racks and the sloshing water mass. Therefore, it has been concluded that sloshing of pool water in a seismic event will have insignificant effects on the fuel storage racks.

8.4 Seismic Bracing and Failed Fuel Canister Racks

The structural design and stress analysis of the seismic bracing and failed fuel canister racks are presented in detail in Appendices G and H with the results summarized in Table 8.4. The stresses in these structures, as well as resulting fuel pool concrete bearing stresses, are within allowable limits of the AISC Code (Reference 5) and the ACI Code (Reference 9) in accordance with the NRC Standard Review Plan, Section 3.8.4.

8.5 Accidental Fuel Assembly Drop Analysis

The results of the fuel assembly drop analysis using energy balance methods are summarized in Table 8.5. From Table 8.5 it can be seen that for the straight drop of fuel assembly on top of the storage cell, the maximum stress in the fuel storage cell is greater than the dynamic yield stress for stainless steel, thus indicating that the fuel storage cell will undergo some local permanent deformation, however, due to the lateral supports provided by the adjacent cells, the cell will not collapse during such an accident event. From Table 8.5 it can also be seen that the maximum shearing stress in the weld between the cell walls and the cell base plate and the maximum bending stress in the 8 inch diameter bearing plate under each leg exceed the allowable values, thus indicating that during the fuel drop accident event, the weld between the cell and the cell base plate will partially shear off and the bearing plate will bend resulting in local crumbling of concrete under each leg. The external kinetic energy of the dropped fuel will be absorbed in the local deformation of the flare at the top of the fuel storage cell, in the partial shearing of the cell wall/cell base plate weld, in the local crumbling of the concrete and in the minor deformation of the liner plate under the rack support feet.

For the case of the inclined drop of the fuel assembly on top of the storage rack, the maximum external kinetic energy (24.16 in.k.) per storage cell is considerably less than the kinetic energy (57.8 in.k.) for the straight drop of fuel assembly on top of the storage cell. Therefore, the damages to the storage rack and the liner plate resulting from the inclined drop of fuel assembly on top of the storage rack will be less severe than that of the straight drop of fuel assembly on top of the storage cell.

The free fall of fuel assembly through the storage cell from a height of 34.0 inches above the top of the storage cell and its impact on top of the cell base plate and rack base structure was analyzed using empirical missile equations (the Ballistic Research Laboratory and Stanford Research Institute Formulae). The results of the analysis indicates that the maximum thickness of steel plate that could be perforated by such a missile is greater than the thickness of the cell base plate. However, the flange member of the rack base structure will resist further penetration of the fuel assembly. Since for this fuel assembly drop case the external energy is absorbed in the flexural deformation of the flexible cell base plate and rack base structure, the reaction load transmitted to the rack base structure, rack feet and pool floor is less than that for fuel assembly drop on top of the storage cell. Therefore, the damage to the pool floor will be less severe for the fuel assembly drop through the storage cell than that for the fuel assembly drop on top of the storage cell. The analysis indicates that the cell base plate and the rack base structure will undergo extensive local deformations; however, the overall structural integrity of the storage rack structure and the leak-tight integrity of the pool floor will be maintained.

It should be noted that the fuel assembly drop analyses have been performed by conservatively assuming that no energy will be absorbed by the fuel assembly itself. The energy absorbed in the deformation of the flexible fuel assembly should result in reduced damage to the storage rack and the pool liner plate than that predicted by the conservative analysis.

It has, therefore, been concluded that neither the straight or inclined drop of the fuel assembly on top of the storage cell or the straight drop of the fuel assembly through the storage cell and impact on top of the rack base structure will damage the storage rack and the pool liner plate sufficiently to adversely affect the value of k_{eff} or the leak-tight integrity of the pool.

8.6 Rack Stability Analysis

The stability analysis of the fuel storage racks using energy balance methods are given in Appendix E. The analysis indicates that during an OBE event the storage racks will not lift up and during an SSE event the legs of the storage racks could lift up as much as 0.081 inches; however, the racks will not overturn.

The maximum stresses in the rack structure rack feet and pool liner plate on recontact with the pool floor are within their allowable stress values. Therefore, it has been concluded that during a seismic event the fuel storage racks will remain stable and the structural integrity of the rack structure and the leak-tight integrity of the liner plate will not be adversely affected by the lift up and recontact of the storage racks.

TABLE 8.1

NATURAL FREQUENCIES OF VIBRATION

AND MODAL PARTICIPATION FACTORS

	Mode Number	Frequency (CPS)	Modal Participation Factor ($X10^{-1}$)
	Case 1 - Fully-Loaded Rack (36 Assemblies)		
X1 Direction (N-S)	1	4.37	15.2200
	2	11.74	0.0222
	3	17.57	12.0180
	4	18.68	0.2171
	5	19.71	-1.8210
	6	20.22	-0.0433
	7	20.46	0.3553
	8	23.48	-0.1152
	9	26.20	-6.8702
	10	28.34	6.4800
	11	31.70	8.6130
X2 Direction (E-W)	1	4.35	15.2680
	2	11.73	0.1950
	3	12.48	11.7710
	4	13.06	0.1837
	5	13.55	-1.7611
	6	13.78	-0.0385
	7	13.90	0.3447
	8	19.93	6.8378
	9	20.18	9.2786
	10	23.46	0.0803
	11	26.18	6.6378
X3 Direction (Vertical)	1	56.3	27.6400
	2	58.8	
	3	58.8	

TABLE 8.1 (cont.)

NATURAL FREQUENCIES OF VIBRATION

AND MODAL PARTICIPATION FACTORS

	Mode Number	Frequency (CPS)	Modal Participation Factor ($X10^{-1}$)
Case 2 - Partially-Loaded Rack (9 Assemblies)			
X1 Direction (N-S)	1	6.30	15.3284
	2	16.94	0.4910
	3	17.59	11.9309
	4	18.69	0.0954
	5	19.72	-1.8127
	6	20.22	-0.0401
	7	20.46	0.3544
	8	28.32	6.7109
	9	28.86	8.3323
	10	33.89	0.1010
X2 Direction (E-W)	1	6.28	15.4623
	2	12.49	11.7261
	3	13.06	0.1293
	4	13.55	-1.7572
	5	13.78	-0.0371
	6	13.90	3.4435
	7	16.93	-0.1049
	8	19.93	6.9985
	9	20.19	9.0867
	10	33.86	0.0329
X3 Direction (Vertical)	1	65.26	12.16
	2	95.33	-0.0
	3	106.02	-3.23
	4	121.85	-0.0

TABLE 8.2.a

RESULTS OF STRUCTURAL/STRESS ANALYSIS - FULLY LOADED RACK

Load Combination	Structural Element Description	Maximum Beam Deflections, inches: Node No.	Deflection	Allowable L/360	Maximum Beam Stresses, ksi: Element No.	Axial**: Calculated fa	Allowable Fa	Bending#: Calculated $f_{b2} + f_{b3}$	Allowable F_b	Combined Stress Ratio*
1. D+L (1a D+L+T) Dead Weight of Rack (Plus Negligible Thermal Load	N-S Edge Member	2,6 (TYP)	-0.0017	0.0875	3, 4 (TYP)	0.0	16.68	1.71	19.8	0.086
	N-S Interior Member	17,19(TYP)	-0.0035	0.175	21, 22	0.0	16.32	3.41	19.8	0.172
	E-W Edge Member	8,14(TYP)	-0.0017	0.0875	45, 46 (TYP)	0.0	16.68	1.71	19.8	0.086
	E-W Interior Member	17,19(TYP)	-0.0035	0.175	63, 64	0.0	16.32	3.41	19.8	0.172
	Quad-Plate (Storage Cell)	65	0.0059	0.453	1 (TYP)	Negligible	18.00	Negligible	12.00	
					17 (TYP)	Negligible	18.00	Negligible	12.00	
	Quad-Plate (Rack Base)				18 (TYP)	Negligible	18.00	Negligible	12.00	
2. D+I.L Dead Weight of Rack Plus 1.0G Vertical Installation Load	N-S Edge Member	4, 46	-0.0183	0.0875	3, 4 (TYP)	0.0	16.68	4.51	19.8	0.228
	N-S Interior Member	25	-0.0226	0.175	9, 10(TYP)	0.0	13.72	2.77	19.8	0.140
	E-W Edge Member	22, 28	-0.0183	0.0875	45, 46 (TYP)	0.0	16.68	4.52	19.8	0.228
	E-W Interior Member	25	-0.0226	0.175	51, 52 (TYP)	0.0	13.72	2.77	19.8	0.140
	Quad-Plate (Storage Cell)	65	0.0487	0.453	1 (TYP)	Negligible	18.00	Negligible	12.00	
					17	0.1	18.00	0.16	12.00	
	Quad-Plate (Rack Base)				18	0.12	18.00	0.15	12.00	
3. D+L+E (3a D+L+T+E) Dead Weight of Rack, Fuel Assembly Weight Plus OBE Seismic Event (Negligible Thermal Load Included)	N-S Edge Member	46	0.0146 (X2)	0.0875	37	1.37	16.68	6.26	19.8	0.398
	N-S Interior Member	39	0.0117 (X2)	0.175	31	1.2	16.32	5.56	19.8	0.369
	E-W Edge Member	28	0.0061 (X1)	0.0875	45	2.31	16.68	5.21	19.8	0.402
	E-W Interior Member	27	0.0050 (X1)	0.175	54	6.23	13.72	5.75	19.8	0.716
	Quad-Plate (Storage Cell)	65	0.2260 (X2)	0.453	9	3.19	18.00	0.22	12.00	
					50	8.00	18.00	0.77	12.00	
	Quad-Plate (Rack Base)				47	4.08	18.00	3.55	12.00	
4. D+L+T+E' Dead Weight of Rack, Fuel Assembly Weight Plus SSE Seismic Event (Negligible Thermal Load Included)	N-S Edge Member	46	0.0292 (X2)	0.0875	37	2.73	26.68	10.98	27.0	0.509
	N-S Interior Member	39	0.0233 (X2)	0.175	31	2.41	21.95	10.67	27.0	0.505
	E-W Edge Member	28	0.0123 (X1)	0.0875	45	7.83	26.68	6.21	27.0	0.495
	E-W Interior Member	39	0.0137 (X1)	0.0875	54	12.47	21.55	11.06	27.0	0.939
	Quad-Plate (Storage Cell)	65	0.450 (X2)	0.453	5 (TYP)	6.5	28.8	0.45	19.6	
					50	15.91	28.8	1.57	19.6	
	Quad-Plate (Rack Base)				47	8.2	28.8	7.10	19.6	
5. D+T+U.L. Dead Weight of Rack, Fuel Assembly Weight Plus 350# Uplift Load (Negligible Thermal Load Included)	N-S Edge Member	6, 44	-0.00172	0.0875	39	0.0	26.68	1.73	27.0	0.064
	N-S Interior Member	38	-0.0034	0.175	21	0.0	26.68	3.32	27.0	0.123
	E-W Edge Member	14	-0.00172	0.0875	82	0.0	26.68	1.73	27.0	0.064
	E-W Interior Member	38	-0.0034	0.175	63	0.0	26.68	3.27	27.0	0.123
	Quad-Plate (Storage Cell)	57	-0.00938	0.453	1 (TYP)	0.47	28.8	1.57	19.6	
					17 (TYP)	Negligible	28.8	Negligible	19.6	
	Quad-Plate (Rack Base)				18 (TYP)	Negligible	28.8	Negligible	19.6	

* Must be ≤ 1.0 for component acceptability per AISC Specification, Section 1.6.1
** For quad plate results, stresses are a combination of axial plus bending stresses
\# For quad plate results, stresses are the maximum shearing stresses

TABLE 8.2.b

RESULTS OF STRUCTURAL/STRESS ANALYSIS - PARTIALLY LOADED RACK

Load Combination	Structural Element Description	Maximum Beam Deflections, inches: Node No.	Maximum Beam Deflections, inches: Deflection	Maximum Beam Deflections, inches: Allowable L/360	Element No.	Maximum Beam Stresses, ksi: Axial[**]: Calculated f_a	Maximum Beam Stresses, ksi: Axial[**]: Allowable F_a	Maximum Beam Stresses, ksi: Bending[#]: Calculated $f_{b2} + f_{b3}$	Maximum Beam Stresses, ksi: Bending[#]: Allowable F_b	Combined Stress Ratio[*]
1a. (D+Lp+T) Dead Weight of Rack Plus 9 Assemblies (Including Negligible Thermal Load)	N-S Edge Member	48	-0.0017	0.0875	41, 42	0.0	16.68	1.57	19.8	0.079
	N-S Interior Member	34	-0.0027	0.1750	22, 23	0.0	16.68	1.14	19.8	0.058
	E-W Edge Member	42	-0.0017	0.0875	83, 84	0.0	16.68	1.57	19.8	0.079
	E-W Interior Member	34	-0.0027	0.175	63, 64	0.0	16.68	1.24	19.8	0.063
	Quad-Plate (Storage Cell)	65	0.9009(X1	0.453	1 (TYP)	Negligible	18.0	Negligible	12.0	
	Quad-Plate (Rack Base)				17 (TYP)	Negligible	18.0	Negligible	12.0	
					18 (TYP)	Negligible	18.0	Negligible	12.0	
3a. (D+Lp+E+T) Dead Weight of Rack, 9 Fuel Assemblies Plus OBE Seismic Results (Including Negligible Thermal Loads)	N-S Edge Member	46	0.0119	0.0875	42	0.43	16.68	5.87	19.8	0.322
	N-S Interior Member	39	0.0095	0.175	24	2.43	16.32	2.75	19.8	0.288
	E-W Edge Member	28	0.0047	0.0875	83	1.82	16.68	3.92	19.8	0.307
	E-W Interior Member	27	0.0038	0.0875	54	5.0	13.72	4.19	19.8	0.545
	Quad-Plate (Storage Cell)	65	0.0940(X2	0.453	5	1.51	18.00	0.009	12.00	
	Quad-Plate (Rack Base)				50	6.48	18.00	0.64	12.00	
					47	3.28	18.00	2.84	12.00	
4a. (D+Lp+E'+T) Dead Weight of Rack, 9 Fuel Assemblies Plus SSE Seismic Results (Including Negligible Thermal Loads)	N-S Edge Member	46	0.0237	0.0875	42	0.86	26.68	10.17	27.0	0.409
	N-S Interior Member	39	0.0190	0.1750	31	1.87	21.95	7.51	27.0	0.363
	E-W Edge Members	28	0.0094	0.0875	83	2.21	26.68	7.79	27.0	0.371
	E-W Interior Members	27	0.0050	0.0875	54	10.0	21.95	8.27	27.0	0.717
	Quad-Plate (Storage Cell)	65	0.1871	0.453	5	3.03	28.8	0.19	19.6	
	Quad-Plate (Rack Base)				50	13.03	28.8	1.26	19.6	
					47	6.57	28.8	5.67	19.6	

* Must be ≤ 1.0 for component acceptability per AISC Specification, Section 1.6.1

** For quad plate results, stresses are a combination of axial plus bending stresses

\# For quad plate results, stresses are the maximum shearing stresses

TABLE 8.3

SPENT FUEL WALL AND FLOOR LOADS

Pool A Wall Loading Summary

Load Location (See Figure 3.1)	Max. Wall Load for Each Seismic Bracing* (kips)	
	Operating Basis Earthquake (OBE) 1% Equip. Damping	Safe Shutdown Earthquake (SSE) 1% Equip. Damping
A	13.7	27.4
B	47.1	94.2
C	13.7	27.4
D	47.1	94.2

Wall Designation	Total Wall Load* (kips)	
	OBE 1% Equipment Damping	SSE 1% Equipment Damping
North Wall	479.2	959.0
South Wall	479.2	959.0
East Wall	800.7	1601.0
West Wall	800.7	1601.0

* Center line of seismic bracing is 8.5" above the spent fuel pool floor.

Pool B Wall Loading Summary

Load Location (See Figure 3.2)	Max. Wall Load for Each Seismic Bracing (kips)	
	Operating Basis Earthquake (OBE) 1% Equip. Damping	Safe Shutdown Earthquake (SSE) 1% Equip. Damping
A	21.2	42.8
B	8.5	17.1
C	4.2	8.6
D	41.6	83.1
E	48.5	96.9
F	27.7	55.2

TABLE 8.3 (cont.)

SPENT FUEL WALL AND FLOOR LOADS

Pool B Wall Loading Summary

Wall Designation	Total Wall Load (kips) OBE 1% Equipment Damping	SSE 1% Equipment Damping
North Wall	559.7	1129.9
East Wall (1)	499.0	996.9
South Wall (2)	97.5	196.9
East Wall (2)	415.5	828.0
South Wall (1)	445.2	898.8
West Wall	914.5	1824.9

Pool Floor Load Summary

Load Description	Spent Fuel Storage Rack Array Size	Maximum Floor Loads (kips) OBE 1% Equip. Damping	SSE 1% Equip. Damping
Max. Load For Each of the Corner Feet	6x6	11.4	19.2
	6x5	9.5	16.0
	5x5	7.92	13.3
	4x6	7.6	12.8
	4x5	6.33	10.7
Max. Load For Each of the Corner Feet Center	6x6	30.2	31.8
	6x5	25.2	26.5
	5x5	21.0	22.1
	4x6	20.1	21.2
	4x5	16.8	17.7
Max. Load For Each of the Mid-Edge Feet	6x6	28.9	47.0
	6x5	24.1	39.2
	5x5	20.1	32.6
	4x6	19.3	31.3
	4x5	16.1	26.1
Total Pool Floor Load	Pool A	1686.0	2886.0
	Pool B	1937.0	3115.0

TABLE 8.4

SEISMIC WALL BRACING ANALYSIS RESULTS

POOL A

LOCATION	DESCRIPTION (SEISMIC BRACING)	SEISMIC BRACING COMPRESSIVE STRESSES (KSI)				CONCRETE BEARING PLATE BENDING STRESSES (KSI)				CONCRETE BEARING STRESSES (KSI)			
		OBE		SSE		OBE		SSE		OBE		SSE	
		CALC	ALLOW	CALC	ALLOW	CALC	ALLOW	CALC	ALLOW	CALC	ALLOW	CALC	ALLOW
North Wall	3.5" SCH 40 XS 40S TYPE	4.91	17.90	9.83	28.63	9.83	19.8	19.6	27.0	0.539	1.05	1.15	1.785
South Wall	3.5" SCH 40 ST 40S TYPE	4.91	17.90	9.83	28.63	9.83	19.8	19.6	27.0	0.539	1.05	1.15	1.785
East Wall	6" SCH 40 ST 40S TYPE	8.69	17.93	17.38	28.69	11.09	19.8	22.2	27.0	0.764	1.05	1.53	1.785
West Wall	6" SCH 40 ST 40S TYPE	8.69	17.93	17.38	28.69	11.09	19.8	22.2	27.0	0.764	1.05	1.53	1.785

TABLE 8.5

RESULTS OF ACCIDENTAL FUEL ASSEMBLY DROP (LOAD CASE 6)

Straight Drop on Top of Storage Cell		Allowable Value
Weight of Fuel Assembly (kip)	1.70	
Maximum Drop Height (in)	34.0	
Kinetic Energy of Drop to be Absorbed (in-k)	57.80	
Maximum Strain in Storage Cell (in/in)	0.00243	0.485[1]
Maximum Cell Deformation (in)	0.397	
Maximum Stress in Cell (ksi)	45.15	41.4[2]
Maximum Transmitted Reaction Load (kips)	224.7	
Maximum Stress in the Weld Between the Cell Wall and the Base Plate (ksi)	52.8	38.6[3]
Maximum Stress in Rack Base Structure (ksi)	26.5	41.4
Maximum Stress in Jackscrew (ksi)	111.6	180.0[4]
Maximum Local Bearing Stress on Concrete Floor (ksi)	2.86	3.57[5]
Maximum Bending Stress in the Bearing Plate (ksi)	34.32	41.4
Maximum Punching Shear Stress in the Liner Plate (ksi)	32.6	41.4
Inclined Drop on Top of Storage Cell		
Maximum External Kinetic Energy per Storage Cell (in-k)	24.16<57.80	

TABLE 8.5 (cont.)

RESULTS OF ACCIDENTAL FUEL ASSEMBLY DROP (LOAD CASE 6)

Straight Drop Through the Storage Cell		Allowable Value
Maximum Free Fall Impact Velocity (ft/sec)	32.58	
Maximum External Kinetic Energy (in.k.)	336.6	
Maximum Unsupported Plate Thickness That May be Perforated by Missile Free Fall Velocity, in		
BRL Formula	0.460	0.50
Stanford Research Institute Formula	0.445	0.50
Maximum Deformation of the Storage Cell Base Plate and Flange Member of The Rack Base Structure (in.)	2.08	
Maximum Transmitted Reaction Load (kips)	141.9	

1. Ultimate strain for stainless steel.
2. The allowable stress value represents dynamic yield stress for stainless steel.
3. Allowable stress in the weld - $1.6 \times 21 \times \frac{41.4}{36} = 38.6$ ksi.
4. Yield stress value for 17-4PH stainless steel.
5. Based on Paragraph 10.14 of ACI 318-71. There will be local crumbling of concrete under each leg but the steel liner will not be perforated.

9. CONCLUSIONS

1. The results of the seismic and structural analysis indicate that the deflections and/or stresses in the rack structure resulting from the loadings associated with the normal and abnormal conditions are within allowable deflection and stress limits for Seismic Category I structures.

2. Sloshing of pool waters in a seismic event will have insignificant effects on the fuel storage racks.

3. The earthquake generated stresses in the seismic wall bracing and failed fuel canister racks are within the specified allowable values.

4. The analysis of the accidental fuel assembly drop condition indicates acceptable local structural damage to the storage cells with no buckling or collapse and local crumbling of the pool concrete floor with no puncturing of the stainless steel liner. Therefore, no significant changes in the value of k_{eff} will occur and the leak tightness of the fuel pool will be maintained.

5. It is concluded that the designs of the Crystal River Unit 3 high density fuel storage racks, the associated seismic bracing, and the failed fuel canister racks are adequate to withstand the loadings of normal and abnormal conditions.

10. REFERENCES

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APPENDIX

A Member Property Table

B Thermal Expansion Analysis

C Water Sloshing Analysis

D Impact Factor to Account for the Impact Between Fuel Assembly and Storage Cell

E Stability Analysis for Crystal River Unit 3 Fuel Storage Racks

F Accidental Fuel Assembly Drop Analysis

G Seismic Wall Bracing Design and Analysis

H Failed Fuel Canister Analysis

APPENDIX N

TYPICAL MEMBER PROPERTIES

CRYSTAL RIVER SPENT FUEL STORAGE RACK "MEMBER PROPERTIES"

BEAM NOS. (TYP)	BEAM DESCRIPTION	AREA (SQ. IN)	I_2 (IN^4)	ℓ_2 (IN)	r_2 (IN)	$\frac{k\ell_2}{r_2}$	I_3 (IN^4)	ℓ_3 (IN)	r_3 (IN)	$\frac{k\ell_3}{r_3}$
1-6 37-42	GRID N-S BOUNDARY	6.75	48.54	63.0	2.68	(k=1) 23.51	6.82	10.5	1.01	10.4
43-48 79-84	GRID E-W BOUNDARY	6.75	48.54	63.0	2.68	(k=1) 23.51	6.82	10.5	1.01	10.4
7-12 (TYP)	GRID N-S INTERIOR	5.72	55.55	63.0	3.12	(k=1) 20.2	4.03	10.5	0.84	12.5
49-54 (TYP)	GRID E-W INTERIOR	5.72	55.55	63.0	3.12	(k=1) 20.2	4.03	10.5	0.84	12.5
	STORAGE CELL	5.31	79.57	163.375	3.87	(k=2.1) 88.65	79.57	163.375	3.87	88.65

APPENDIX B : THERMAL EXPANSION ANALYSIS

MAX. POOL TEMPERATURE IS ASSUMED TO BE 205 F°. (DESIGN CITERIA, REF. 2 - PG. III-3)

MEAN COEFFICIENT OF THERMAL EXPANSION (STAINLESS STEEL) IN A TEMPERATURE CHANGE FROM 70°F TO 205°F IS :

$$\alpha_{mean} = 9.34 \times 10^{-6} \text{ in/in/}^\circ F$$

THERMAL EXPANSION

① N-S DIRECTION (POOL LENGTH = 130")

$$\Delta l_{N-S} = 9.34 \times 10^{-6} \text{ in/in/F}^\circ \times (205-70)^\circ F \times (130)$$

$$= 0.227" \text{ or } \approx 1/4"$$

② E-W DIRECTION (POOL LENGTH = 386")

$$\Delta l_{E-W} = 9.34 \times 10^{-6} \text{ in/in/}^\circ F \times (205-70) \times (386)$$

$$\Delta l_{E-W} = 0.487" \text{ or } \approx \tfrac{1}{2}"$$

MECHANICAL CLEARANCES ARE PROVIDED FOR INSTALLATION OF THE FUEL RACKS. THE SUM OF THE TOTAL CLEARANCES BETWEEN THE RACKS (1/16" TYPICAL BETWEEN RACKS AND 1/8" BETWEEN SEISMIC BRACINGS AND POOL WALLS) AND BETWEEN RACK BRACING & POOL WALL WILL SLIGHTLY EXCEED THAT REQUIRED FOR THERMAL EXPANSION

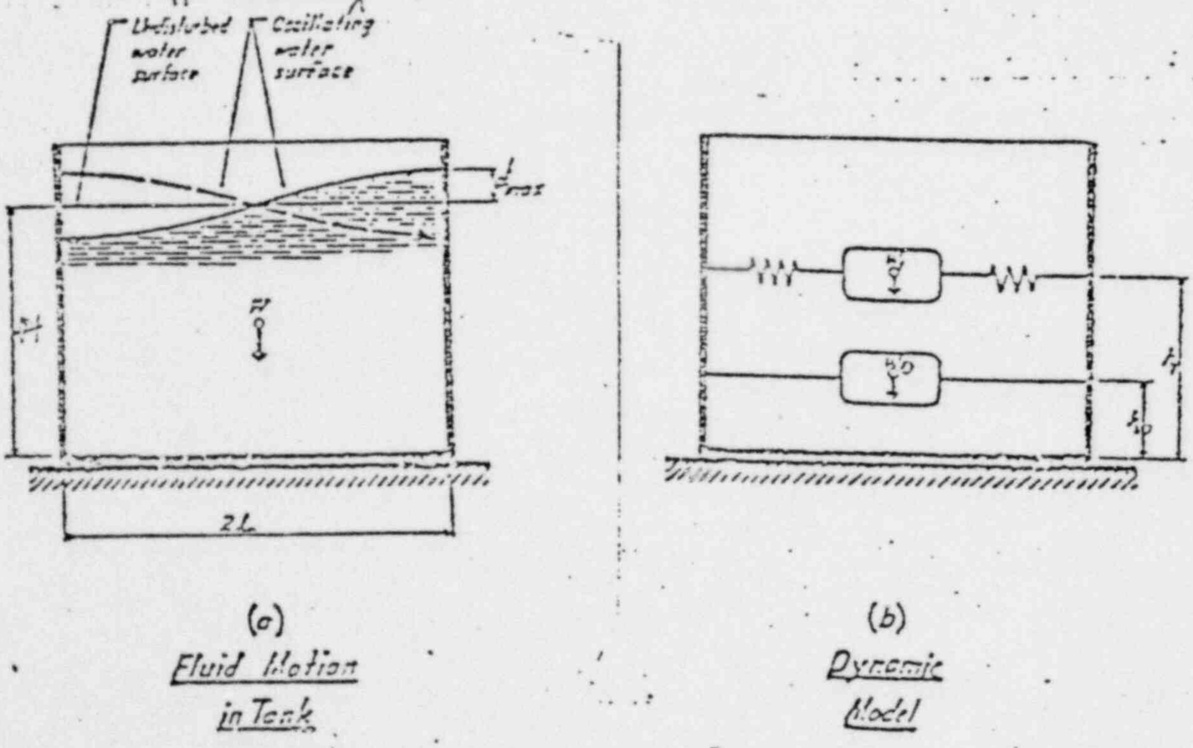
THEREFORE, NO THERMAL STRESSES WILL RESULT FROM RESTRICTED EXPANSION.

WATER SLOSHING EFFECTS - APPENDIX C

The sloshing effects of water on the fuel racks have been evaluated using analytical methods given in the following References

(1) "Structural Analysis and Design of Nuclear Plant Facilities" American Society of Civil Engineers 1976. (Ref. 7)

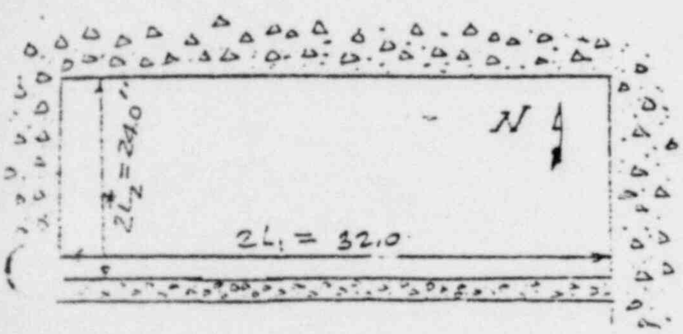
(2) N.M. Newmark; E. Rosenblueth "Fundamentals of Earthquake Engineering"

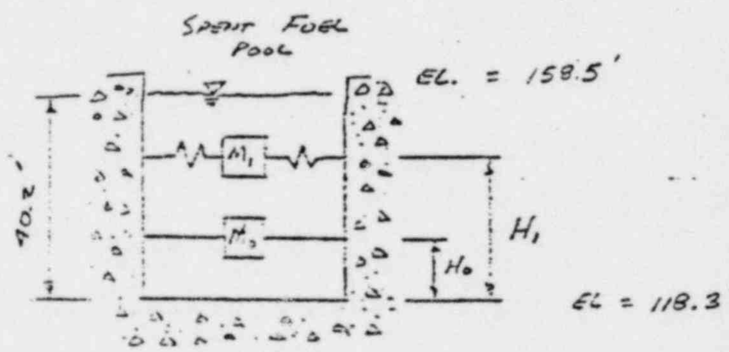


Dynamic Model for Fluid Tank Supported on the Ground

During seismic event a certain portion of the water inside the fuel storage pool will act as if it were a solid mass in rigid contact with the pool walls and it will have same maximum acceleration as that of the floor and pool walls. The seismic event also induces oscillations of the fluid

Contributing additional dynamic pressures. This mass responds as if it were a solid oscillating mass flexibly connected to the pool walls. Using formulas given in section 5.4.3 of Reference 1 and Section 6.3 of Reference 2, various parameters for the Crystal River pool will be calculated as indicated below





PLAN (Ref.)

CRYSTAL RIVER POOL DIMENSIONS

ELEVATION (Ref.)

*(CONSIDERED TO INCLUDE UPENDER REGION)

. for a rectangular tank that measures $2L$ in the direction of motion is

$$M_0 = \frac{\tanh 1.7L/H}{1.7L/H} M$$

$$M_1 = \frac{0.83 \tanh 1.6H/L}{1.6H/L} M$$

$$H_0 = 0.38H\left[1 + \alpha\left(\frac{M}{M_0} - 1\right)\right] \qquad (6.19)$$

$$H_1 = H\left[1 - 0.33\frac{M}{M_1}\left(\frac{L}{H}\right)^2 + 0.63\beta\frac{L}{H}\sqrt{0.28\left(\frac{ML}{M_1H}\right)^2 - 1}\right]$$

(Page 199 Ref. 2.)

and spring stiffness

$$K = \frac{3gM_1^2H}{ML^2}$$

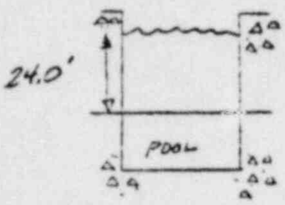
$\alpha = 1.33$; $\beta = 0.33$

FOR E-W SEISMIC EVENT

$$2L = 32.0' \qquad H = 40.2' \qquad \frac{H}{L} = \frac{40.2}{32.0/2} = 2.5125$$

$$\frac{H}{L} = 2.5125 > 1.5 \qquad \text{FICTITIOUS DATUM } H = 1.5 \times L = 1.5 \times 16 = 24.0'$$

$$H_{modified} = \underline{24.0'}$$



ELEVATION

$$\text{MASS OF LIQUID} = \frac{62.4 \times 24 \times 32 \times 24.0}{386.4 \times 1000} = 2.98 \frac{k\text{-sec}^2}{in}$$

$$M_0 = \frac{\tanh 1.7\left(\frac{16}{24}\right)}{1.7\left(\frac{16}{24}\right)}(2.98) = 2.135 \frac{k\text{-sec}^2}{in}$$

$$M_1 = \frac{0.83 \tanh 1.6\left(\frac{24}{16}\right)}{1.6\left(\frac{24}{16}\right)}(2.98) = 1.014 \frac{k\text{-sec}^2}{in}$$

$$H_0 = 0.38(24)\left[1 + \alpha\left(\frac{2.98}{2.135} - 1\right)\right] \qquad \text{WHERE } \alpha = 1.33$$

$$= 13.92$$

$$H_1 = 24\left[1 - .33\,\frac{2.98}{1.014}\left(\frac{16}{24}\right)^2 + 0.63\beta\,\frac{16}{24}\sqrt{0.28\left(\frac{2.98}{1.014} \times \frac{16}{24}\right)^2 - 1}\right]$$

$$\text{WHERE } \beta = 0.33 \qquad H_1 = 14.57'$$

SO SPRING STIFFNESS:

$$K = \frac{(3)(386.4)(1.014)^2(24 \times 12)}{(2.98)(16 \times 12)^2} = 3.125\ k/in$$

$$T = 2\pi\sqrt{\frac{M_1}{K}} = 2\pi\sqrt{\frac{1.014}{3.125}} = 3.58 \text{ SECS}$$

$$\omega = 2\pi/3.58 = 1.76 \text{ RAD/sec} \qquad f = \frac{1}{3.58} = 0.28 \text{ cps}$$

MAX. DISPLACEMENT d max

The amplitude of the height of waves set up by the vibration may be taken equal to the horizontal displacement amplitude x of mass M_1 times the factor

$$\eta = \frac{0.84KL/M_1 g}{1-(x/L)(KL/M_1 g)^2} \qquad (6.21)$$

in rectangular tanks. These expressions are satisfactory provided ηx does not exceed about $0.2R$, $0.2L$, or $.02H$. Beyond these limits nonlinear phenomena become important.

(Page 199 Reference 2)

$$d_{max}\text{ (amplitude of wave)} = \eta x = \frac{(0.84)(3.125)(16)12/(1.014)(386.4)}{1-\left(\frac{x}{16\times 12}\right)\left[\frac{(3.125)(16)12}{(1.014)\,386.4}\right]^2} x = \left[\frac{1.286}{1-0.012x}\right] x$$

NOTE: FREQUENCY OF THE OSCILLATING SPRING-MASS SYSTEM REPRESENTING THE SLOSHING WATER IS BELOW THE AVAILABLE SPECTRAS. THEREFORE A MIN. ACCELERATION OF 0.1 IS USED IN DETERMINING WATER HEIGHT.

$$X = M_1 \times \alpha G / K = \frac{1.014 \times (.1 \times 386.4)}{3.125} = \underline{\underline{12.54}} \text{ in}$$

$$\text{AMPLITUDE OF WAVE} = \left[\frac{1.286}{1-0.012(12.54)}\right] 12.54 = +19.04 \text{ in}$$

CONCLUSIONS:

$$\text{POOL LIQUID DEPTH} = 40.2' \quad \& \quad \text{RACK HEIGHT} = \frac{177.875}{12} = 14.82'$$

$$\text{DEPTH OF OSCILLATING MASS} = 40.2 \times \frac{1.014}{2.98} = 13.68'$$

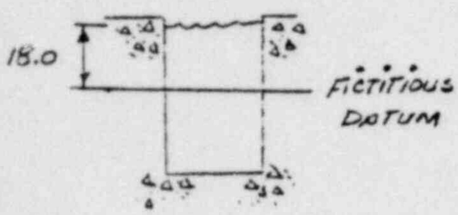
DEPTH OF CONSTRAINED WATER ABOVE RACK =

$$40.2 - 14.82' - 13.68 - \frac{19.04''}{12} = \underline{\underline{10.1}}'$$

FOR N-S SEISMIC EVENT:

$$2L = 24 \qquad H = 40.2 \qquad \text{SO} \quad \frac{H}{L} = \frac{40.2}{12} = 3.35$$

SINCE $\frac{H}{L} > 1.5$ FICTITIOUS DATUM $H = 1.5 \times 12 = 18'$ (FROM LIQUID SURFACE)



$$\text{MASS OF LIQUID} = \frac{62.4 \times 24 \times 32 \times 18}{386.4 \times 1000} = 2.23 \ \frac{K\text{-}SEC^2}{IN}$$

$$M_0 = \frac{\tanh 1.7 \left(\frac{12}{18}\right)}{1.7 \times \left(\frac{12}{18}\right)} 2.23 = 1.60 \ \frac{K\text{-}SEC^2}{IN}$$

$$M_1 = \frac{0.83 \ \tanh 1.6 \left(\frac{18}{12}\right)}{1.6 \left(\frac{18}{12}\right)} 2.23 = 0.759 \ \frac{K\text{-}SEC^2}{IN}$$

$$H_0 = 0.38 \,(18)\left[1 + 1.33\left(\frac{2.23}{1.6} - 1\right)\right] = 10.42'$$

$$H_1 = 18\left[1 - 0.33 \ \frac{2.23}{.759}\left(\frac{12}{18}\right)^2 + 0.63\,(0.33)\ \frac{12}{18}\sqrt{0.28\left(\frac{2.23 \times 12}{.759 \times 18}\right)^2 - 1}\right]$$

$$= 10.92'$$

$$\text{EQUIVALENT SPRING STIFFNESS } K = \frac{(3)(386.4)(0.759)^2(18 \times 12)}{(2.23)(12 \times 12)^2} = 3.12 \ K/IN$$

$$T = 2\pi\sqrt{\frac{M_1}{K}} = 2\pi\sqrt{\frac{.759}{3.12}} = 3.1 \ SEC$$

$$\omega = 2\pi/T = 2\pi/3.1 = 2.03 \ RAD/SEC$$

$$f = \frac{1}{3.1} = 0.32 \ cps.$$

MAX DEFLECTION OF WAVE:

$$d_{(MAX)} = \frac{0.84\,(3.12)\,(12\times12)/(0.759)\,386.4}{1-\left(\frac{x}{12\times12}\right)\left[\frac{(3.12)(12\times12)}{(0.759)\,386.4}\right]^2}\,x = \left[\frac{1.297}{1-0.0163x}\right]x$$

ASSUMING A MINIMUM SPECTRA ACCELERATION VALUE = .1

$$x = \frac{M_1 x\,(.1)G}{K} = \frac{.759\times(.1)(386.4)}{3.12} = 9.4$$

MAX AMPLITUDE OF WAVE $= \left[\frac{1.297}{1-0.0163(9.4)}\right]9.4 = 14.29''$ OR $1.19'$

THEREFORE: CONSTRAINT COVER OVER RACKS

DEPTH OF OSCILLATING MASS $= 40.2\times\frac{.759}{2.23} = 13.68'$

CONSTRAINED WATER ABOVE TOP OF RACK:

$= 40.2 - 13.68' - 14.82 - 1.19 = 10.51'$

SINCE THE SLOSHING WATER MASS IS A MINIMUM OF 10.1' FEET ABOVE THE TOP OF THE SPENT FUEL RACKS, ANY DYNAMIC EFFECTS WILL BE MINIMAL.

APPENDIX D

IMPACT FACTOR CALCULATION ANALYSIS

THE RELATIVELY LARGE GAP EXISTING BETWEEN A FUEL ASSEMBLY AND ITS STORAGE CELL ($\approx$ 0.25" ALL AROUND) WILL RESULT IN A RATTING OF THE FUEL ASSEMBLY DURING A SEISMIC EVENT.

THE LOADS PRODUCED BY THE RATTLING FUEL ASSEMBLIES INCLUDE AN IMPACT BECAUSE OF THE GAPS.

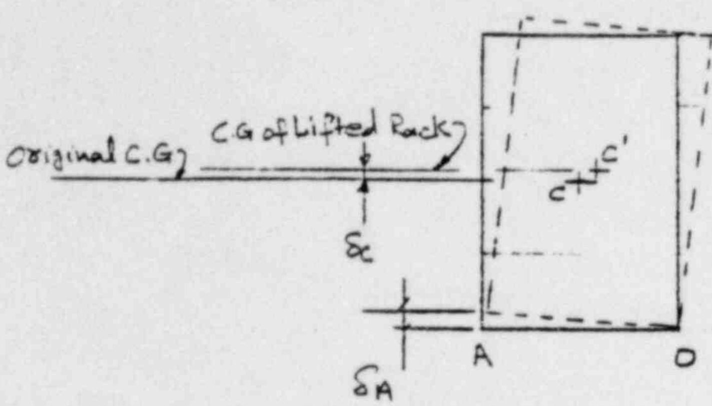
THESE IMPACT EFFECTS ARE ACCOUNTED FOR BY APPLYING A FACTOR OF 2 TO THE IMPACTING MASSES. AN EFFECTIVE IMPACT FACTOR FOR THE ENTIRE STRUCTURE IS THEN CALCULATED BY APPLYING THE FACTOR OF 2.0 TO THE IMPACTING MASSES OF THE FUEL ASSEMBLY, ADDING THE REMAINING WEIGHT INCLUDING WATER AND DIVIDING BY TOTAL MASS. THIS EFFECTIVE IMPACT FACTOR IS THEN APPLIED TO THE HORIZONTAL SEISMIC EVENT PRODUCING THE WORST STRESSES AND COMBINED BY SRSS METHOD TO REFLECT THE RESULTS OF THE MOST CRITICAL SITUATION.

$$I.F = \frac{(2)\,1700 + 2410 - 1700}{2410} = 1.704$$

NOTE: ASSUMING IMPACT OF ENTIRE FUEL ASSEMBLY (ASSEMBLY NOT CAPTURED)

APPENDIX E

STABILITY ANALYSIS



Level 1 $W_1 = 36 \times 2 \times 187.5$ lbs.

Level 2 $W_2 = 36 \times 2 \times 375$ lbs.

Level 3 $W_3 = 36 \times 2 \times 375$ lbs.

Level 4 $W_4 = 36 \times 2 \times 375$ lbs.

Level 5 $W_5 = 36 \times 2 \times 187.5$ lbs.

From Computer Output S-2, the maximum Overturning moment due to SSE seismic event in XZ (E-W) direction

$= 36 \times 3.328 \times 2 \times 10.5 = 2516$ K.in. (without Impact)

Maximum Stabilizing Moment $= 36 \times 2.13 (1 - \frac{2}{3} \times 0.1) \times \frac{63}{2} = 2254$ K.in < 2516 K.in.

∴ Due to Lateral seismic inertia loads, the fuel storage rack will tip over about point O, the original center of gravity C of the storage rack will be lifted up by an amount δ_C to C'

Max. Kinetic energy of the Rack due to Seismic event $E_e = \sum_1^5 \frac{1}{2} m_i v_i^2$

From Computer Output S-2, the maximum lateral velocities at different levels due to SSE seismic event in XZ (E-W) direction

At Level 1 $V_1 = 6.759$ in/sec. ; At level 2 $V_2 = 4.446$ in/sec

$V_3 = 2.333$ in./sec. ; $V_4 = 0.76$ in./sec. $V_5 = 0.22$ in./sec.

∴ Max. Kinetic energy of the Rack due to SSE seismic event

$$E_e = \frac{1}{2} \times \frac{36 \times 2 \times 187.5}{386.4} \left\{ 6.759^2 + 2 \times 4.446^2 + 2 \times 2.333^2 + 2 \times .76^2 + 0.22^2 \right\}$$

$= 1699.9$ in. lbs. $= 1.6999$ in. K. (without Impact)

$= 1.704 \times 1.6999 = 2.8966$ in. K. (with Impact)

Potential Energy Required to raise the storage rack CG by an amount $\delta_C = E_i = 36 \times 2.13 (1 - \frac{2}{3} \times .1)\, \delta_C$ in. K

$= 71.568\, \delta_C$ in. K

$\therefore$ Using Energy Balance $E_e = E_i$

$$1.704 \times 1.6999 = 71.568\, \delta_c$$

$$\therefore \delta_c = 1.704 \times 0.02375 = 0.0405 \text{ in.}$$

$\therefore$ Leg A of the rack will be lifted up by $\delta_a = 2\delta_c = 0.081$ in.

Since the C.G of the tilted rack due to seismic event will remain within AO, the rack structure will not overturn.

During an Operating Basis Earthquake (OBE) event, maximum overturning moment $= 36 \times \frac{3.323}{2} \times 2 \times 10.5 = 1258$ K.in (without Impact)

$= 1.704 \times 1258 = 2143$ K.in (with Impact)

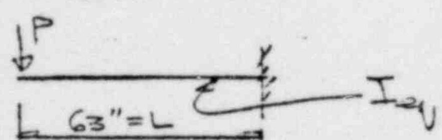
Maximum Stabilizing Moment $= 36 \times 2.13 \left(1 - \frac{2}{3} \times \frac{0.1}{2}\right) \frac{63}{2}$

$= 2334.91$ K.in > 2143 K.in

$\therefore$ The rack will not overturn or even lift up during an OBE event.

Analysis for Impact Loads and Stresses Resulting from Recontact of the Rack to the Floor.

During lift up and recontact, assume that the rack base structure acts as a cantilever beam.

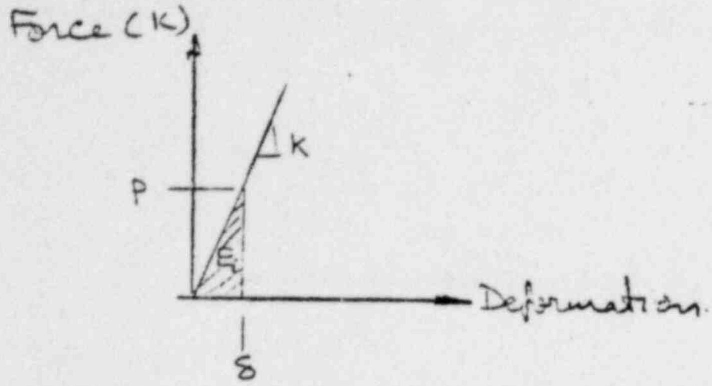


Equivalent Moment of Inertia of the Rack Base Structure

$$I_{eq} = 5 \times 55.55 + 2 \times 48.54 = 374.83 \text{ in.}^4$$

$\therefore$ Stiffness of the Cantilever Beam $= \frac{3EI_{eq}}{L^3} = \frac{3 \times 29000 \times 374.83}{(63)^3}$

$$K = 125.92 \text{ K/in.}$$



$P = K\delta$

Internal strain Energy $E_i = \frac{P\delta}{2} = \frac{PP}{2K}$

External Kinetic Energy of Impact during recontact is same as the external kinetic energy developed during the SSE Seismic event = 2.3966 in.K.

Using Energy Balance

$$\frac{PP}{2K} = 2.3966$$

$$\therefore P = \sqrt{2 \times 125.92 \times 2.3966} = 27.01\ K.$$

$\therefore$ Max. Impact Load generated at the time of Recontact

$= 27.01\ K$

Max. stress in the rack base structure due to impact load $= \frac{27.01 \times 63}{374.83} \times 4.0 = 18.16$ Ksi

Max. Reaction load on each support pads along rack face A

$= \frac{27.01}{3} = 9.0\ K$

Max. axial stress in the 1½" Jack Screw

$= \frac{9.0}{1.41} = 6.38$ Ksi.

Max. Concrete bearing stress $= \frac{9.0 \times 4}{\pi \times 8^2} = 0.179$ Ksi.

Max. bending stress in the bearing plate

$$= 0.179 \left(\frac{8-4.0}{2}\right)^2 \times \frac{1}{2} \times \frac{6}{1(1.0)^2}$$

$$= 2.15 \text{ Ksi}$$

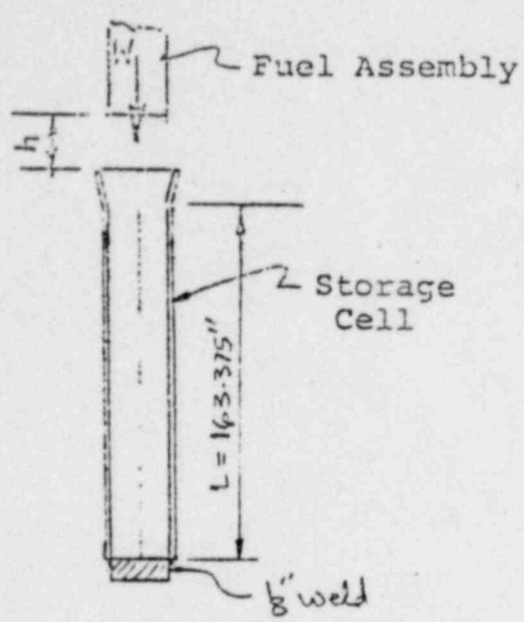
:- Conclusions:

1. During an OBE seismic event the storage rack will not lift up and will not overturn.
2. During an SSE seismic event the storage rack feets could lift up as much as 0.081 inches however the rack will not tip over
3. During recontact of the rack with the floor the maximum impact load generated will be of the order of 27K. The resulting stresses in the rack base structure, 1½" Jack Screws, 8"ϕ base plates are of the order of 18.16 Ksi, 6.38 Ksi and 2.15 Ksi respectively. The maximum concrete bearing stresses would be 0.179 Ksi.

APPENDIX F

FUEL ASSEMBLY DROP ON SPENT FUEL STORAGE CELL

Case 1



Reference (1) NES 81A0426 "Final Structural Design of a Fuel Storage Well Crash Pad for the LACBWR Nuclear Power Plant", Nuclear Energy Services, Inc. Danbury, Connecticut October 5, 1975.

Weight of Fuel Assembly W = (1550+150) lbs
= 1700 lbs ✓

Maximum drop height h = 34 in ✓

Conservatively neglect the energy losses in the local deformation of the flare at top of the Fuel Storage Cell. The local deformation of the flare at top of the cell will reduce damage to the cell (therefore less damage to the stored fuel) and will also reduce damage to the liner plate and reinforced concrete floor under the storage cell.

Assume Uniform Storage Cell Compression

External Kinetic Energy of the Fuel assembly

$= 1.700 \times 34 \qquad = 57.80$ in.k ✓

Internal Strain Energy of the storage cells

$$E_i = \frac{128.5}{1.166} \varepsilon_x^{1.166} ALN$$

N = number of cells effective in absorbing the energy : 1

A = Cross sectional area of cell = (4×2.215×.125+3×3.6075×.06) 0.95
including 4 angles
= 4477 in^2

ε_x = strain in storage cell

L = Length of Cell = 163.375 in

Equating External and Internal Energy

$$\frac{128.5}{1.166} \varepsilon_x^{1.166} ALN = 57.80$$

Strain in Storage Cell

$$\varepsilon_x = \left(\frac{1.166\ E_i}{128.5\ ALN}\right)^{1/1.166} = \left(\frac{1.166\ (57.80)}{128.5(4.977)(163.375)(1)}\right)^{1/1.166}$$

$= 0.001836$ in/in

Percent of Ultimate Strain

$$\%\varepsilon_u = \frac{\varepsilon_x}{\varepsilon_u} \times 100 = \frac{0.001836}{0.485} \times 100 = 0.378\%$$

Maximum deformation of the cell = δ_x

$\delta_x = \varepsilon_x L = 0.001836 \times 163.375 = 0.30$ in.

Maximum Stress in the Cell

$$\sigma_x = 128.5\ \varepsilon_x^{\ 0.166} = 128.5(0.001836)^{0.166} = 45.15 \text{ ksi}$$

Dynamic yield stress for stainless steel using 38% increase over static yield (Ref. 1)
$= 1.38 \times 30 = 41.4$ Ksi.

Maximum Transmitted Reaction Load

$R_x = A\sigma_x = 4.977 \times 45.15 = 224.73$ kips.

The reaction load will be transmitted through the four cell walls and for corner angles to the base plate to the rack base structure to the pool floor through the rack base feet.

Maximum load transmitted by the two 0.06" walls of the storage cell $= 8 \times 8.6075 \times 0.06 \times 45.15 = 186.54$ K.

Maximum stress in the 1/8" weld between the cell wall and cell base plate $= \dfrac{186.54}{4 \times 10.0 \times .707 \times 0.125} = 52.77$ Ksi

$> 1.6 \times 21.0 \times \dfrac{41.4}{36}^{*} = 38.6$ Ksi < 52.77 Ksi ($\sigma_{yield} = 36$ Ksi)

21 Ksi is allowable stress in weld for A-36 steel as per AISC Code
41.40 Ksi is dynamic yield stress value for stainless steel
and factor 1.6 is increase in allowable stress values permitted by USNRC standard review plan 3.8.4 (Reference 3)

∴ The analysis indicate that the weld between the cell and base plate yield.

* SEE PAGE 10/12 FOR DYNAMIC YIELD STRESS CALCULATION.

Maximum energy absorbed in shearing the weld

$$E_w = \tau_u \times \varepsilon_u \times \text{weld volume}$$

where τ_u = Ultimate shear stress for stainless steel

$= \text{Ultimate tensile stress}/2 = \dfrac{128.5(0.485)^{0.166}}{2}$

$= 57.0$ Ksi.

ε = ultimate strain for stainless steel $= 0.485$ in/in

$\therefore E_w = 57 \times 0.485 \times 4 \times 10 \times 0.707 \times 0.125 = 97.73$ in.K > 57.8 in.K

$\therefore$ The weld will absorb some energy in partially shearing but it will not completely shear off.

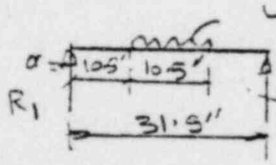
Maximum Stress in the Rack Base Structure.

Load transmitted by each wall of the storage cell to the rack structure underneath it $= \dfrac{224.73}{4}$, this load will act as a uniform load over 10.5 in length of the base structure beam (Span $= \dfrac{63.0}{2} = 31.5''$)

$$w = \frac{224.73}{4 \times 10.5} = 5.351 \text{ K/in.}$$

$R_1 = \dfrac{224.73}{4 \times 2}$

$a = 10.5''$



8" wide flange beam
Section modulus $S = 13.89$ in.^{3}

Max. moment $= R_1\left(a + \dfrac{R_1}{2w}\right) = \dfrac{224.73}{4 \times 2}\left(10.5 + \dfrac{224.73}{4 \times 2 \times 2 \times 5.351}\right)$

$= 368.69$ K.in.

max. bending stress $= \dfrac{368.69}{13.89} = 26.54$ Ksi. < 41.4 Ksi.

Storage Rack Feet :

In order to evaluate the the effects on the rack feet and pool liner plate, conservatively assume that the fuel assembly drops on top of the storage cell walls directly above the rack feet. Since the maximum reaction load will be distributed to various rack feet through the rigid rack base structure, conservatively assume that 70% of the total load is transmitted to one rack feet.

Max. Compressive stress in $1\frac{1}{2}$" dia. Jack Screws

$$= \frac{0.7 \times 224.73}{1.41} = 111.6 \text{ Ksi.}$$

< 180.0 Ksi. yield stress for 17-4 PH Stainless steel.

Max. punching stress in the 8"Ø bearing plate $= \dfrac{0.7 \times 224.73}{\pi(4.0+1.0)1.0} = 10.01$ Ksi. < 41.4 K

Max. Punching stress in the liner plate $= \dfrac{0.7 \times 224.73}{\pi(3.0+0.1875)0.1875} = 32.6$ Ksi < 41.4 Ksi.

Because of the presence of concrete underneath, the liner plate will not be punch

Max. Bearing Stress on Concrete Floor $= \dfrac{0.7 \times 224.73 \times 4}{\pi(3+2\times 0.1875)^2} = 2.86$ Ksi. < 3.57 Ksi.

Max. bending stress in the Bearing Plate $= 2.86\left(\dfrac{8-4.0}{2}\right)^2 \times \dfrac{1}{2} \times \dfrac{6}{1\times 1.0^2}$

$= 34.32$ Ksi > 30 Ksi < 41.4 Ksi.

∴ The Bearing Plate will be deformed. This deformation of the bearing plate may reduce the bearing area with the floor thereby causing local crumbling of concrete under the Jack Screw feet.

Design Check for Maximum cell strain (Assuming a Minimum Stress Increase Due to Impact of 20%)

Strain Energy Capacity of Intermediate Cylinders

$$E_i = \frac{116.9}{1.2} \varepsilon_x^{1.20} \text{ ALN}$$

Equating External and Internal Energy and Rearranging - Maximum Strain is

$$\varepsilon_x = \left[\frac{1.2\ E_i}{116.9\ \text{ALN}}\right]^{1/1.20} = \left[\frac{1.2 \times 57.80}{116.9 \times 4.977(163.375)(1)}\right]^{1/1.20}$$

$$= 0.00243 \text{ in/in}$$

* Allowable Bearing Stress, Ultimate Strength Design Per ACI 318-71 para. 10.14 $= \sqrt{\dfrac{A_2}{A_1}} \times .85\phi f'c = 2 \times .85 \times .7 \times 3.0 = 3.57$ ksi

Per Cent of Ultimate Strain

$$\% = \frac{\varepsilon_x}{\varepsilon_u} \times 100 = \frac{0.00243}{0.485} \times 100 = 0.501\%$$

Maximum deformation of the cell

$$\delta_x = \varepsilon_x L = 0.00243 \times 163.375 = 0.397 \text{ in.}$$

CONCLUSIONS:

(1) Drop of fuel assembly on the fuel storage cell will cause the following damage:

a. There will be local deformation at the top of the fuel storage cells.

b. There will be shearing of the weld between the storage cell walls and cell base plate

c. There will be local crumbling of concrete under the rack support feet

d. The 1.0" support feet base plate will be deformed.

(2) There will be no buckling or collapse of the fuel storage cell so that significant changes in k_{eff} will not occur.

(3) The liner plate will not be perforated

(4) There will not be any leakage.

Case 2

FUEL ASSEMBLY DROP AND SUBSEQUENT TIPPING ANALYSIS

After the drop of the fuel assembly on to the storage cell, the fuel assembly will tip over and fall on several storage cells.

Conservatively assuming that the fuel assembly falls diagonally across the spent fuel storage racks so that it will impact least no. of storage cells

Length of the fuel assembly L = 163.375"

Diagonal Center to Center distance between storage cells

$= \sqrt{(10.5)^2 + (10.5)^2} = 14.849$ in.

∴ Minimum number of Cells that can be impacted by the fuel assembly $= \frac{163.375}{14.849} \approx 11.$

Total Kinetic energy of the fuel assembly acquired during tipping $= 1.70 \times \frac{163.375}{2} = 138.87$ in.K

Average Kinetic energy per storage Cell $= \frac{138.87}{11} = 12.62$ in.K

Assuming that $\frac{1}{11}$ th of the fuel assembly mass drops from a height of $\left(163.375 - \frac{163.375}{2 \times 11}\right)$ in. $= 155.95''$, maximum external Kinetic energy $= \frac{1.7}{11}(155.94) = 24.1$ in.K.

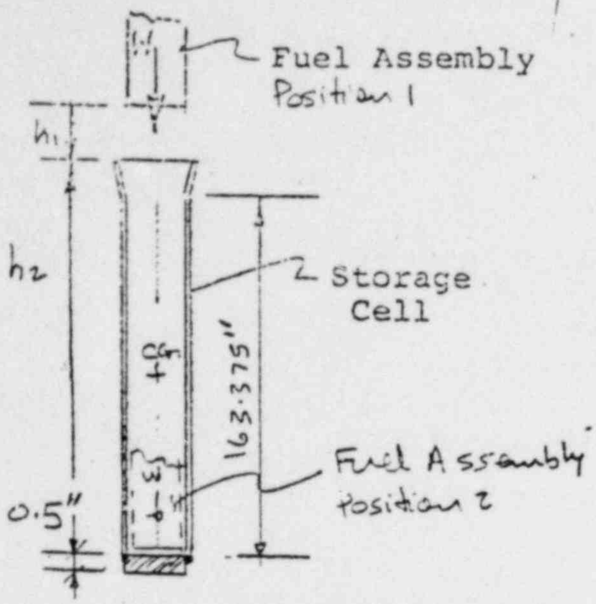
This external kinetic energy per storage cell is much less than the Kinetic energy (57.8 in.K) associated with the vertical drop of fuel assembly on top of the storage cell. Therefore the consequences of Tipping of the fuel assembly on the storage racks will be less sever than those analyzed above for the vertical drop of fuel assembly.

Case 3.

FUEL ASSEMBLY DROP THROUGH THE STORAGE CELL AND IMPACT AT THE STORAGE CELL BASE PLATE LOCATION

Reference 1: Topical Report BC-TOP-9A Rev. 2 "Design of Structures for Missile Impact" Bechtel Power Corp., San Francisco, California.

If a fuel assembly falls straight through the storage cell, it will impact the Cell base plate.



Conservatively neglect the drag resistance as the fuel assembly travels through the storage cell.

$h_1 = 34.0$ in.

$h_2 = 164$ in

Maximum kinetic energy of impact E_e $= 1.70 \times (164 + 34) = 336.6$ in.k

$= 28050.0$ ft lbs

Maximum velocity of impact V_s $= \sqrt{2 \times 32.17 \times \frac{(164+34)}{12}} = 32.58$ ft/sec

Assume the fuel assembly as a missile striking the stainless steel base plate. The analysis for the plate thickness that could be perforated by a missile can be performed using Ballistic Research Lab. (BRL) equation given in Reference 1.

The BRL Formula is shown below, modified by setting a material constant K = 1 and solving directly for steel plate thickness, T, which will just be perforated by the missile.

$$T = \frac{\left(\frac{MV_s^2}{2}\right)^{2/3}}{672D} \qquad (2\text{-}7)$$

where:

T = Steel plate thickness to just perforate (inches)

M = Mass of the Missile (lb sec^2/ft)

$= \frac{1700}{32.17} = 52.844$

A = Frontal Area of Fuel Assembly = $8 \times 1.75 \times 0.5 = 7$ in.^{2}

W = Weight of the Missile (lb) = 1700

Vs = Striking Velocity of the Missile Normal to Target Surface (ft/sec) = 32.58

D = Diameter of the Missile (in) $= \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4 \times 7}{\pi}}$
$= 2.985$ in

$$T = \frac{\left\{\frac{52.844}{2}(32.58)^2\right\}^{2/3}}{672 \times 2.985} = 0.460 \text{ in}$$

The thickness, t_p, of a steel barrier required to prevent perforation should exceed the thickness for threshold of perforations. It is recommended to increase the thickness, T, by 25 percent to prevent perforation.

$$t_p = 1.25T \qquad (2\text{-}8)$$

$$t_p = 1.25 \times 0.460 = 0.575 \text{ in} > 0.50 \text{ in}$$

Stanford Research Institute (Ref. 1)

$$\frac{E}{D} = \frac{S}{46,000}\left(16,000\,T^2 + 1,500\,\frac{W}{W_s}\,T\right)$$

T = steel thickness to be just perforated (in)

D = diameter of the missile (in) = 2.985 in

E = critical kinetic energy required for perforation (ft-lb), = 28050.0 (page 6 of 10)

S = ultimate tensile strength of the target minus the tensile stress in the steel (psi) = 114000.0

W = length of a square side between rigid supports (in) = 4 in

W_s = length of a standard width (4 in). (See Ref. 1) = 4 in

$$\therefore \frac{28050.0}{2.985} = \frac{114000}{46,000}\left(16000\,T^2 + 1500.0\,T \times 1\right)$$

$$\text{or } T^2 + 0.09375T - 0.23956 = 0$$

$$\therefore T = \frac{-0.09375 \pm \sqrt{(0.09375)^2 + 4 \times 0.23956}}{2} = 0.4448 \text{ in.}$$

$\therefore$ Required thickness of plate to prevent perforation

$= 1.25T = 1.25 \times 0.4448 = 0.5560 \text{ in} > 0.5 \text{ in.}$

Since the storage cell base plate has 5" diameter hole, its effective thickness to resist puncturing will be less than 0.5 in.

Equating the volumes of plate with and without holes, the effective thickness of the plate could be estimated as

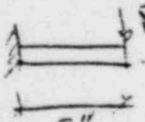
$$t_{eff.}(8.9375)^2 = 0.5\left\{8.9375^2 - \frac{\pi}{4}(5)^2\right\}$$

$$\therefore t_{eff} = 0.377 \text{ in} < 0.46 \text{ or } 0.445 \text{ in}$$

Therefore the analysis indicates that the cell base plate will be perforated, however the presence of the flange member of the rack base structure will prevent further penetration of the fuel assembly.

Maximum deformations of the impacted base plate and flange members can be estimated using energy balance techniques.

Load deformation characteristics of the of the 1/2" Base plate



$t = 0.5"$; $b = 10.5"$

Elastic section modulus $S = \frac{bt^2}{6} = \frac{10.5(0.5)^2}{6} = 0.4375 \text{ in.}^3$

Plastic " " $Z = \frac{bt^2}{4} = \frac{10.5(0.5)^2}{4} = 0.65625 \text{ in.}^3$

Dynamic yield stress for stainless steel (increase static yield by 38%, Ref. 1) $= 1.38 \times 30 = 41.4$ Ksi $= \sigma_y$

Stress at 50% ultimate strain $(.5 \times .485 \text{ in/in.})$ $= 128.5(0.5 \times 0.485)^{0.166}$

$\sigma_{.5\epsilon_u} = 83.1$ Ksi.

$\therefore$ Plastic moment capacity $M_p = \sigma_y Z_p$

$= 41.4 \times 0.65625 = 27.2$ K in.

$\therefore$ Load Carrying capacity at $M_p = P_y = \frac{M_p}{2.0} = \frac{27.2}{2} = 13.58$ K.

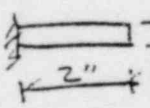
Moment at 50% ultimate strain $= M_{.5\epsilon_u} = S\left[\left(\frac{Z_p}{S} - 1\right)\sigma_y + \sigma_{.5\epsilon_u}\right]$

$= 0.4375\left[\left(\frac{.65625}{.4375} - 1\right) 41.4 + 83.1\right]$

$= 45.41$ K.in.

Load at " " " $P_{.5\epsilon_u} = \frac{45.41}{2} = 22.71$ K.

Flange plate of wide flange beam members:



$t = 0.375"$; $b = 10.5"$

$S = 10.5\frac{(.375)^2}{6} = 0.2461$ in.^{3}

$Z_p = 10.5\frac{(.375)^2}{4} = 0.369$ in.^{3}

$\therefore M_p = 41.4 \times 0.369 = 15.28$ K in.

$P_Y = \frac{15.28}{2} = 7.64$ K.

$M_{.5\epsilon_u} = 0.2461\left[\left(\frac{0.369}{0.2461} - 1\right) 41.4 + 83.1\right] = 25.55$ K in.

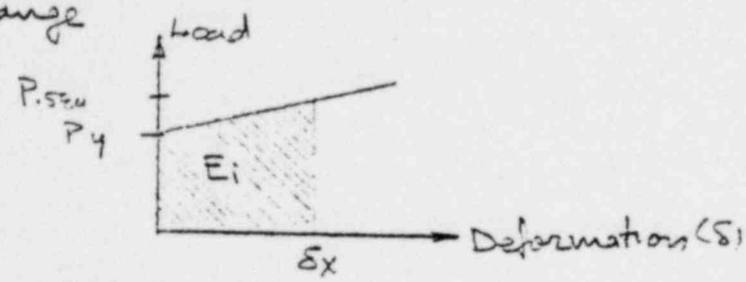
$\therefore P_{.5\epsilon_u} = \frac{25.55}{2} = 12.77$ K.

∴ Maximum Load carrying Capacity of 4 sides of the base plate and flange plate at yield = $P_Y = 4(13.58 + 7.64) = 84.88$ K

Maximum Load carrying capacity of 4 sides of the base plate and flange plate at 50% ultimate strain

$$P_{.5\varepsilon u} = 4(22.71 + 12.77) = 141.92 \text{ K}$$

Conservatively assuming linear variation of stress-strain relation in the plastic range



Load Deformation Characteristics

Internal strain Energy $E_i = (P_{.5\varepsilon u} + P_Y)\frac{1}{2}\delta_X$
$= (141.92 + 84.6)\frac{1}{2}\delta_X$ in.K.

External kinetic energy of fuel assembly drop $E_e = 336.7$ in.K

Conservatively assuming 30% loss in external energy due to drag, buoyancy, piston effects and in local deformations of fuel assembly, storage cell etc.

External energy to be absorbed by the base plate and flange plate
$= 0.7 \times 336.7 = 235.69$ in.K

∴ Using energy balance
$$(141.92 + 84.6) \times \frac{1}{2}\delta_X = 235.69$$

$$\therefore \delta_X = 2.08 \text{ in}$$

∴ The base plate and flange plate will deform 2.08 in.

The maximum transmitted reaction load $= P_{.5\varepsilon u} = 141.92$ K.

This reaction load is less than $0.7 \times 224.73 = 157.31$ K (Page 4) for the straight fuel assembly drop on top of the storage cell. Therefore the overall damage to the rack base structure, rack feet and the pool liner plate will be less severe for Case 3 than that for Case 1.

Conclusions:

(1) Drop of the fuel assembly through the storage cell and impact on the rack base structure cause the following.

a. The storage cell base plate and the flange member of the rack base structure will undergo significant local deformations

b. The cell base plate will be perforated

c. The maximum transmitted reaction load 141.92K is less than 0.7x 224.75 = 157.31 K for the straight fuel assembly drop on top of the storage cell therefore, the resulting overall damage to the rack base structure, rack feet and the pool liner plate will be less severe.

(2) There will be no buckling or collapse of the fuel storage cell or the rack structure so that significant changes in K_{eff} will not occur

3) The pool liner plate will not be perforated so that there will not be any leakage.

APPENDIX G

SEISMIC WALL BRACING

DESIGN / ANALYSIS

OBJECTIVE: DESIGN SEISMIC BRACING TO ADEQUATELY WITHSTAND LATERAL LOADS PRODUCED DURING A SEISMIC EVENT.

LOADING: FROM NES COMPUTER BINDER NO. 527, RUNS NO. 52700-W4 & 52700-Q7, SEISMIC WALL LOADS ARE OBTAINED WHICH HAVE BEEN DEVELOPED DUE TO THE SSE ACCELERATION OF A FULLY-LOADED-RACK. THIS LATERAL LOADS WERE CONCEIVED FROM THE WORST RESULTS OBTAINED CONSIDERING AN IMPACT FACTOR OF 1.704 APPLIED IN EITHER THE X1 OR X2 DIRECTIONS. THE SEISMIC RESULTS ARE THEN COMBINED BY AN SRSS METHOD AND ADDED DIRECTLY WITH DEAD WEIGHT RESULTS.

THE OBE SEISMIC EVENTS HAVE SPECTRUM ACCELERATION VALUES EQUAL TO HALF THOSE FOR AN SSE EVENT. THE WALL LOADS DUE TO AN OBE EVENT ARE AVAILABLE IN POST RUNS 52700-90.

THE SEISMIC BRACING IS DESIGNED USING THE LARGEST REACTION LOADS OF THE SEISMIC BRACING.

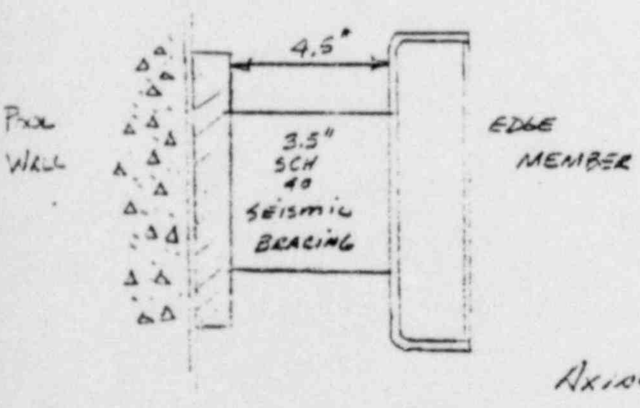
SEISMIC BRACING (N-S DIRECTION) POOL A

LOADING FROM POST RUNS (52700-Q7 & 52700-W4) FOR SSE AND RUN NO. 52700-90 FOR OBE EVENT, LARGEST SEISMIC BRACING LOADS (X1 DIRECTION) OCCUR WHEN IMPACT IS APPLIED TO X1 DIRECTION.

MAX LOAD = 42.73^k (SSE) & 21.4^k (OBE). THESE AXIAL LOAD IS OBTAINED FROM A HYPOTHETICAL SITUATION WHERE THE SPENT A FUEL RACK IS BEING LOADED BY A TOTAL OF 26 CELLS. THE ACTUAL SEISMIC LOADS IN POOL A WILL RESULT FROM THE ACCELERATION OF ONLY 16 CELLS. THEREFORE:

DESIGN LOAD $= \frac{16}{26} \times 42.73^k = 26.33$ KIPS (SSE)

$\frac{16}{26} \times 21.4^k = 13.17$ KIPS (OBE)



$A_{REQ} = \frac{26.33^k}{1.9 S_y} = \frac{26.33}{27.0} = 0.975 in^2$ (SSE)

OR $13.17/.6F_y = 13.17/19 = 0.731 in^2$ (SS

USING 3½" SCH 40 PIPE AREA = 2.68 in^2

AXIAL FREQUENCY $= 3.13\sqrt{\frac{K}{W}}$ (Ref. 10)

$W = 27.4^k$ $K = \frac{AE}{L} = \frac{(2.68)(29000)}{4.5} = 16676$ K/in

$f = 3.13\sqrt{\frac{16676}{27.4}} = 77.2$ CPS

SINCE SEISMIC BRACING FREQ. IS EXCEEDINGLY HIGH, ASSUME RIGID FOR RACK ANALYSIS AND WILL HAVE LITTLE EFFECT ON RACK FREQ.

CONCRETE BEARING PLATE ANALYSIS (SSE EVENT) POOL A

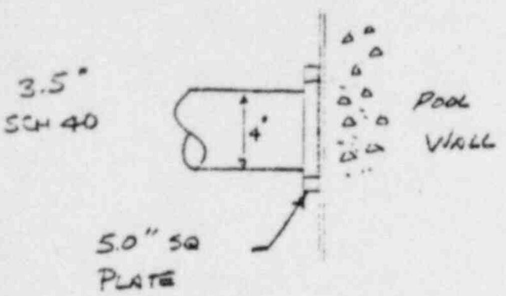
DESIGN LOAD = 26.33 KIPS

F_p (CONCRETE BEARING STRESS ALLOWABLE) $= (3)(.85)(.7) = 1.785$ ksi (Ref. 9)

BEARING PLATE AREA REQ = $\frac{26.33}{1.785} = 14.75 \text{ in}^2$

USING SQUARE PLATE OF 5.0" WITH .5" BEVELED CORNERS:

$A_{PLATE} = (5.0)^2 - \frac{(.5)^2}{2} \times 4 = 24.5 \text{ in}^2$



PIPE PROPERTIES

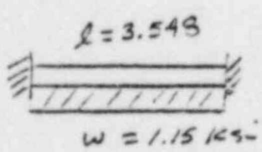
$a = 3.548$"

WALL THICKNESS = .226"

$b = \sqrt{\left(\frac{5.0}{2}\right)^2 + \left(\frac{5.0}{2}\right)^2} - \left(\frac{3.548}{2} + .226\right)$

$= 1.54$ in

ASSUMING A 1" STRIP TAKEN FROM THE INSIDE DIAMETER OF PI[PE]



w = CONCRETE BEARING STRESS = $\frac{26.33}{24.5} = 1.075$ ksi

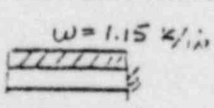
$M_{(FIXED\ ENDS)} = \frac{wl^2}{12} = \frac{(1.075)(3.548)^2}{12} = 1.13$ K-in

REQUIRED PLATE THICKNESS

$S_{REQ'D} = \frac{M}{f_b} = \frac{1.13 \text{ K-in}}{27 \text{ ksi}} = 0.0418 \text{ in}^3$

$S_{REQ'D} = \frac{t^2}{6}$ (ASSUMING 1" SECTION) $= 0.0418 \text{ in}^3$ so $t = \sqrt{.251} = .50$ in

CONSIDERING THE CANTILEVERED PLATE EDGE:



$M_{max} = \frac{(1.075)(1.54)^2}{2} = 1.275$ K-in $\quad S_{REQ'D} = \frac{1.275}{27} = 0.04$

$\frac{t^2}{6}$ (ASSUMING 1" SECTION) $= 0.0472 \text{ in}^3 \therefore t = \sqrt{.283} = .532$"

USE 0.625" × 5" × 5" BEARING PLATE (½" BEVELED EDGES)

ACTUAL PLATE STRESS = $1.275 / (.625)^2 \times 6 = 19.58$ ksi < 27.0 ksi

CONCRETE BEARING PLATE ANALYSIS (OBE EVENT) - Pool A

DESIGN LOAD = 13.2^{K} $F_p = (.35)(3 ksi) = 1.05$ ksi

BEARING PLATE AREA REQ. $\frac{13.2}{1.05} = 12.57\ in^2$

USING A 5" SQUARE PLATE WITH 1/2" BEVELED EDGES:

$A_{PLATE} = 24.5\ in^2$ SO CONCRETE BEARING STRESS $= \frac{13.2^K}{24.5} = 0.539$ ksi

BEARING PLATE STRESS: FROM PREVIOUS PAGE – BASE PLATE THICKNESS = (.625")

CONTROLLING $M = \frac{(.539)(1.54)^2}{2} = 0.64$ K-IN (FLANGE OF PLATE)

ASSUMING 1" STRIP: SECTION MODULUS $= \frac{(.625)^2}{6} = 0.065\ in^3$

PLATE STRESS $= \frac{0.64}{0.065} = 9.83$ ksi < 18.0 ksi ok

SEISMIC BRACING COMPRESSIVE STRESS ANALYSIS — POOL A

PIPE: 3½ SCH 40 ST 40S

PROPERTIES: $A_{pipe} = 3.17\ in^2$, $r = 1.337$ in, $l = 4.5$ in,

$\frac{Kl}{r} = \frac{(1)4.5}{1.337} = 3.37$

(FROM REF. #5) $C_c = \sqrt{\frac{2\ \pi^2\ 28000}{30}} = 135.7$, $\frac{Kl/r}{C_c} = \frac{3.35}{135.7} = 0.025$

SSE $F_a = \frac{1.6\ [1 - .5(.0248)^2]\ 30}{\frac{5}{3} + \frac{3}{8}(0.0248) - \frac{1}{8}(0.0248)^3} = 28.63$ ksi

$f_a = 26.33/2.68 = 9.83$ ksi $<$ 28.63 ksi ok (SSE)

FOR OBE SEISMIC EVENT:

F_a (ALLOWABLE) $= \dfrac{28.63 \text{ ksi}}{1.6} = 17.90$ ksi

$f_a = 13.17/2.68 = 4.91$ ksi $<$ 17.90 ksi ok

SEISMIC BRACING WELD ANALYSIS: N-S DIRECTION - POOL A

THE WELDS ARE REQUIRED ONLY TO MAINTAIN THE DEAD LOADS OF THE SEISMIC BRACING PLUS A VERTICAL EARTHQUAKE ACCELERATION.

BRACING FREQ. (ASSUMING CANTILEVER OF $\ell = 4.5''$)

$$= f = \frac{3.13}{\sqrt{\dfrac{(W + 0.236 W\ell)\ell^3}{3EI}}}$$

WHERE W = BEARING PLATE
$W\ell$ = WEIGHT OF BRACING

BRACING WEIGHT $= \frac{5.0}{12} \times 9.11 = 4^{\#}$

Plate WEIGHT $= 24.5 \times .625 \times .284 = 4.4^{\#}$

$I = 4.79 \text{ in}^4$

$$f = \frac{3.13}{\sqrt{\dfrac{[4.4 + (.236)(4)](5.0)^3}{(3)(28000)(4.79)}}} = 76.82 \text{ cps}$$

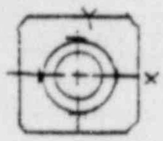
SO: VERTICAL ACCELERATION $= .067 G$ (SSE)
0.033 (OBE)

WELD DESIGN LOAD $= 1.067 \times (4 + 4.4) = 9.0^{\#}$

$1.033 \times 8.4 = 8.69^{\#}$

USE $^{\#}20.0$ lb.

1. AT CONNECTION BETWEEN BRACING & BEARING PLATE



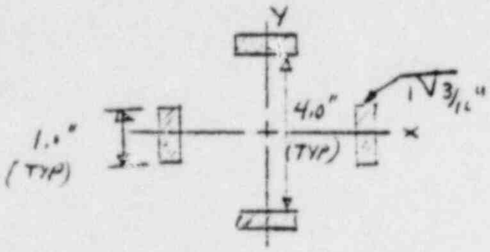
ELEVATION

PLACING MINIMUM WELD AT DESIGNATED LOCATIONS.

MIN. SIZE $= \frac{3}{16}''$

MIN LENGTH $= 4 \times \frac{3}{16} = \frac{12}{16}'' = .75$

USING $\frac{3}{16}''$ WELDS, 1" LENGTH:



"ASSUMED WELD CROSS-SECTION"

$I_x = I_y = 2\left(\frac{3}{16}\right)(1)(.707)(2.0)^2 = 1.061\ in^4$

$S_x = S_y = \frac{1.061}{2} = 0.53\ in^3$

DESIGN MOMENT $= 20^{\#} \times \frac{4.5}{2} = .045\ K\text{-}in$

WELD STRESSES

BENDING STRESS $= \frac{.045}{.53} = 0.085\ ksi$ (ABOUT X & Y AXES)

SHEAR STRESS $= \frac{20^{\#}}{(4)\left(\frac{3}{16}\right)(1)(.707)} = .033\ ksi$ (BOTH X & Y DIRECTIONS)

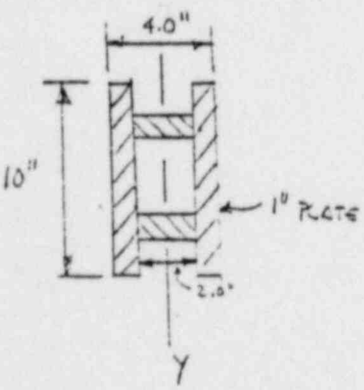
COMBINED STRESS $= \sqrt{2(0.085)^2 + (.033)^2\, 2} = 0.13\ ksi$ OK

	Ref.
DESIGN OF SEISMIC BRACING BEAM (UPENDER AREA)	

NES DWG. 80E1504
DETAIL No. 1 & 2

THE SEISMIC BRACING CAN NOT BE LOCATED BETWEEN RACKS AND WALL IN UPENDER AREAS OF POOL A WITHOUT CAUSING INTERFERENCE TO UPENDER OPERATION. THEREFORE A SEISMIC BRACING BEAM IS DESIGNED TO HANDLE SEISMIC LOADS AND TRANSFER THEM TO POOL WALLS WITHOUT INTERFERENCE TO UPENDER OPERATION.

SEISMIC BRACING BEAM PROPERTIES



$A_{BEAM} = (2)(10)(1) + (2)(2)(1) = 24.0\ in^2$

$I_y = \frac{2}{12}(10)(1)^3 + (2)(10)(1)(1.5)^2 + \frac{2}{12}(1.0)(2.0)^3$

$= 48.0\ in^4$

$S_y = \frac{I_y}{c} = \frac{48.0}{2.0} = 24.0\ in^3$

SEISMIC BRACING BEAM LOADING:

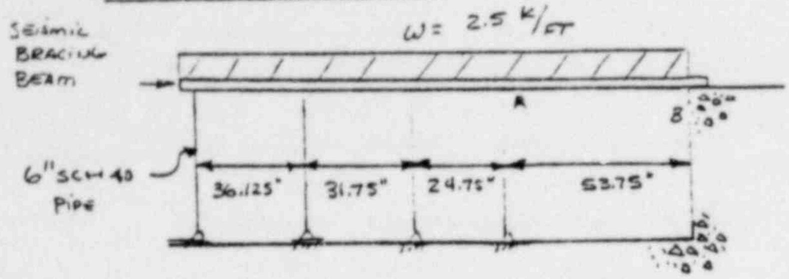
FROM POST RUNS 52600-W4 & -Q7, MAX. REACTIONS THAT THE SEISMIC BRACING BEAM WILL BE EXPOSURED TO FROM ONE RACK OCCUR WHEN X1 IMPACT IS APPLIED. THE TOTAL REACTION = 263.3 KIPS. THE TRUE REACTION RESULTING FROM 16 CELLS VS 26 CELLS AS USED IN POST RUN CALCULATION $= \frac{16}{26} \times 263.3 = 162.0$ KIPS

OR $\frac{162.0^K}{65.0} = 2.5\ ^K/_{FT}$ (RACK LENGTH = 65.0")

FOR DESIGN - ASSUME BEAM BEING UNIFORMILY LOADED WITH $2.5\ ^K/_{FT}$.

SEISMIC BRACING BEAM DESIGN (CONT'T) | Ref.

LOADING SITUATION



NES DWG 30E1504 DETAILS 1 & 2

FROM MOMENT DISTRIBUTION METHOD, MOMENT(MAX) = 678.1 K-IN OCCURS AT PT. B

AND MAX REACTION = 106.2K OCCURS AT PT. A

MAX. BEAM STRESSES : (OVERALL EFFECTS)

DESIGN MOMENT = 678.1 K-IN , $SHEAR_{MAX} = 106.2^K$

BENDING STRESS = $\frac{678.1}{24.0}$ = 28.2 ksi

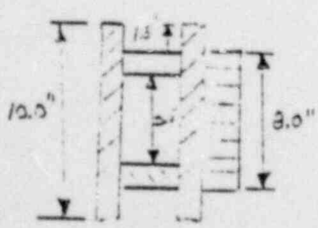
ALLOWABLE BENDING STRESS = (0.6)(30)(1.6) = 28.8 ksi > 28.2 ksi OK — #5

ALLOWABLE SHEAR STRESS = (0.4)(30)(1.6) = 19.2 ksi

ACTUAL SHEAR STRESS = $\frac{106.2}{24}$ = 4.4 ksi < 19.2 ksi OK — #5

LOCAL EFFECTS

THE MAX. INDIVIDUAL REACTION IS 26.3 K. APPLYING THIS LOAD TO AN INDIVIDUAL SHIM (6"×4") THE LOADING PER HORIZONTAL INCH OF SHIM IS 26.3/4 = 6.58 K/IN. TO DETERMINE LOCAL PLATE STRESSES, ASSUME A 1" STRIP WITH TOTAL LOAD EQUAL 6.59K — NES DWG 30E1504



$\omega = \frac{6.59^k}{8} = 0.82$ K/IN

DESIGN MOMENT = $\frac{(0.82)(5.0)^2}{12}$ = 1.71 K-IN

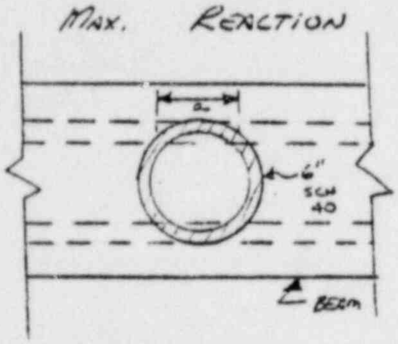
SEISMIC BRACING BEAM (CON'T)	Ref.

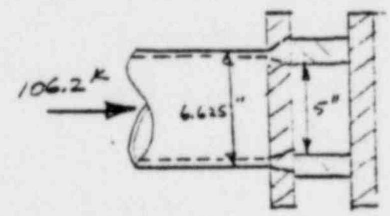
ASSUMING 1" STRIP : $S = \frac{bd^2}{6} = \frac{(1)(1)^2}{6} = 0.167\ in^3$

ACTUAL BENDING STRESS = $\frac{1.71}{0.167}$ = 10.25 ksi < 28.9 ksi ok #5

LOCAL EFFECTS (DUE TO 6" SCH 40 PIPING)

MAX. REACTION BETWEEN BEAM & PIPING = 106.2^K





BECAUSE OF THE INTERACTION BETWEEN PIPING AND THE WEBS OF THE BEAM, LITTLE PUNCHING SHEAR EXISTS. CONSIDERING BEARING:

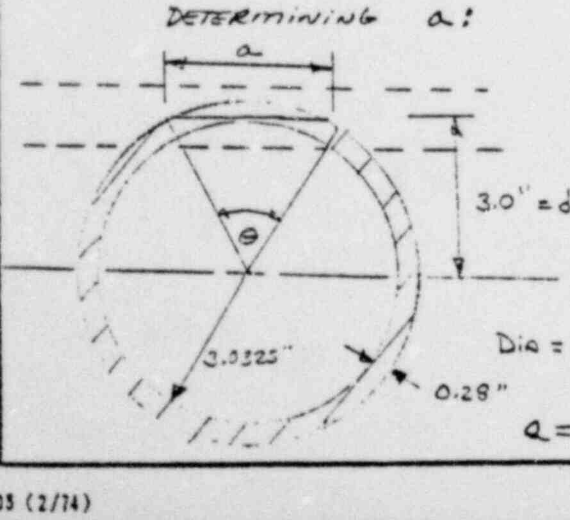
BEARING BETWEEN BRACING AND FLANGE:

BEARING AREA = 5.58 in^2 ; BEARING STRESS = $\frac{106.2}{5.58}$ = 19.03 ksi

ALLOWABLE BEARING = (.9)(30) = 27 ksi > 19.03 ksi Ref #5 ok

BEARING BETWEEN BRACE & WEBS: A SMALL PORTION OF BRACE BEARS ON WEBS OF BEAM. THE BEARING REGION INDICATED BY REGION a OF ABOVE SKETCH.

DETERMINING a: $d = R\cos\frac{\theta}{2}$ $\cos\frac{\theta}{2} = \frac{d}{R}$



$\cos\theta = \frac{3.0}{6.345/2} = 0.946$

$\frac{\theta}{2} = \cos^{-1} 0.946$

$\theta = 38.0°$

Dia = $\frac{6.625 + 6.065}{(2)}$ = 6.345" (MID-DEPTH OF PIPE)

$a = 2R\sin\frac{\theta}{2} = 2\left(\frac{6.345}{2}\right)\sin\frac{38.0}{2} = 2.07"$

	Ref.
SEISMIC BRACING BEAM (CONT)	

ASSUMING A 45° ANGLE INCREASE OF BEARING AREA ACROSS 1.0" FLANGE MEMBER, THE EFFECTIVE BEARING AREA BETWEEN PIPE & WEB:

$= [2.07 + (1.0) \times 2] \times 2 \times 1.0 = 8.14 \text{ in}^2$

BEARING STRESS $= 106.2/8.14 = 13.04 \text{ ksi} <$

ALLOWABLE BEARING STRESS $= (0.9)(30) = 27 \text{ ksi}$ ok — #5

BEARING PLATE ANALYSIS

Max. AXIAL LOAD in 6" SCH 40 pipe is 106.2^K.

min. REQUIRED BEARING AREA $= \dfrac{106.2}{(.85)(0.7)(3.0)} = 59.4 \text{ in}^2$ — #9

USING 8" x 9" PLATE:

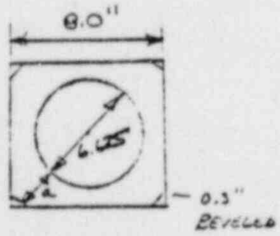


PLATE AREA $= (9)8 - (4)\dfrac{(.5)^2}{2} = 63.5 \text{in}^2$

CONCRETE BEARING STRESS $= \dfrac{106.2}{63.5} = 1.67 \text{ ksi}$

$< (0.85)(0.7)(3.0 \text{ ksi}) = 1.79 \text{ ksi}$ ok — #9

BEARING PLATE STRESSES: ASSUME 1" STRIP OF PLATE CANTILEVERED FROM PIPE RIM.

$a = \sqrt{(4)^2+(4)^2} - \dfrac{6.625}{2} - \sqrt{2(.5)^2} = 1.64"$

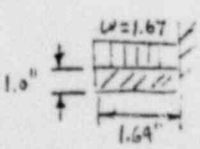


PLATE ELEVATION

$M = \dfrac{(1.67)(1.64)^2}{2} = 2.25 \text{ K-in}$

$S_{plate} = \dfrac{bt^2}{6} = \dfrac{(1)(1)^2}{6} = 0.167 \text{ in}^3$

PLATE BENDING STRESS $= \dfrac{2.25}{0.167} = 13.5 \text{ ksi} < 23.8 \text{ ksi}$ ok — #5

	Ref.
ADJUSTED FREQUENCY CALCULATIONS:	

CONSIDERING THE LONGEST SPAN - $\ell = 53.75''$

ASSUMING FIXED END BEAM:

$$f = 3.55 \Big/ \sqrt{\frac{W\ell^3}{384EI}}$$ WHERE: $W = 53.75 \times 2.5 = 134.4^K$, $I = 48.0\ in^4$,

$$f = 3.55 \Big/ \sqrt{\frac{(134.4)(53.75)^3}{(384)(28000)(48)}} = 17.7\ cps$$ #

ASSUMING SIMPLY-SUPPORTED:

$$f = 3.55 \Big/ \sqrt{\frac{5W\ell^3}{384EI}} = 3.55 \Big/ \sqrt{\frac{5(134.4)(53.75)^3}{(384)(28000)(48)}} = 7.89\ cps$$

$$f_{AVG} = 17.7 + 7.89 / 2 = 12.9\ cps$$

AXIAL FREQ. OF BRACING:

$$f = 3.13\sqrt{\frac{K}{W}}$$ $k = \frac{AE}{L} = \frac{(5.58)(28000)}{54} = 2895^K/_{in}$

$W = 106.2^K$

$$f = 3.13\sqrt{\frac{2895}{106.2}} = 16.34\ cps$$

COMBINED FREQ $= \frac{1}{(f_c)^2} = \frac{1}{(12.9)^2} + \frac{1}{(16.34)^2} = 10.09\ cps$

FOR A FULL LOADED RACK - (OUTPUT NO. 52700-DB XI HQR RUN)

f (2ND MODE) = 11.74 cps (DEVELOPED FOR RACK WITH 25 ADJACENT CELLS)

ADJUSTMENT FACTOR (TO ACCOUNT FOR Δ IN NO. OF CELLS)

$$\Delta f_{axial} = \sqrt{\frac{\Delta K}{\Delta W}} = \sqrt{\frac{25/16}{16/25}} = \frac{25}{16}$$ WHERE $\Delta K = \frac{EA}{\Delta L} = \frac{EA}{16/25}$

ADJUSTED $f = 11.74 \times \frac{25}{16} = 18.34\ cps$

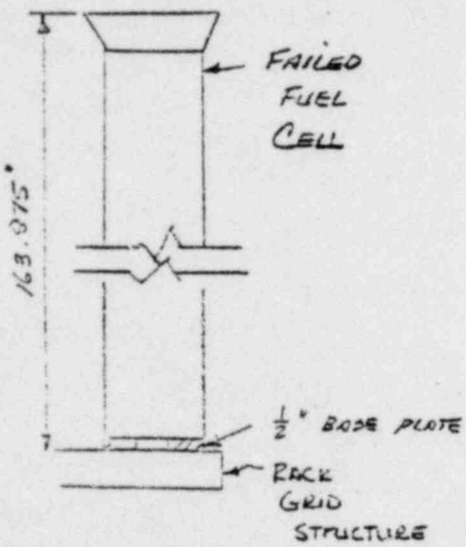
COMBINED RACK & SEISMIC BRACING FREQ. $= \frac{1}{f_c^2} = \frac{1}{(10.09)^2} + \frac{1}{(18.34)^2}$
$= 8.83\ cps$ *

* MODIFIED FREQ. TO ACCOUNT FOR DEFORMATION OF THE SEISMIC BRACING STRUCTURE IN THE UPENDER AREAS.

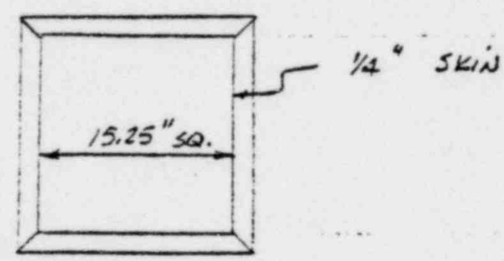
FAILED FUEL CELL ANALYSIS

APPENDIX H

OBJECTIVE: TO DESIGN FAILED FUEL CELL TO WITHSTAND EARTHQUAKE AND SECURE FUEL FUEL CANISTER.



ELEVATION



FAILED FUEL CELL PLAN

EACH FAILED FUEL CELL IS CANTILEVERED FROM THE RACK-BASE STRUCTURE SIMILAR IN DESIGN TO THE SPENT FUEL CELLS.

WEIGHT ANALYSIS

FAILED FUEL CELL WEIGHT $= \left\{\left[(15.25 + 2 \times .125)^2 - (15.25)^2\right](163.875)\,.286 + .286\left[(15.25)^2 \times .5\right]\right.$ (BASE PLATE) $\left.\right\}$ 1.1 (ASSUME 10% INCREASE)

$= 433^{\#}$

FAILED FUEL CANISTER $= 460^{\#}$ (DWG. NO. 136139 - B&W)

FUEL ASSEMBLY WEIGHT $= 1700^{\#}$

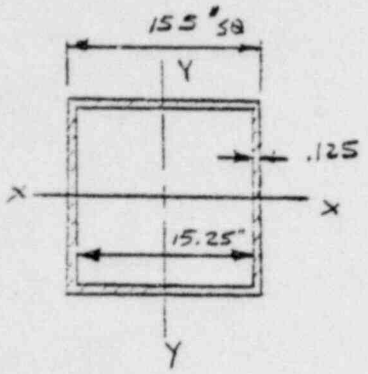
60% WATER ENTRAPPED $= (.6)(15.25)^2(0.0361)(163.875) = 826.0^{\#}$

HYDRODYNAMIC WATER = 100% WATER DISPLACED =

$$(1.0)(15.5)^2(163.875)(.03611) = 1421.7^{\#}$$

TOTAL $W = 433^{\#} + 460 + 1700^{\#} + 826.0^{\#} + 1422^{\#} = 4841.0^{\#}$
(INCLUDING HYDRODYNAMIC MASS)

FAILED FUEL CELL PROPERTIES



$$I_{xx} = I_{yy} = \frac{1}{12}\left[(15.5)^4 - (15.25)^4\right] = 302.9 \text{ in}^4$$

$$AREA = (15.5)^2 - (15.25)^2 = 7.69 \text{ in}^2$$

$$r_{xx} = r_{yy} = \sqrt{\frac{I}{A}} = \sqrt{\frac{302.9}{7.69}} = 6.28 \text{ in}$$

$$S_{xx} = S_{yy} = \frac{I}{c} = \frac{302.9}{15.5/2} = 39.09 \text{ in}^3$$

INDIVIDUAL CELL FREQUENCY:

$$f = \frac{3.89}{\sqrt{\frac{W l^3}{8EI}}} \quad \text{Ref. 11}$$

WHERE:
$W = 4.841^{K}$
$l = 163.875"$
$E = 28000$ ksi
$I = 302.9 \text{ in}^2$

$$f = \frac{3.89}{\sqrt{\frac{(4.841)(163.875)^3}{(8)(28000)(302.9)}}} = 6.94 \text{ cps}$$

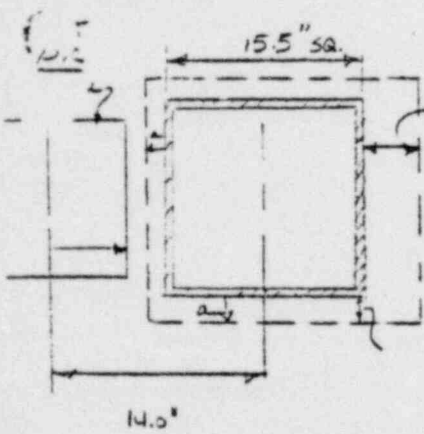
NOTE: SINCE THE FAILED FUEL CELL FREQ. IS GREATER THAN THE SPENT FUEL CELL FREQ. ($\approx$ 4.35 CPS), THE RACK FREQ. OF THE SPECIAL RACKS INCORPORATING THE FAILED FUEL CELLS WILL BE SIMILAR IF NO HIGHER THAN THE DESIGN RACK.

FAILED FUEL CELL STABILITY ANALYSIS

LOADING - THE CELL WILL BE SUBJECT TO HORIZONTAL ACCELERATION EQUAL TO PEAK ACCELERATION FROM ACC. SPECTRUM IN FPC SPEC. NO. SP-6486, G = 0.88.

STRESSES RESULTING FROM THE OBE OR SSE EARTHQUAKES ARE CAUSED FROM THE ACCELERATION OF DEAD WEIGHT PLUS SOME HYDRODYNAMIC MASS.

THE HYDRODYNAMIC MASS PRODUCING STRESSES IS EQUAL TO WEIGHT OF TRIBUTARY WATER AROUND THE FAILED FUEL CELL.



$$a = \frac{\left[14.0 - \frac{15.5}{2} - \frac{9.5075}{2}\right]}{2} = 0.75''$$

TRIBUTARY AREA = $(15.5 + 3.5 + .75)((.75)2 + 15.5) - (15.5)^2 = 95.5$ SQ IN

TRIBUTARY MASS = $95.5 \times 163.875 \times 0.0361 = 565^{\#}$

USE $575^{\#}$

DESIGN WEIGHT = $433^{\#} + 460^{\#} + 1700^{\#} + 826^{\#} + 575^{\#} = 3994.0^{\#}$

USE $\underline{4.0}^{K}$

BENDING MOMENTS: (ASSUMING MAX G = .88 & I.F. = 1.704)

M_{max} (X1 DIRECTION & X2) $= (4.0)(0.88)(1.704)\frac{163.875}{2} = 491.5$ K-IN

ALLOWABLE STRESSES: CHECKING THE COMPRESSION FLANGE FOR ELASTIC BUCKLING.

THE FAILED FUEL CELL SKIN IS FILLET WELDED VERY 3" ALONG TWO LAPPED JOINTS COMPOSED INTO CONSTRUCTION OF THE CELL. THE MINIMUM REQUIREMENTS OF REF. 5 ARE FOLLOWED INCLUDING MINIMUM AMOUNTS OF LAP AND SPACING (1.17.9) OF FILLET WELD HOLES.

OVERALL EFFECTS

DURING A SEISMIC EVENT, MAX. SHEAR OCCURS AT BASE OF CELL. MAX SHEAR = (1.704)(4.0)(.88) = 6.0 K

WHERE I.F = 1.704, G = 0.88 & FAILED FUEL WEIGHT = 4.0 K

LONGITUDINAL SHEARING FORCE/lin Ft (X DIRECTION) = $\dfrac{(6.0)\left(\frac{7.69}{2}\right)7.75}{244.5}$ = 0.731 K/in

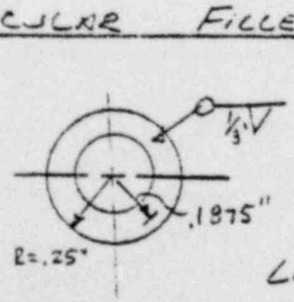
WHERE: I_{eff} = 244.5 in^4, A = 7.69 in^2

(y DIRECTION) = $\dfrac{(6.0)\left(\frac{7.69}{2}\right)(6.875)}{244.5}$ = 0.649 K/in

(ASSUMED MAX SHEAR TRANSMITTED THROUGH WELD)

FOR 3" SECTION (X DIRECTION) = (3)(.757) = 2.27 K
(Y DIRECTION) = (3)(.672) = 2.01 K

CIRCULAR FILLET WELD:



LENGTH OF FILLET = $2\pi\left(0.25 - \frac{0.125}{2}(.707)\right)$ = 1.29 in

EFF. AREA = 1.29 × .707 × .125 = 0.114 in^2

$S_x = (.707)(0.125)\,\pi(.1875)^2$ = 0.0098 in^3 Ref. 12

LONGITUDINAL WELD STRESSES:

(X DIRECTION) = $\dfrac{2.27^K}{.114}$ = 19.9 ksi

(y DIRECTION) = $\dfrac{2.01}{(1.704)(.114)}$ = 10.35 ksi (without impact)

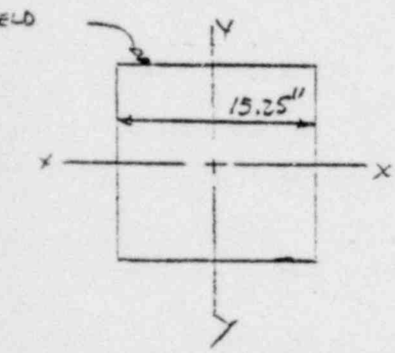
LOCAL EFFECTS THE LONGITUDINAL STRESSES RESULT FROM EXTERNAL MOMENTS PRODUCING TRANSVERSE STRESSES ACROSS THE WELDS. LOCAL EFFECTS PRODUCE STRESSES FROM SHEAR & LOCAL WEB BENDING.

LOCAL SHEAR LOAD : (ASSUME LOAD DISTRIBUTED TO 3" SEGMENTS)

$$L = (.83)(4)\,1.704 \times \frac{3}{163.875} = 0.11^{K}$$

LOCAL SHEAR STRESS (RESISTED BY TWO WELDS) $= \dfrac{0.11}{(2)(.114)} = 0.5$ ksi

LOCAL BENDING LOAD: (ACROSS WELD)



$$\omega = \frac{0.11^{K}}{15.25} = 0.0072\ ^{K}/_{in}$$

$$M_{max} = \frac{\omega l^2}{12} = \frac{(.0072)(15.25)^2}{12} = 0.140\ K\text{-}in$$

(ASSUMING WELD IN MIDDLE OF WEB)

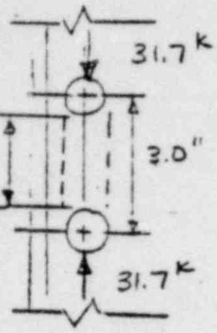
WELD BENDING STRESS $= \dfrac{0.140}{0.0098} = 14.34$ ksi

RESULTANT SHEAR STRESS $= \sqrt{(19.9)^2 + (10.35)^2 + (14.34+.5)^2} = 26.9$ ksi

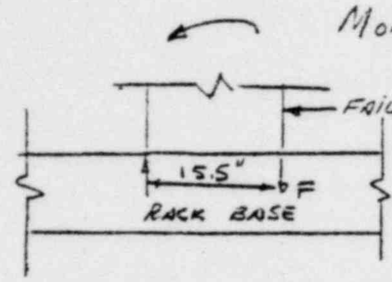
< 28.0 ksi ok Ref.5

NOTE: ALTHOUGH weld STRESS high, METHOD OF ANALYSIS is EXTREMELY CONSERVATIVE AND THEREFORE STRESSES SATISFACTORY.

LOCAL CELL BUCKING - INVESTIGATING THE REGION BETWEEN WELDS FOR LOCAL BUCKING.



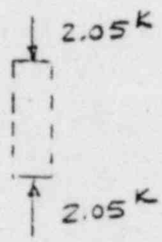
Moment = 491.5 K-in



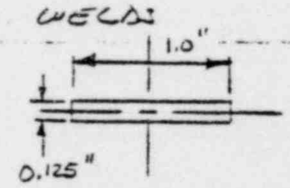
$F = \frac{491.5}{15.5} = 31.7^K$

OR uniform $w = \frac{31.7}{15.5} = 2.05^K/in$

ASSUMING THAT 31.7^K APPLIED ALONG COMPRESSION EDGE, & TREAT COMPRESSION EDGE AS COLUMN. LOCAL BUCKING WILL MOST LIKELY OCCUR NEAR UNWELDED EDGE ASSUMING 1" STRIP BETWEEN WELDS



SECTION PROPERTIES:



$A = 0.125 \text{ in}^2$

$I = \frac{1}{12}(1)(.125)^3 = 0.000163$

$S = \frac{0.000163}{.125/2} = 0.0026 \text{ in}$

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{0.000163}{.125}} = 0.036$

$\frac{Kl}{r} = \frac{(1)\ 2.5}{0.036} = 69.23$

$\frac{Kl/r}{C_c} = \frac{69.23}{135.7} = 0.510$

ALLOWABLE AXIAL LOAD (Ref. 5) $= \frac{\left[1-\left(\frac{Kl/r}{C_c}\right)^2 .5\right] F_y}{\frac{5}{3} + \frac{3}{8}\left(\frac{Kl/r}{C_c}\right) - \frac{1}{8}\left(\frac{Kl/r}{C_c}\right)^3} \times 1.6$

$F_a = \frac{\left[1 - (0.51)^2 .5\right]\ 30}{\frac{5}{3} + \frac{3}{8}(.51) - \frac{1}{8}(.51)^3} \times 1.6 = \underline{22.69 \text{ ksi}}$

$f_a = \frac{2.05}{.125} = 16.36 \text{ ksi} < 22.69 \text{ ksi}$ ok (Ref. 5)

FAILED FUEL CELL BASE PLATE WELD

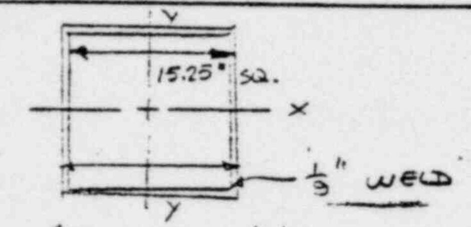
THE FAILED FUEL CELL SKIN IS WELDED TO ITS BASE PLATE WITH A $\frac{1}{8}$" FILLET WELD AND MUST WITHSTAND DESIGN MOMENT OF 491.5 K-in in ONE DIRECTION & 491.5/ = 799.4 K-in IN OTHER DIRECTION.

WELD AREA $= 4(15.25)(.707)(.125) = 5.39\ in^2$

$I_{xx} = I_{yy} = \frac{2}{12}(.125)(.707)(15.25)^3 + (2)(15.25)(.125)(.707)(7.75)^2$

$= 214.1\ in^4$

$S_{xx} = S_{yy} = \frac{214.1}{7.75} = 27.63\ in^3$



ASSUMED WELD CROSS-SECTION

WELD STRESSES

f_b (X DIRECTION) $= \frac{491.5}{27.63} = 17.79$ ksi (WITH IMPACT)

f_b (Y DIRECTION) $= \frac{288.4}{27.63} = 10.44$ ksi

SHEAR STRESS $= \frac{6.0^k}{5.39^k} = 1.11$ ksi (BOTH DIRECTIONS) WITH IMPACT. CONSERVATIVE

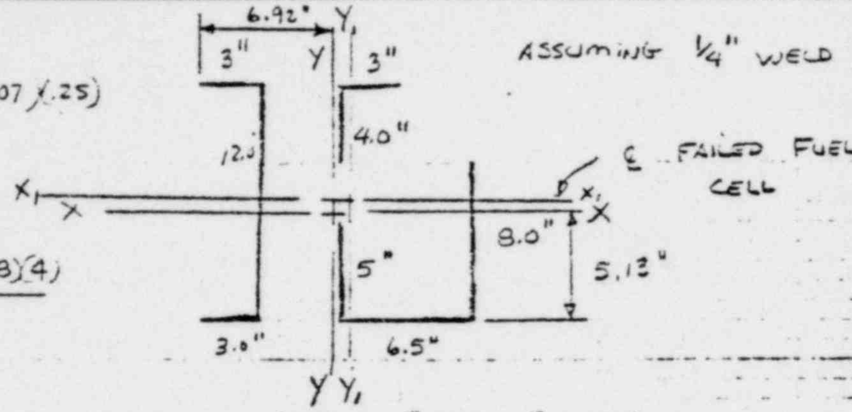
RESULTANT STRESS $= \sqrt{(17.79)^2 + (10.44)^2 + (1.11)^2 + (1.11)^2}$

$= 20.69$ ksi $<$ 28.0 ksi ok Ref. 5

WELD BETWEEN FAILED FUEL CELL & RACK BASE

THE FAILED FUEL CELLS ARE WELDED TO THE RACK GRID STRUCTURE WITH 1/4" FILLET WELD ATTACHING CELL BASE PLATE TO SURROUNDING FLANGES OF RACK STRUCTURE. THE WELD SELECTED FOR ANALYSIS HAS THE TYPICAL CROSS-SECTION AS ALL THE FAILED FUEL CELL EXCEPT FOR SOME limitation IN WELD AMOUNT DUE TO INTERFERRENCE, AND THEREFORE IS ASSUMED TO BE THE CONTROLLING WELD FOR ANALYSIS.

ASSUMING 1/4" WELD



ASSUMED WELD CROSS-SECTION

Weld Area $= [(3)(3) + 4 + 12 + 6.5 + 5 + 8](.707)(.25)$
$= 7.37 \text{ in}^2$

$$N.A_x = \frac{2(3)(12) + (12)(6) + (4.0)(10) + (5)(2.5) + (8)(4)}{(3)(3) + 12 + 4 + 5 + 6.5 + 8}$$

$= 5.13$" ✓ (FROM BOTTOM)

$$N.A_y = \frac{(2)(3)(1.5) + (12)(3) + (3)(8.5) + (4+5)(7) + (6.5)(10.25) + (8.0)(13.5)}{(3)(3) + 12 + 4 + 5 + 6.5 + 8} = 6.92\text{" FROM (LEFT EDGE)}$$

MOMENT OF INERTIA (ABOUT N.A OF WELD)

$$I_{N.A_x} = (2)(3)(.25)(.707)(12-5.13)^2 + \frac{1}{12}(.25)(.707)(12)^3 + (3)(.25)(.707)(5.13)^2 + (12)(.25)(.707)(\ldots$$
$$+ \frac{1}{12}(.25)(.707)(4)^3 + (4.0)(.25)(.707)(10-5.13)^2 + (5)(.25)(.707)(5.13-2.5)^2$$
$$+ (6.5)(.707)(.125)(5.13)^2 + \frac{1}{12}(.25)(.707)(8.0)^3 + (.25)(.707)(8)(5.13-4.0)^2$$
$$+ \frac{1}{12}(.25)(.707)(5)^3 = 141.2 \text{ in}^4$$

$$S_{NA_x} = \frac{141.2}{(12-5.13)} = 20.55 \text{ in}^4$$

$$I_{NA_y} = \frac{(2)}{12}(.25)(.707)(3)^3 + (2)(3)(.707 \times .25)(6.92-1.5)^2 + (12.0)(.25 \times .707)(3.92)^2$$
$$+ (5+4)(.25 \times .707)(.08)^2 + \frac{1}{12}(.707 \times .25)(3)^3 + (3)(.707 \times .25)(1.58)^2$$
$$+ \frac{1}{12}(.707)(.25)(6.5)^3 + (6.5)(.25 \times .707)(3.25+.08)^2 + (8.0)(.707 \times .25)(6.\ldots$$
$$= 144.32 \text{ in}^4$$

$$S_{NAy} = \frac{144.32}{6.92} = 20.85 \text{ in}^4$$

WELD STRESSES: ASSUMING IMPACT IN DIRECTION OF WEAKER SECTION MODULUS

DESIGN MOMENT (Y DIRECTION) = 491.5 K-IN
(X DIRECTION) = 288.4 K-IN

BENDING STRESSES:

$$f_{by} = \frac{491.5}{20.85} = 23.57 \text{ ksi} \quad (\text{with impact})$$

$$f_{bx} = \frac{288.5}{20.55} = 14.04 \text{ ksi} \quad (\text{WITHOUT IMPACT})$$

$$\text{SHEAR STRESS} = 6.0/7.87 = 0.76 \text{ ksi}$$

$$\text{RESULTANT STRESS} = \sqrt{(23.57)^2 + (14.04)^2 - (0.76)^2} = 27.45 \text{ ksi}$$

< 28.0 ksi ok Ref. 5.

NES 81A0523

THERMAL-HYDRAULIC DESIGN ANALYSIS REPORT

FOR THE

CRYSTAL RIVER NUCLEAR POWER STATION, UNIT 3

HIGH DENSITY FUEL STORAGE RACKS

Prepared Under NES Project 5127

For The

FLORIDA POWER CORPORATION

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1. SUMMARY

The adequacy of natural circulation flow to cool the spent fuel assemblies in the rack matrix was verified by establishing, for the worst row of assemblies, a thermal-hydraulic balance between the driving head produced by decay heat generation and the pressure losses existing in the natural circulation flow path.

Calculations have shown that under conservative assumptions the maximum assembly exit temperatures are below the saturation temperature of the pool water at fuel assembly elevations. Consequently, local boiling will not occur in any fuel assembly even with the bulk temperature of the spent fuel pool at its maximum value.

2. INTRODUCTION

In the NES rack design the cross flow of water between adjacent fuel assemblies is prevented by the stainless steel cells in the fuel rack. The effect is such that each of the fuel assemblies becomes isolated and, therefore, sits in its own thermal chimney. The chief thermal-hydraulic concern is the possibility of local boiling due to flow starvation in some cells of the rack matrix as a result of excessive pressure losses in the natural circulation loops established in the spent fuel pool.

The adequacy of the natural circulation flow to cool the hottest assembly in the rack configuration has been verified by establishing a thermal-hydraulic balance for the worst row of assemblies. Pressure losses in the downcomers, in the rack inlet plenum, and along the fuel assemblies were explicitly considered in the analysis. Crossflows in the rack inlet plenum area have been conservatively neglected.

The purpose of the analysis is to demonstrate that, even under the most conservative circumstances, local boiling will not occur in the most adversely located fuel assemblies which, as a result of flow maldistribution, might receive less than the fuel pool average assembly flow rate.

3. METHOD OF ANALYSIS AND ASSUMPTIONS

The natural circulation flow is calculated by establishing a thermal hydraulic balance for the worst row of assemblies. The flow is maintained by the thermal driving head or draft produced by the decay heat generation in each assembly. The pool itself is modeled as a large volume with a bulk temperature unaffected by local disturbances. The pressure losses considered in the analysis include:

1. Friction losses in the downcomer region, in the rack inlet plenum and in the fuel assembly.

2. Losses in bends (including the right angle turn that the flow must negotiate to turn from the horizontal rack inlet plenum channel into the vertical fuel assemblies).

3. Form losses in the fuel assembly inlet-outlet nozzles and spacer grids.

The chief concern is the possibility of substantial pressure drop along the inlet manifold channel, causing flow starvation of the fuel assemblies in the limiting fuel assembly string. Pressure losses along the channel in the fuel assemblies, spacer grid losses and inlet/exit pressure losses were explicitly considered in the analysis. Cross-flows have been neglected. All fuel assemblies are assumed to be generating the heat rate corresponding to 1.2 times the average power fuel assembly. A pool bulk temperature of 205°F is assumed.

The detailed thermal-hydraulic calculations are presented in Appendix A.

4. RESULTS OF ANALYSIS AND CONCLUSIONS

The thermal-hydraulic calculations indicate that even with the most conservative assumptions, the natural circulation in the spent fuel pool is adequate to preclude local boiling by a substantial margin. The maximum temperature increase in the assembly with the minimum flow is 28.1°F which would result in an outlet temperature of 233.1°F assuming a bulk pool temperature of 205°F.

The saturation temperature corresponding to the static head at the top of the fuel assembly is 240°F.

CRYSTAL RIVER - UNIT NO. 3

HIGH DENSITY SPENT FUEL RACKS

THERMAL-HYDRAULIC ANALYSIS

1. STATEMENT OF PROBLEM

Perform the thermal-hydraulic analysis to verify that the flow maldistribution resulting from the fuel storage rack configuration allows adequate cooling of the spent fuel assemblies.

2. ASSUMPTIONS

(1) The pool arrangement considered in this analysis combines the most conservative features of pools A and B : 5" downcomer gap (Pool A) with seismic bracing and a 10-assembly row from wall to pool centerline (pool B).

(2) The 10-assembly row consists of high peaking-factor assemblies discharged in the full-core discharge case. Peaking factor = 1.2

(3) The pool bulk temperature is 205°F. (SP-6486, 3:06.3)

(4) The pool water depth is 40 ft. The top of fuel is 25.5 ft from the water surface.

(5) The saturation temperature corresponding to the static head at the top of the fuel assembly is 240°F.

(6) Each bundle is isolated from adjacent assemblies by the cell walls and sits in its own thermal chimney.

(7) The inlet to the 10-assembly row is a 4.5" × 10.5" channel (ignoring the support pads) acting as an inlet manifold with 10 ports. The effect of crossflow in the open bottom plenum is conservatively neglected as this would tend to reduce the possibility of flow starvation.

(8) The assemblies are 15×15 (208 fuel rods: 0.430" OD, 0.568" pitch, 1 instrumentation tube: 0.493" OD, 0.441" ID, 16 Control Rod Guide Tubes: 0.530" OD, 0.498" ID).

(9) The can is 8.9375" square, 6 spacers, rod length = 155"
inlet channel length = 105" (cell pitch = 10.5")

(10) Average assembly heat generation = 149720 Btu/hr (SP-6486, 3:07.4, 16), assembly heat generation used in analysis = 149720 × 1.2 = 179660 Btu/hr

(11) Water properties based on estimated bundle average temperature of 215°F.

(12) The downcomer friction, seismic bracing and 90°-turn pressure losses, inlet plenum floor ΔP, bundle rod friction, spacer grid form losses, inlet and outlet form losses, 90°-turn losses from main channel into individual assemblies are considered.

3. ANALYSIS

3.1 DOWNCOMER LOSSES

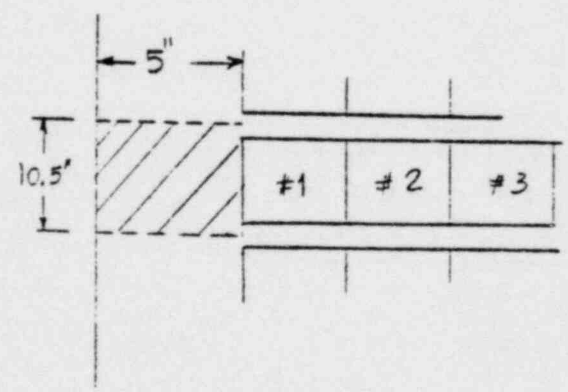
3.1.1 Pool Wall Friction ΔP

Flow Area = $5 \times 10.5'' = 52.5 \ in^2$

Wetted Perim = $2 \times 10.5'' = 21$ in

Hyd. Dia, $D_e = \frac{4A}{P} = \frac{4 \times 52.5}{21} = 10$ in

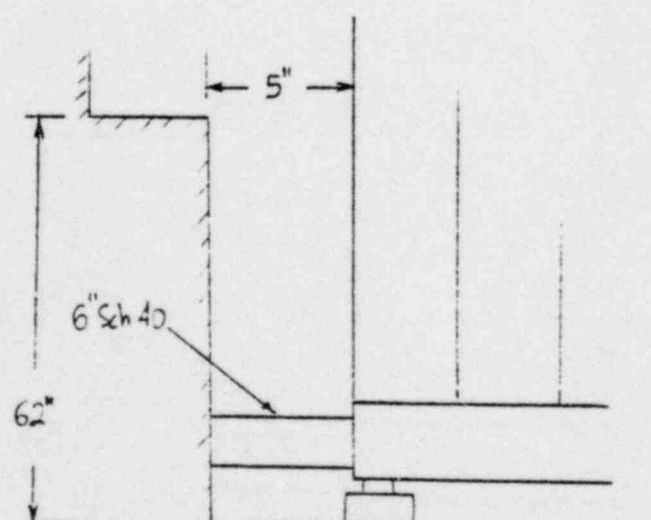
$L = 62$ in



Water properties @ 215°F

$\rho = 59.74 \ lb/ft^3$

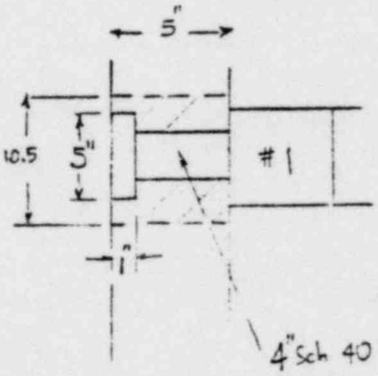
visc $\mu = .66 \ lb/hr\text{-}ft$



Assuming uniform flow in each assembly:

Flow/Assy (lb/hr)	Total Flow (lb/hr)	Velocity (ft/sec)	Re	f	ΔP_{df} $\left(=f\frac{L}{D}\frac{V^2}{2g}\rho\right)$
2500	25000	.319	86600	0.0185	.00008
5000	50000	.638	173200	0.0160	.00026
7500	75000	.957	259900	0.0145	.00053
10000	100000	1.275	346500	0.0140	.00091

3.1.2 Seismic Bracing ΔP



4" Sch 40 nom. dia = 4.5"

Free Flow Area $= (10.5 - 4.5) \times 5 - 5 \times 1 = 25 \text{ in}^2$

Ref: Idel'chik, Handbook of Hydraulic Resistance, AEC-tr-6630, 1966

Diag. 6-11

$\psi = 1.0$ since $\theta = 90°$

$S_1 = 10.5"$

$d_{out} = 4.5"$

$S_2 = 100\, d_{out}$ (to invalidate the effect of the 2nd row)

$z = 1$

$a_1 = 1.52\left(\frac{10.5 - 4.5}{450 - 4.5}\right)^{-.2} = 3.59$ $\qquad b_1 = \left(\frac{10.5}{4.5} - 1\right)^{-.5} = .87$

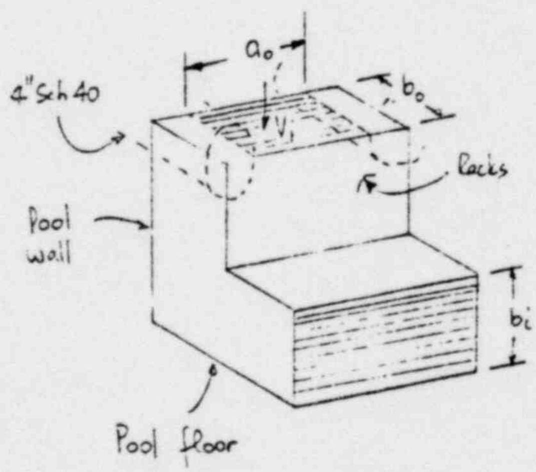
Re @ 25000 lb/hr = 81900 $\rightarrow Re^m = .104$ $m = -.2$) (using free flow area)

$K = 1.0 \times (3.59 \times .87) \times .104 \times 1 = .325$

Flow/ass'y (lb/hr)	Total Flow (lb/hr)	Velocity (ft/sec)	ΔP_{sb} $\left(= K \frac{V_p^2}{2g}\right)$ (psi)
2500	25000	.670	.00094
5000	50000	1.339	.00375
7500	75000	2.009	.00845
10000	100 000	2.678	.01501

3.1.3 Downcomer Turn ΔP

Idel'chik, Diag. 6-6



$a_o = 6"$
$b_o = 4"$
$b_i = 4.5"$

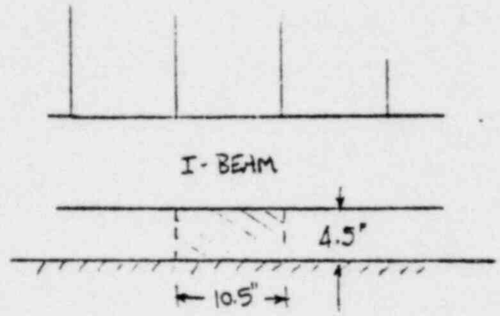
$\frac{a_o}{b_o} = \frac{6}{4} = 1.5$ $\qquad$ $\frac{b_i}{b_o} = \frac{4.5}{4} = 1.125$

$K \approx 1.2$

Flow/Ass'y (lb/hr)	Total Flow (lb/hr)	Vel. (ft/sec)	ΔP_t psi
2500	25000	.670	.00347
5000	50000	1.339	.01386
7500	75000	2.009	.03120
10000	100 000	2.678	.05544

3.2 BOTTOM PLENUM LOSSES

3.2.1 Floor friction losses



Ignoring support pads :

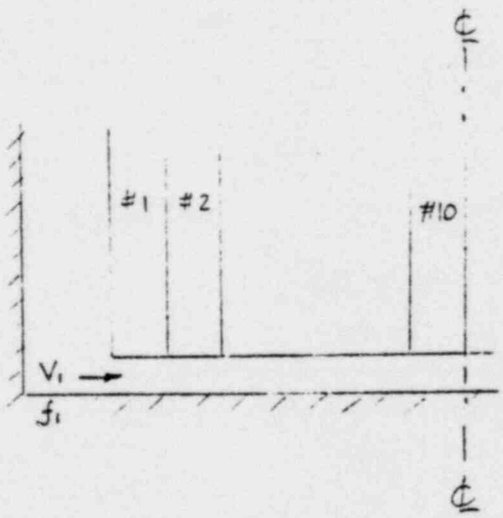
Flow Area = 10.5 × 4.5 = 47.25"

Wetted per. = 10.5 × 2 = 21"

(assuming I-beams to form top boundary for inlet plenum channel)

$$D_e = \frac{4(47.25)}{21} = 9"$$

$$L = 10.5 \times 10 = 105"$$



Assuming uniformly distributed flow in 10 bundles

$$\Delta P_{1-10} = C\, f_1 \frac{L}{D_e} \frac{\rho V_1^2}{2g}$$

Correction factor C = .364 for number of ports > 8 and $f \propto Re$.

(Chemical Engineering, Deskbook Issue, April 14, 1968, p178)

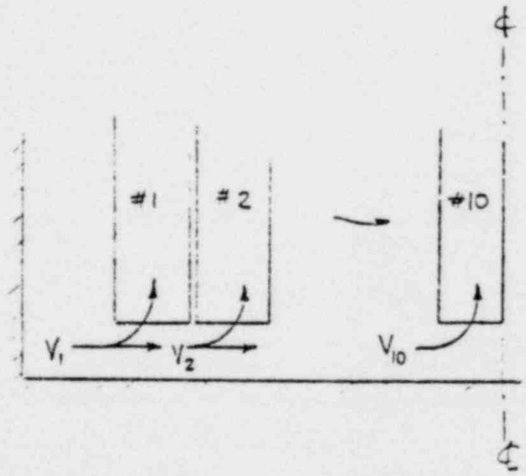
Flow/assy (lb/hr)	Total Flow (lb/hr)	V_1 (ft/sec)	Re	f_1	ΔP_{1-10} (= $C\Delta P_L$) (psi)
2500	25000	.355	67280	.0195	.00007
5000	50000	.709	134540	.0170	.00023
7500	75000	1.062	201800	.0155	.00048
10000	100000	1.412	269070	.0145	.00079

3.2.2 Floor bend losses

Assuming a "diverging wye" configuration, bend losses from the bottom plenum into individual assemblies:

V_1 is based on total flow

V_{10} is based on total flow /10



Idel'chik, Diagram 7-21

K ~ 1.0

Total Flow (lb/hr)	Flow/assy (lb/hr)	V_1 (ft/sec)	V_{10} (ft/sec)	ΔP_1 (psi)	ΔP_{10} (psi)
25000	2500	.355	.036	.00080	.00001
50000	5000	.709	.071	.00324	.00003
75000	7500	1.062	.107	.00727	.00007
100000	10000	1.412	.142	.01295	.00013

3.3 BUNDLE LOSSES

3.3.1 Rod friction ΔP

← 8.9375" →

208 @ .43" OD

1 @ .493" OD, .441" ID

16 @ .530" OD, .498" ID

Rod height = 155"

$$\text{Flow area} = (8.9375)^2 - 208\left(\frac{\pi}{4}\right)(.430)^2$$

$$-(1)\frac{\pi}{4}\left[(.493)^2-(.441)^2\right]$$

$$-16\frac{\pi}{4}\left[(.530)^2-(.498)^2\right] = 49.22 \text{ in}^2$$

$$\text{Wetted perim} = 4\times 8.9375 - 208\pi(.43) + \pi(.493+.441) + 16\pi(.53+.498) = 371.34 \text{ in}$$

$$D_e = \frac{4A}{P} = \frac{4\times 49.22}{371.34} = .53 \text{ in}$$

Flow/assy (lb/hr)	V_{cl} (ft/sec)	Re	f	ΔP_b $\left(= f\frac{L}{D}\frac{\rho V^2}{2g}\right)$
2500	.034	489	.132	.00029
5000	.068	979	.066	.00057
7500	.102	1470	.044	.00086
10000	.136	1957	.052	.00181

3.3.2 Bundle Form losses

$$K_{total} = K_{entr} + 6\,K_{spacer} + K_{exit}$$

$$= .5 + 6(.6) + 1.0 = 5.1 \quad \text{(estimate)}$$

Flow/Assy (lb/hr)	Vel (ft/sec)	$\Delta P_{bf} (= K \frac{\rho V^2}{2g})$
2500	.034	.00004
5000	.068	.00015
7500	.102	.00034
10000	.136	.00061

3.4 TOTAL ΔP

Flow/assy (lb/hr)	ΔP_1 $\Delta P_{df} + \Delta P_{sb} + \Delta P_t + \Delta P_1 + \Delta P_b + \Delta P_{bf}$	ΔP_{10} $\Delta P_{df} + \Delta P_{sb} + \Delta P_t + \Delta P_{1-10} + \Delta P_{10}$ $\Delta P_b + \Delta P_{bf}$
2500	.0056	.0049
5000	.0218	.01882
7500	.04865	.04193
10000	.08673	.07470

3.5 NATURAL CIRCULATION HEAD

The driving force for the natural circulation flow is the density difference between the average hot column in the assemblies and the cooler bulk temperature in the downcomer.

The water column is assumed to be 14.5 ft, conservatively neglecting any plume effects. The head is calculated by

$\Delta P_H = H(\rho_{cold} - \rho_{hot})$

$\rho(205°) = 59.988$ lb/ft³

$\rho(215°) = 59.737$

$\Delta P_H = 14.5(59.988 - 59.737)\frac{1}{144} = .0253$ psi/10°F

$= .00253$ psi/°F

avg Δ°F = $T_{avg\,hot} - T_{avg\,cold}$ (ΔT between avg. hot bundle column and the downcomer)

$T_{avg\,hot} = \frac{T_{in} + T_{out}}{2}$

$T_{in} = T_{avg\,cold} = T_{bulk} = 205$°F

avg Δ°F $= \frac{205 + T_{out}}{2} - 205 = \frac{T_{out}}{2} - 102.5$

Q = Assembly heat generation = 179660 Btu/hr

Flow/Assy (lb/hr)	$T_{out} - T_{in}$ (°F) $\left(\frac{Q}{Flow}\right)$	T_{out} (°F) (T_{in} = 205)	avg Δ°F	ΔP_H (psi)
2500	71.86	276.86	35.93	.0909
5000	35.93	240.93	17.97	.0455
7500	23.95	228.95	11.98	.0303
10000	17.97	222.97	8.99	.0227

4, RESULTS

From Fig. 1, the minimum flow in the pool for worst heat load case:

$$\Delta T = \frac{179660}{6400} = 28.1 \ ^\circ F$$

$$T_{EXIT} = 205 + 28.1 = 233.1 \ ^\circ F$$

$T_{EXIT} = 233.1^\circ$ is less than saturation temperature of 240°F at highest elevation.

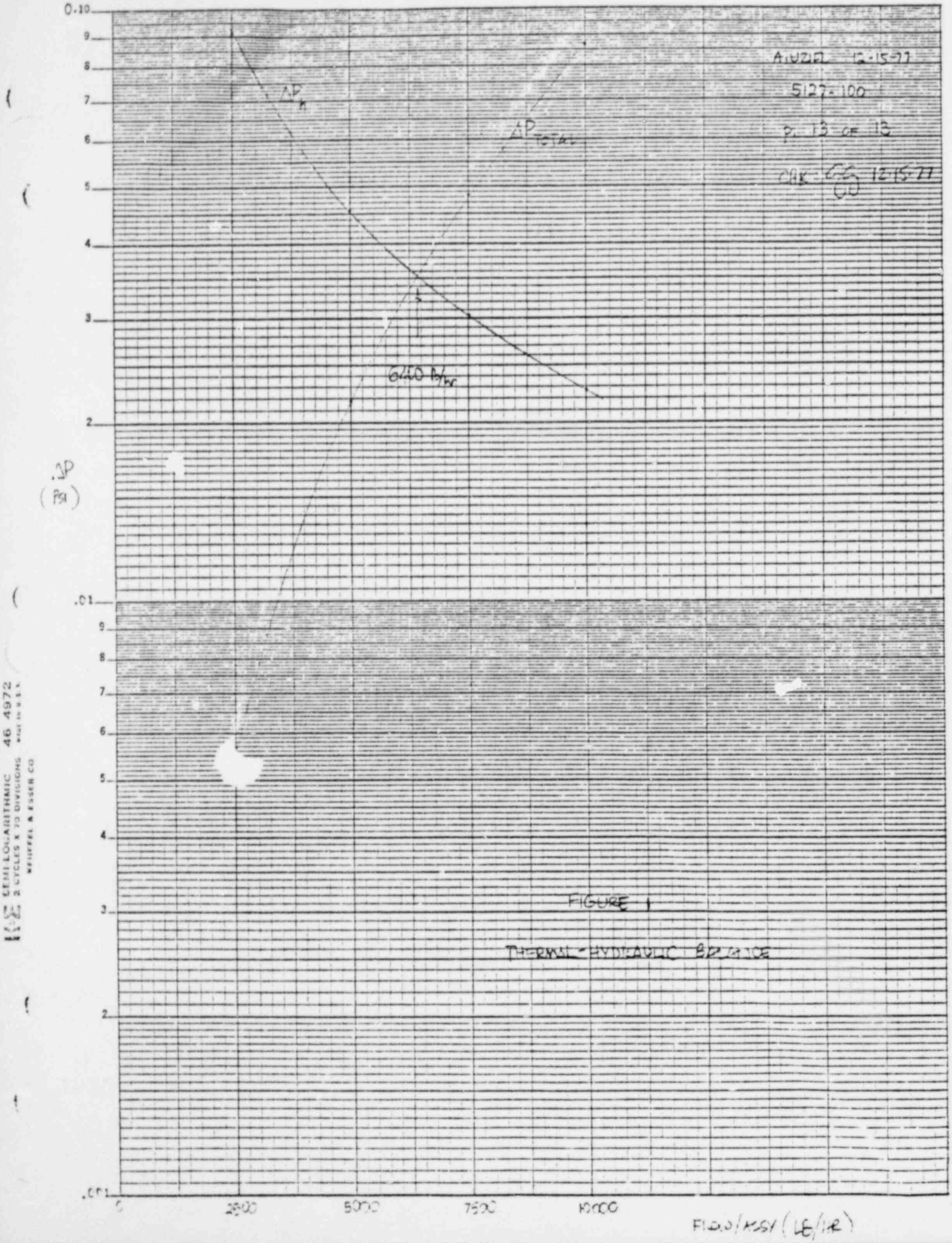


FIGURE 1

THERMAL-HYDRAULIC BALANCE