Fermi 2
6400 North Dixie Hwy., Newport, MI 48166

Detroit Edison

May 31, 2000
NRC-00-0042

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington D C 20555-0001

References: 1) Fermi 2
NRC Docket No. 50-341
NRC License No. NPF-43

2) Detroit Edison letter to NRC "Proposed Technical Specification Changes (License Amendment) – Design Features/ Fuel Storage (Technical Specification 4.3) and Programs and Manuals/High Density Spent Fuel Racks (Technical Specification 5.5.13)", dated November 19, 1999

Subject: Response to Request for Additional Information on Technical Specifications Change Request Related to Spent Fuel Pool Expansion at Fermi 2 (TAC No. MA7233)

On February 9, 2000, Detroit Edison received a set of NRC questions pertaining to Enclosure 4 (Licensing Report) of the proposed Technical Specifications change request to increase the capacity of the Fermi 2 Spent Fuel Pool (Reference 2). On May 3, 2000, a teleconference between Detroit Edison, NRC, and Holtec staffs was conducted to discuss these questions. Enclosed is Detroit Edison's response to the questions.

Should you have any questions or require additional information, please contact Mr. Norman K. Peterson of my staff at (734) 586-4258.

Sincerely,

William T. O'Connor

William T. O'Connor, Jr.
Vice President - Nuclear Generation

Enclosure

cc: A. J. Kugler
M. A. Ring
NRC Resident Office
Regional Administrator, Region III
Supervisor, Electric Operators,
Michigan Public Service Commission

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I, WILLIAM T. O'CONNOR, JR., do hereby affirm that the foregoing statements are based on facts and circumstances which are true and accurate to the best of my knowledge and belief.

William T. O'Connor Jr.
WILLIAM T. O'CONNOR, JR.
Vice President - Nuclear Generation

On this 31 st day of May, 2000 before me personally appeared William T. O'Connor, Jr., being first duly sworn and says that he executed the foregoing as his free act and deed.

Notary Public

NORMAN K. PETERSON
Notary Public, Monroe County, MI
My Commission Expires July 24, 2002

ENCLOSURE TO
NRC-00-0042

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION ON FERMI 2 RERACK LICENSE SUBMITTAL

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION ON FERMI 2 RERACK LICENSE SUBMITTAL

Question 1

You indicate that the structural analyses for the required loading conditions were performed in compliance with the NRC Standard Review Plan (SRP) and the former NRC Office of Technology (OT) Position Paper related to spent fuel storage. With respect to your structural analyses using the DYNARACK computer code presented in Chapter 6 of Enclosure 4:

(a) Explain how the target (design basis) response spectra (refer to in Section 6.4) were obtained. Also demonstrate that the synthetic time histories used in the analyses satisfy the power spectral density (PSD) requirements of SRP 3.7.1.

(b) Provide the analysis results that show that the design criteria related to kinematic stability (i.e., safety factors against rack overturning) described in Chapter 2 of Enclosure 4 have been satisfied. Also, provide the dimensions of the gaps between the racks, and between the racks and the spent fuel pool (SFP) walls.

Response to Question 1 (a)

The target (design basis) response spectra were obtained from Section 3.7 of the Fermi 2 UFSAR. In particular, the seismic response spectra for fifth floor elevation 684'-6", which is 38'-9" above the SFP floor slab, were conservatively used to generate bounding acceleration time histories. The target spectra for Fermi 2 were originally developed by Sargent & Lundy.

Two sets of artificial time histories were used in the structural analyses: a Safe Shutdown Earthquake (SSE) and an Operating Basis Earthquake (OBE). These synthetic time histories are statistically independent with their cross correlation coefficients less than 0.15. Furthermore, they satisfy the response spectrum and PSD enveloping requirements of SRP 3.7.1. Figures 1.1 through 1.6 provide a comparison of the PSD curves associated with the generated time histories and the PSD curves associated with the design basis response spectra. These figures clearly show that the generated PSD curves envelope the target curves over the entire frequency range.

Response to Question 1 (b)

In order to demonstrate that the spent fuel racks are kinematically stable, a bounding single rack analysis was performed (See Run No. 32 on page 6-24 of the Licensing Report). Rack D was selected for this analysis because it has the highest aspect ratio (i.e., length to width ratio), which, on geometric grounds, makes this rack kinematically most limiting with respect to overturning. From the rack seismic analysis results, the maximum computed displacement is 0.8295 inch. To reach the incipient point of overturning, the top of Rack D must displace nearly 38 inches (i.e., distance between pedestal centerlines). Therefore, the minimum safety factor against rack overturning is 45 (38.0 in/0.8295 in), which is far greater than the minimum required safety factor of 1.5. Clearly, the kinematic acceptance criteria stated in Chapter 2 and Subsection 6.7.1 of the Licensing Report are satisfied.

Finally, Figure 1.7 shows the gaps between the racks and the gaps between the racks and the spent fuel pool (SFP) walls for the final rerack Campaign III.

Question 2

You indicate in Enclosure 4 that the calculations of SFP capacity were performed satisfying the design conditions described in SRP 3.8.4 and American Concrete Institute (ACI) Code 349-85. With respect to the SFP capacity calculations using the ANSYS computer code presented in Chapter 8 of Enclosure 4:

(a) Explain how the interface between the liner and the concrete slab is modeled, and also, how the liner anchors are modeled. Explain how such modeling accurately represents the real structural behavior.

(b) Provide physical dimensions of the reinforced concrete slab and walls, liner plate and the details of the liner anchorage.

Response to Question 2 (a)

The structural capacity analysis of Fermi 2 Spent Fuel Pool is based on Campaign III of the rerack as described in Section 8.3 of the Licensing Report. The pool liner is not included in the overall 3-D ANSYS structural model of the spent fuel pool. Any contribution to the pool structural support by the thin liner is conservatively neglected. The stress analysis of the liner is considered in a separate analysis focused on the in-plane stress distribution. The liner in the Fermi 2 pool is assembled from austenitic steel plates which are seam welded along their contiguous edges resulting in a sealed container geometry to hold the pool water. The seam weld lines are also locations of anchor. The integrity analysis of the pool liner consisted of the following evaluations:

(i) Confirmation that the in-plane stresses in the liner during the seismic event would not cause rupture in the liner from a single load application.

(ii) Confirmation that repetitive loading during a seismic event will not lead to fatigue failure in the liner (1 SSE and 20 OBEs occurring in sequence is the design basis).

To evaluate the stress field in the liner, it is modeled as a 2-Dimensional plate, which is fixed along its edges to simulate the weld seams. The liner anchors are assumed to be rigid, and therefore, are not explicitly modeled. A bounding liner geometry was utilized wherein the weld lines are conservatively assumed to be nearest to the rack pedestal location. The finite element solution evaluated the stress distribution at the line of support representing the weld seam.

Response to Question 2 (b)

The inside (plan) dimensions of the Spent Fuel Pool are 34'-0" (North-South) by 40'-0" (East-West). The pool depth, which is measured from the top of the liner on the fuel pool floor (EL. 645'-9") to the top of the pool curb (EL. 684'-10"), is 39'-1".

The contents of the pool are supported by a two-way, reinforced concrete slab. The minimum thickness of the slab is 72 inches, excluding the grout layer. The SFP walls to the east and to the west are 6 feet thick. The thickness of the north wall is reduced from 72 inches to 48 inches above elevation 659'-6", where the new fuel storage pool is located. The south wall of the pool is an integral part of the concrete reactor shield wall, and it has a minimum thickness of 4 feet. The walls are braced from the outside by several intermediate slabs.

An array of 1/4 inch thick stainless steel plates form the pool liner. The plates are spliced together by 1-1/2 inch × 1/2 inch rectangular bars, which also provide the backing surface for the liner seam welds. The liner anchorage consists of an array of 3/8 inch diameter by 4 inch long bolts, which are fastened to the rectangular bar. These bolts are embedded in the concrete on 12 inch spacing along the plate splices.

Question 3 (a)

Section 7.5.3 "Deep Drop Event" in Enclosure 4 states that the maximum compressive stress in the concrete slab for a rack deep drop accident is 8.3 ksi, which is larger than the concrete compressive strength of 5.9 ksi. Under such conditions, explain the basis for your conclusion that substantial damage to the pool slab is not indicated.

Response to Question 3 (a)

The results of the deep drop analysis show that the high stress region of the slab, where the maximum compressive stress is larger than the compressive strength of the concrete, is limited to a circular area of less than 4.0 inches in diameter of the slab as shown in the stress contour plot of Figure 7.5.4 of the Licensing Report. The rest of the slab area is shown to be in tension with the maximum tensile stress of only 29.5 psi, which can be easily supported by the concrete without cracking. Also, the piercing of liner and the rack pedestal support penetration through the concrete slab are not indicated by this analysis. The pool concrete slab thickness is 72 inches. Analysis shows that the estimated depth of local concrete crushing is 0.032 inch.

The conclusion that the concrete slab sustains a localized crushing is based on a conservative assumption. The analysis conservatively uses a static compressive strength of the concrete, i.e., 5.9 ksi, as the failure limit of the concrete for a dynamic event such as the deep drop. In fact, the concrete failure limits for a dynamic event should be much higher than the static limit, as suggested by many credible textbook references. With regards to Fermi 2 spent fuel pool structure, the upper stratum of the pool slab supporting the stainless steel liner, which is laterally confined and simultaneously compressed from the interior of the pool water pressure, exhibits a triaxial compressive stress behavior, which reduces the tendency of internal cracking. This suggests that a stress-strain curve, which can accurately represent the triaxial behavior of the concrete slab, would be more appropriate for this analysis. Attachment 1 provides a plot of stress-strain curves for concrete subjected to triaxial compression, which has been taken from Park and Paulay's text (Reference 1). This stress-strain plot has been used as input in the drop accident analyses for several different nuclear plants including Union Electric's Callaway Plant, Wolf Creek Plant and Commonwealth Edison's Byron and Braidwood plants. Based on this plot, the concrete failure stresses are much higher than the unconfined compressive strength of 5.9 ksi. For example, A.M. Neville's textbook, "Properties of Concrete", (Reference 2) gives a failure stress of 14,700 psi for 3,500 psi concrete, which is consistent with Park and Paulay (Reference 1).

Therefore, although the concrete failure limit is much higher than 5.9 ksi, a conservative conclusion that the concrete slab is locally crushed was made in Section 7.5.3 of the Licensing Report based on the unconfined static compressive strength 5.9 ksi.

Question 3 (b)

Section 7.5.3 "Rack Drop Event" in Enclosure 4 states that the maximum compressive stress in the concrete slab in the case of the heaviest rack drop accident is 36.8 ksi compared to the compressive strength of 5.9 ksi. Describe the type, and quantitative results, of your analysis that shows that a primary failure in the SFP structure will not occur, as described in your application.

Response to Question 3 (b)

The rack drop analysis performed for Fermi 2 uses the standard modeling techniques and methodology used on several past licensing submittals including Union Electric's Callaway Plant, Wolf Creek Plant, and Commonwealth Edison's Byron and Braidwood plants. The LS-DYNA3D model used in the rack drop analysis investigates the local damage sustained by the pool liner and the supporting pool slab resulting from a rack drop. In this model, the rack was conservatively modeled as a rigid structure represented by the rack pedestal support block. The density of the material describing the pedestal block was modified to coincide to the weight of the rack. This simplified model neglects the impact energy absorption in the rack structure to conservatively evaluate the local damage.

The modeling of the rack as the rigid support block is an extremely conservative assumption due to the fact that during the drop event there will be no absorption of the impact energy by the rigid support block and therefore, the entire impact energy will be transferred to the pool slab causing enormous impact forces in the slab.

In fact, the rack assembly, which is made of 0.075 inch thick and 175 inch long sheets formed into 6.0 inch square boxes and interconnected with intermittent welds along the cell height (as described in Chapter 3.0 of the Licensing Report), is a much more flexible structure than the reinforced concrete slab. This implies that during the drop event, a significant portion of the impact energy will be absorbed by the rack structure and thus ensuring a substantial reduction in the impact energy transferring to the pool slab.

In order to provide accurate and realistic results with respect to the impact energy absorption by the rack and the concrete slab to demonstrate the safety margin against the failure of the SFP structure, the existing LS-DYNA3D model was re-run with the same parameters as those used in the existing model, except that in this model the cellular rack cells were modeled as elastic-plastic type.

The time history of the impact force generated by this simulation was filtered at the appropriate frequency level to calculate the resulting impact force. The maximum impact force is indicated to be 2.25×10^5 lbf per pedestal. The concrete stratum directly beneath the pedestal sustains localized high compressive stresses exceeding the concrete compressive strength of 5.9 ksi. However, this high compressive stress region is limited to a region less than 5.0 inches in diameter. This indicates that the slab will experience localized crushing. The rest of the modeled slab area is in tension but the maximum tension stress is only 99.6 psi, which can be easily supported by the concrete without cracking. Analysis shows that the estimated depth of local crushing of the concrete is 0.641 inch.

To demonstrate the safety margin against the primary failure evaluation of the Spent Fuel Structure slab, a comparison of the total slab load associated with the racks and fuel assemblies present during a rack drop accident with the corresponding load used in the spent pool structural evaluation (Chapter 8.0 of the Licensing Report) was made.

The total load applied to the slab during a rack drop accident includes the impact load of a dropped empty rack, the combined weight of racks and fuel assemblies present in the pool at the time of the accident. As described in Chapter 1.0 of the Licensing Report, Fermi 2 Spent Fuel Pool will be reracked into three Campaigns. The heaviest rack (Rack B) will be installed in Campaign I. For conservatism, the impact force resulting from the postulated drop of the heaviest rack B will be combined with the load from racks and fuel assemblies present during the accident from all three Campaigns to determine the bounding total slab load.

Using the maximum impact force of 2.25×10^5 lbf per pedestal, the total impact load transferred to the slab by the dropped rack is (2.25×10^5 lbf X 4 pedestals) 9.0×10^5 lbf. Campaign II has the maximum combined weight of existing racks and fuel assemblies from all three campaigns. During Campaign II, conservatively assuming that all existing storage cells are filled with fuel assemblies, there are up to 3146 fuel assemblies as shown in Chapter 1.0 of the Licensing Report, each of which weighs 690 lbf (actual weight of the fuel assembly is 680 lbf), stored in the existing racks whose combined weight is 461,445 lbs. By using the buoyancy factor of 0.87, the bounding combined wet weight of the existing racks plus fuel assemblies is 2.29×10^6 lbf. Combining this load with the dropped rack impact load of 9.0×10^5 lbf yields a total slab load of 3.19×10^6 lbf.

In the structural analysis (Chapter 8.0 of the Licensing Report) of the Spent Fuel Pool, the bounding slab load associated with (wet) fully loaded racks including the seismic effect applied to the slab is 4.5×10^6 lbf. The pool slab structure under this bounding load has been shown to be safe with a significant margin shown in Chapter 8.0 of the Licensing Report.

The above comparison demonstrates that the total load applied to the slab by racks during a rack drop accident is much smaller than the corresponding load used in the structural analysis for the spent fuel pool. It is therefore concluded that a rack drop event would not lead to primary failure of the pool slab.

Question 3 (c)

In section 7.5.3 "Rack Drop Event" in Enclosure 4, you state that the liner plate experiences a maximum stress of 46.8 ksi in the heaviest rack drop accident resulting in the yielding of the liner plate without failure. As this liner plate stress is higher than the yield stress, describe the extent of the inelastic deformation of the plate and the impact to the integrity of the pool structure.

Response to Question 3 (c)

The liner plate is plastically deformed during the heaviest rack drop accident. The maximum vertical deflection of the liner plate is 0.641 inch. The liner plate experiences a maximum stress of 46.8 ksi, which is less than the failure limit of 71 ksi. This indicates that the deformed liner transfers the impact load to the concrete slab through the contact interface between the liner and the concrete slab without breaching the liner. The compressive stress distribution in the slab indicates a localized crushing of the concrete. The remaining area of the slab is in tension with maximum stress of 99.6 psi. The rack pedestal punch through the pool slab is not shown to occur by this analysis.

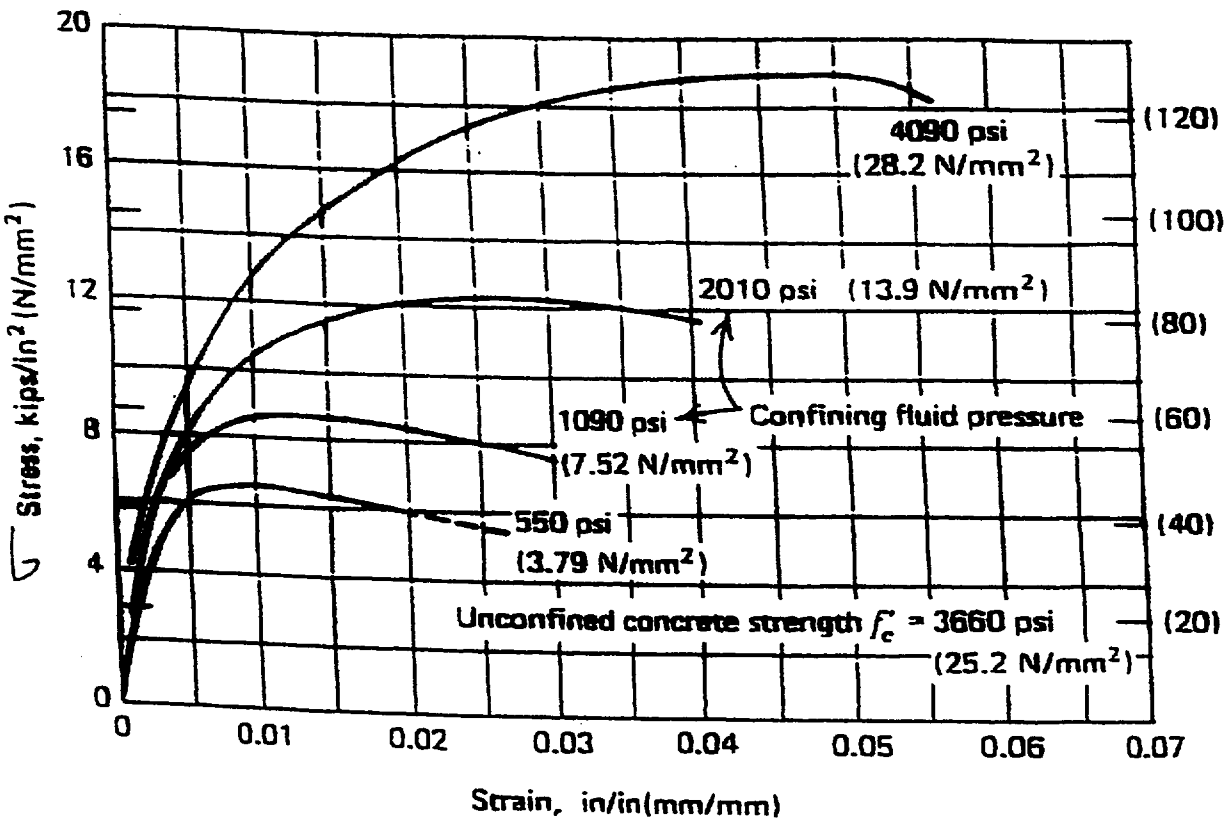
References:

1. R. Park and T. Paulay, "Reinforced Concrete Structure", John Wiley and Sons, 1975

2. A.M. Neville, "Properties of Concrete", 4th Edition

Attachment 1

Stress-Strain Curves for concrete subjected to triaxial compression *



* R. Park and T. Paulay, "Reinforced Concrete Structures," Figure 2.19, John Wiley and Sons, 1975.

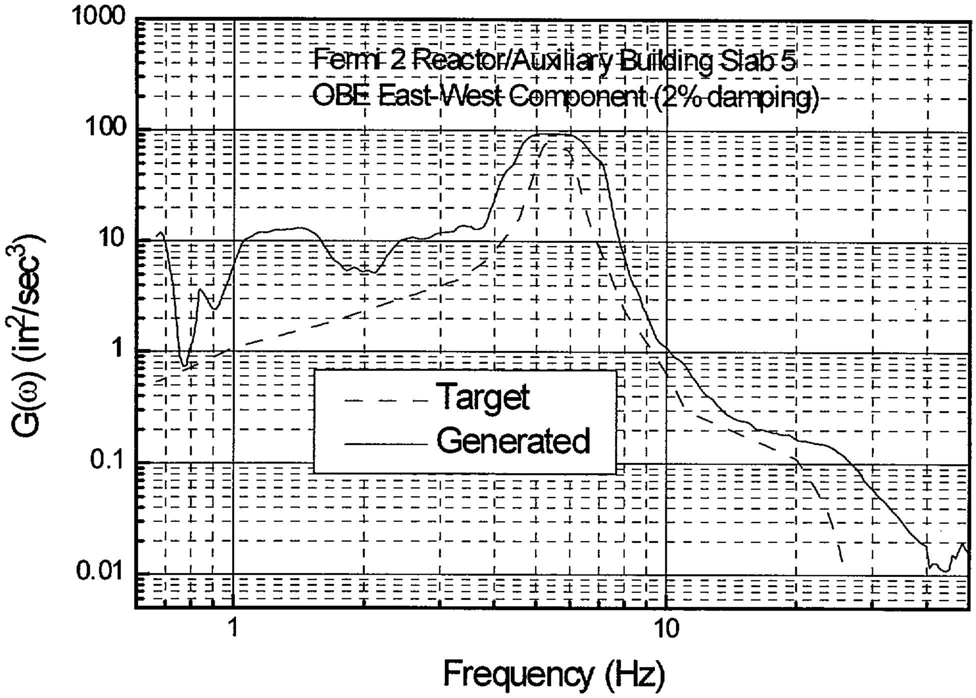


Fig. 1.1 - OBE power spectral density function in the x-direction (East-West)

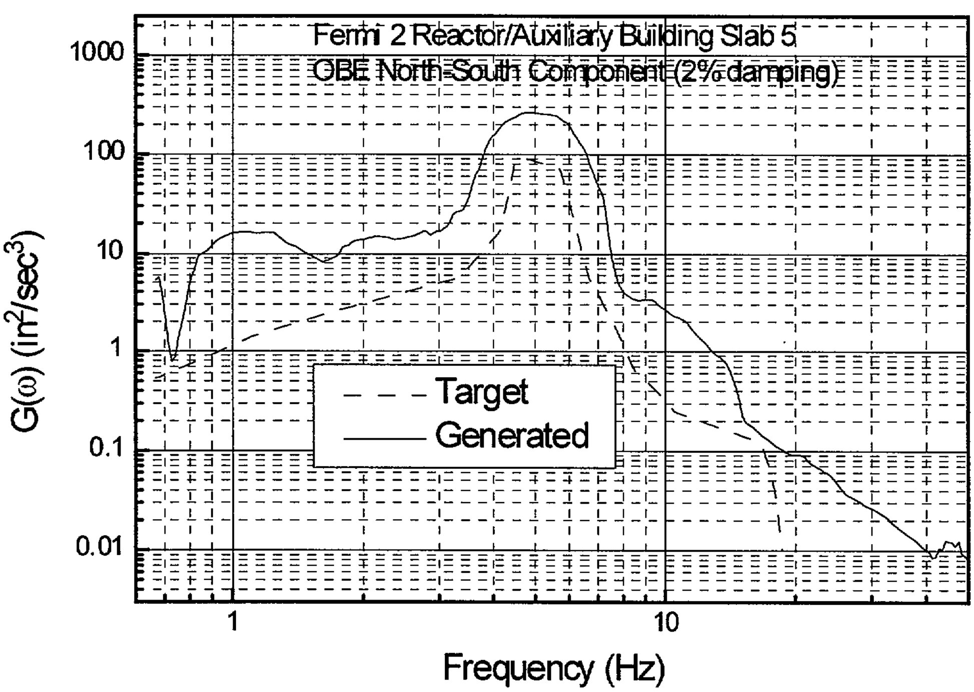


Fig. 1.2 - OBE power spectral density function in the y-direction (North-South)

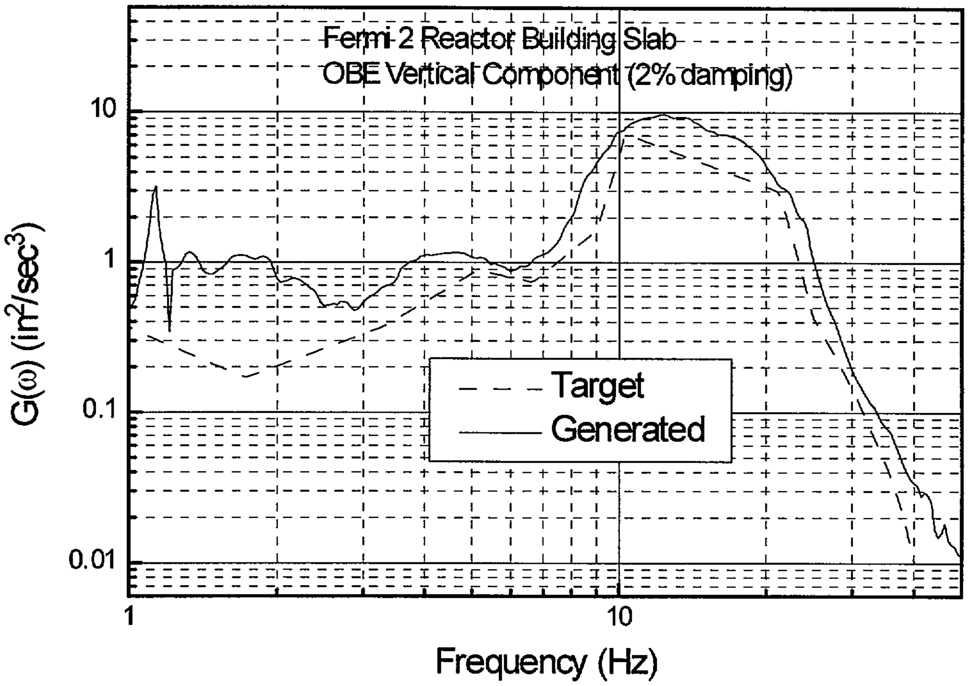


Fig. 1.3 - OBE power spectral density function in the z-direction (Vertical)

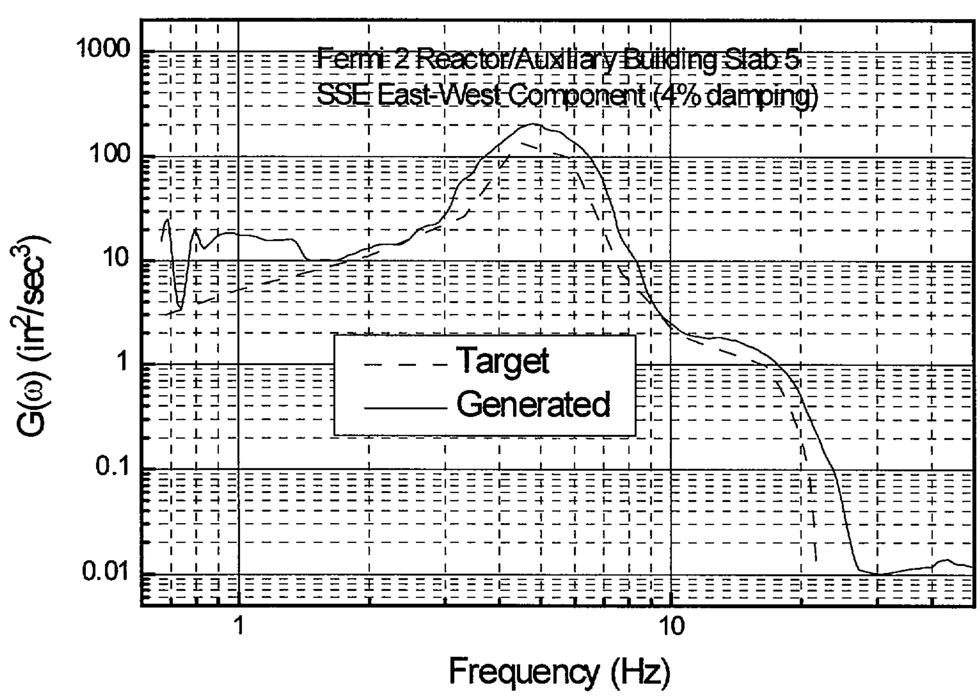


Fig. 1.4 - SSE power spectral density function in the x-direction (East-West)

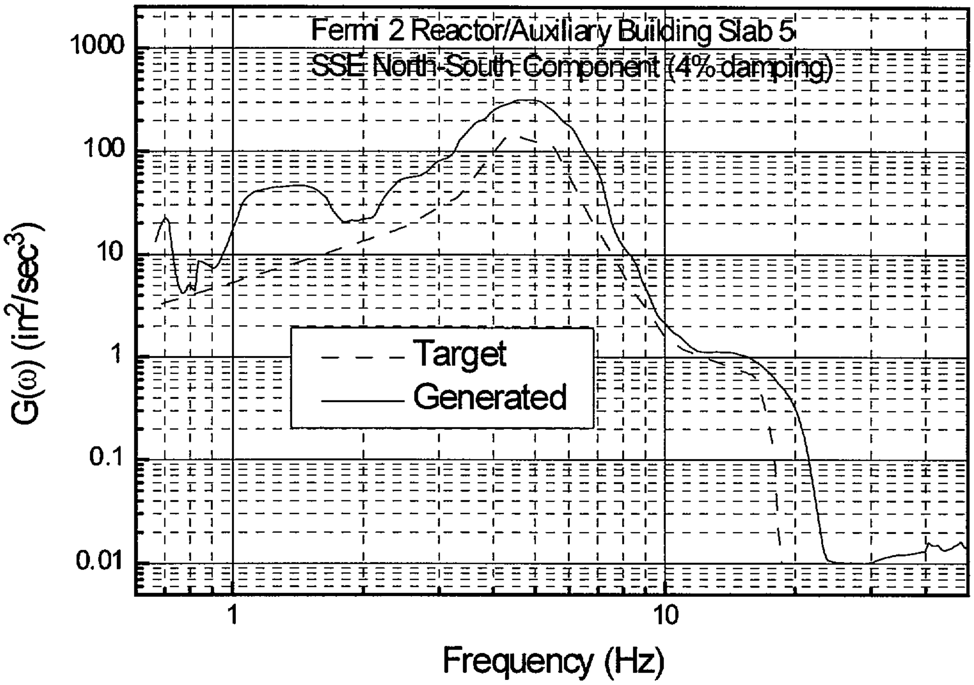


Fig. 1.5 - SSE power spectral density function in the y-direction (North-South)

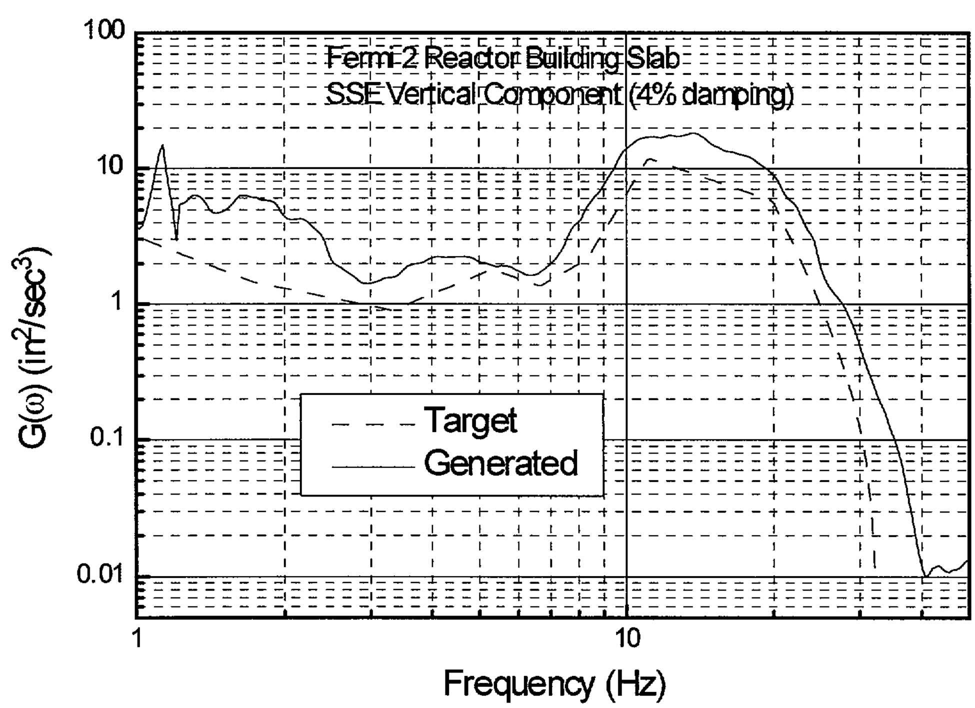


Fig. 1.6 - SSE power spectral density function in the z-direction (Vertical)

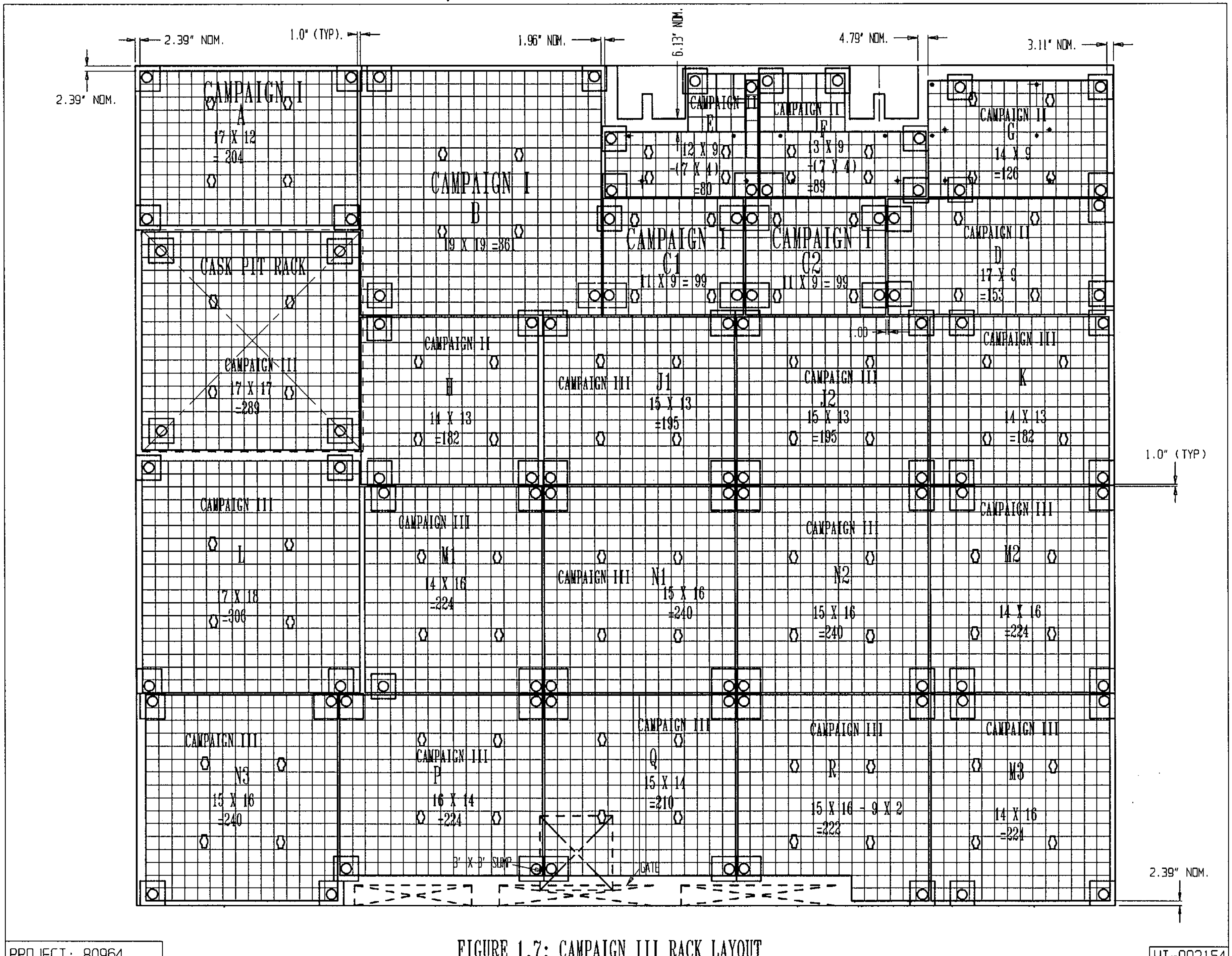


FIGURE 1.7; CAMPAIGN III RACK LAYOUT