VERMONT YANKEE
NUCLEAR POWER CORPORATION

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September 16, 1999
BVY 99-115

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555

References: (a) Letter, VYNPC to USNRC, "Technical Specification Proposed Change No. 207, Spent Fuel Pool Storage Capacity Expansion," BVY 98-130, dated September 4, 1998.

(b) Letter, USNRC to VYNPC, "Vermont Yankee Nuclear Power Station, Request for Additional Information Regarding Spent Fuel Pool Storage Capacity Expansion (TAC No. MA 3490)," NVY 99-68, dated July 14, 1999.

Subject: **Vermont Yankee Nuclear Power Station**
License No. DPR-28 (Docket No. 50-271)
Response to Request for Additional Information Regarding
Spent Fuel Pool Storage Capacity Expansion

In Reference (a), Vermont Yankee proposed a change to the Technical Specifications to increase the spent fuel storage capacity from 2,870 to 3,355 fuel assemblies. During the review of the proposed change, the Staff requested via Reference (b) the submittal of additional information with regard to Civil and Mechanical Engineering considerations. Attachment A, which includes related Attachments 1A through 1K provided our response to Staff Question number 1. It is noted that Attachments 1A through 1E contain proprietary information and it is requested that these attachments be withheld from public disclosure per 10CFR2.790(a)(4). Attachment 2 provides the response to Staff question number 2. Attachment 3 contains Holtec International's proprietary information affidavit for the proprietary information contained within Attachments 1A through 1E.

We trust that the enclosed information will enable you to complete your review of Reference (a).

If you have any questions on this transmittal, please contact Mr. Thomas B. Silko at (802) 258-4146.

Sincerely,

VERMONT YANKEE NUCLEAR POWER CORPORATION

Don M. Leach
Vice President Engineering

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PDR 1 1 w/o prop

STATE OF VERMONT)
)ss
WINDHAM COUNTY)

Then personally appeared before me, Don M. Leach, who, being duly sworn, did state that he is Vice President Engineering of Vermont Yankee Nuclear Power Corporation, that he is duly authorized to execute and file the foregoing document in the name and on the behalf of Vermont Yankee Nuclear Power Corporation, and that the statements therein are true to the best of his knowledge and belief.

T.B. Silko

Thomas B. Silko, Notary Public
My Commission Expires February 10, 2003

cc: USNRC Region 1 Administrator
USNRC Resident Inspector – VYNPS
USNRC Project Manager – VYNPS
Vermont Department of Public Service

Docket No. 50-271
BVY 99-115

Attachment 3

Vermont Yankee Nuclear Power Station

Response to Request for Additional Information Regarding
Spent Fuel Pool Storage Capacity Expansion
(Civil and Mechanical Engineering Considerations)

Holtec International's proprietary information affidavit

AFFIDAVIT PURSUANT TO 10CFR2.790

I, Alan I. Soler, being duly sworn, depose and state as follows:

(1) I am the Executive Vice President for Holtec International and have been delegated the function of reviewing the information described in paragraph (2) which is sought to be withheld, and have been authorized to apply for its withholding.

(2) The information sought to be withheld is contained in the following attachment documents to "RAI Response for Vermont Yankee Nuclear Power Station"
Attachment 1A; Holtec Position Paper WS-115, Revision 1, 3D Single Rack Analysis of Fuel Racks.
Attachment 1B; Holtec Report HI-87113, Rev. 0, Evaluation of Fluid Flow for In-Phase and Out-of-Phase Rack Motions.
Attachment 1C; Holtec Report HI-87114, Rev. 0, Estimated Effects of Vertical Flow Between Racks and Between Fuel Cell Assemblies.
Attachment 1D; Holtec Report HI-87102, Rev. 0, Study of Non-Linear Fluid Coupling Effects.
Attachment 1E; Holtec Report HI-87112, Rev. 0, Fluid Flow in Narrow Channels Surrounding Moving Rigid Bodies.

(3) In making this application for withholding of proprietary information of which it is the owner, Holtec International relies upon the exemption from disclosure set forth in the Freedom of Information Act ("FOIA"), 5 USC Sec. 552(b)(4) and the Trade Secrets Act, 18 USC Sec. 1905, and NRC regulations 10CFR Part 9.17(a)(4), 2.790(a)(4), and 2.790(b)(1) for "trade secrets and commercial or financial information obtained from a person and privileged or confidential" (Exemption 4). The material for which exemption from disclosure is here sought is all "confidential commercial information", and some portions also qualify under the narrower definition of "trade secret", within the meanings assigned to those terms for purposes of FOIA Exemption 4 in, respectively, Critical Mass Energy Project v. Nuclear Regulatory Commission, 975F2d871 (DC Cir. 1992), and Public Citizen Health Research Group v. FDA, 704F2d1280 (DC Cir. 1983).

(4) Some examples of categories of information which fit into the definition of proprietary information are:

a. Information that discloses a process, method, or apparatus, including supporting data and analyses, where prevention of its use by Holtec's competitors without license from Holtec International constitutes a competitive economic advantage over other companies;

b. Information which, if used by a competitor, would reduce his expenditure of resources or improve his competitive position in the design, manufacture, shipment, installation, assurance of quality, or licensing of a similar product.

c. Information which reveals cost or price information, production, capacities, budget levels, or commercial strategies of Holtec International, its customers, or its suppliers;

d. Information which reveals aspects of past, present, or future Holtec International customer-funded development plans and programs of potential commercial value to Holtec International;

e. Information which discloses patentable subject matter for which it may be desirable to obtain patent protection.

The information sought to be withheld is considered to be proprietary for the reasons set forth in paragraphs 4.a, 4.b, 4.d, and 4.e, above.

(5) The information sought to be withheld is being submitted to the NRC in confidence. The information (including that compiled from many sources) is of a sort customarily held in confidence by Holtec International, and is in fact so held. The information sought to be withheld has, to the best of my knowledge and belief, consistently been held in confidence by Holtec International. No public disclosure has been made, and it is not available in public sources. All disclosures to third parties, including any required transmittals to the NRC, have been made, or must be made, pursuant to regulatory provisions or proprietary agreements which provide for maintenance of the information in confidence Its initial designation as proprietary information, and the subsequent steps taken to prevent its unauthorized disclosure, are as set forth in paragraphs (6) and (7) following.

(6) Initial approval of proprietary treatment of a document is made by the manager of the originating component, the person most likely to be acquainted with the value and sensitivity of the information in relation to industry knowledge. Access to such documents within Holtec International is limited on a "need to know" basis.

(7) The procedure for approval of external release of such a document typically requires review by the staff manager, project manager, principal scientist or other equivalent authority, by the manager of the cognizant marketing function (or his designee), and by the Legal Operation, for technical content, competitive effect, and determination of the accuracy of the proprietary designation. Disclosures outside

Holtec International are limited to regulatory bodies, customers, and potential customers, and their agents, suppliers, and licensees, and others with a legitimate need for the information, and then only in accordance with appropriate regulatory provisions or proprietary agreements.

(8) The information classified as proprietary was developed and compiled by Holtec International at a significant cost to Holtec International. This information is classified as proprietary because it contains detailed historical data and analytical results not available elsewhere. This information would provide other parties, including competitors, with information from Holtec International's technical database and the results of evaluations performed using codes developed by Holtec International. Release of this information would improve a competitor's position without the competitor having to expend similar resources for the development of the database. A substantial effort has been expended by Holtec International to develop this information.

(9) Public disclosure of the information sought to be withheld is likely to cause substantial harm to Holtec International's competitive position and foreclose or reduce the availability of profit-making opportunities. The information is part of Holtec International's comprehensive spent fuel storage technology base, and its commercial value extends beyond the original development cost. The value of the technology base goes beyond the extensive physical database and analytical methodology, and includes development of the expertise to determine and apply the appropriate evaluation process.

The research, development, engineering, and analytical costs comprise a substantial investment of time and money by Holtec International.

The precise value of the expertise to devise an evaluation process and apply the correct analytical methodology is difficult to quantify, but it clearly is substantial. Holtec International's competitive advantage will be lost if its competitors are able to use the results of the Holtec International experience to normalize or verify their own process or if they are able to claim an equivalent understanding by demonstrating that they can arrive at the same or similar conclusions.

The value of this information to Holtec International would be lost if the information were disclosed to the public. Making such information available to competitors without their having been required to undertake a similar expenditure of resources would unfairly provide competitors with a windfall, and deprive Holtec International of the opportunity to exercise its competitive advantage to seek an adequate return on its large investment in developing these very valuable analytical tools.

STATE OF NEW JERSEY)
) ss:
COUNTY OF BURLINGTON)

Alan I. Soler, being duly sworn, deposes and says:

That he has read the foregoing affidavit and the matters stated therein are true and correct to the best of his knowledge, information, and belief.

Executed at Marlton, New Jersey, this 13th day of September 1999.

Alan I. Soler

Alan I. Soler
Holtec International

Subscribed and sworn before me this 13th day of September, 1999.

Maria C. Pepe

MARIA C. PEPE
NOTARY PUBLIC OF NEW JERSEY
My Commission Expires April 25, 2000

Attachment 1

Vermont Yankee Nuclear Power Station

Response to Request for Additional Information Regarding
Spent Fuel Pool Storage Capacity Expansion
(Civil and Mechanical Engineering Considerations)

Response to Question 1

REQUEST FOR ADDITIONAL INFORMATION ON THE SPENT FUEL POOL STORAGE RACK MODIFICATION AT VERMONT YANKEE NUCLEAR STATION

Item 1

You indicated that the calculated seismic loading stresses in a fully-loaded rack will not exceed that of SRP Section 3.8.4 which was used as a guide. With respect to your stress calculations using the DYNARACK computer code presented in Chapter 6 of the submittal, provide the following:

a) Explain how the simple stick model used in the dynamic analyses can represent accurately and realistically the actual highly-complicated nonlinear hydrodynamic fluid-rack structure interactions and behavior of the fuel assemblies and the box-type rack structures. Discuss whether or not a finite element (FE) model with 3-D plate, beam and fluid elements together with appropriate constitutive relationships would be a more realistic, accurate approach to analyze the fluid-structure interactions in contrast to the stick model.

b) Provide the results of any prototype or experimental study that verifies the correct or adequate simulation of the fluid coupling utilized in the numerical analyses for the fuel assemblies, racks and walls. If there is no such experimental study available, provide justification that the current level of the DYNARACK code verification is adequate for engineering application without further experimental verification work.

c) Provide the stiffness calculations for the rack used in single rack analysis for the three (North-South, East-West and Vertical) directions.

d) Provide the physical dimensions of the racks, gaps between the racks, and the gaps between the racks and the walls.

e) Demonstrate that the artificial seismic time histories used in the analyses satisfy the power spectral density (PSD) requirement of Standard Review Plan (SRP) Section 3.7.1.

Response 1a

As explained in Sections 6.2 and 6.5 of Holtec International Report HI-981932, the Whole Pool Multi-Rack (WPMR) model used to predict the dynamic behavior of the storage racks contains elements specifically designed to represent the attributes necessary to simulate rack motions during earthquakes. These elements include non-linear springs to develop the interaction between racks, between racks and walls, and between fuel assemblies and rack internal cell walls. Linear springs having the necessary characteristics to capture the lowest natural frequencies of the ensemble of fuel cells acting as an elastic beam-like structure in extension/compression, two-plane bending, and twisting are used to simulate rack structural elastic action. Hydrodynamic effects within these interstitial spaces are accounted for using Fritz's classical method which relates the fluid kinetic energy in the annulus due to relative motion to an equivalent hydrodynamic mass. In the next section, we present a historical overview of the fluid coupling effect as applied to the modeling of spent fuel racks in a seismic environment.

The phenomenon of fluid coupling between rectangular planform structures was sparsely investigated until the 1980s. Fritz's classical paper (ca. 1972) was used in the *earliest* version of DYNARACK to model rack-to-surrounding fluid effects in the so-called single rack 3-D simulation. Enrico Fermi Unit 2 (ca. 1980) and Quad Cities Units 1 and 2 (ca. 1982) were licensed using the Fritz fluid coupling terms embedded in DYNARACK. The Fermi 2 and Quad Cities 1 and 2 submittals were the *first* rerack applications wherein a rack module was analyzed using the 3-D time-history technique. The adoption of a nonlinear time-history approach helped quantify the motion of a rack under a 3-D earthquake event and as a byproduct, also served to demonstrate that solutions using the Response Spectrum Method (which, by definition, presumes a linear structure) can be woefully non-conservative. Practically all rerack licensing submittals from 1980 on utilized the 3-D time-history method. While the nonlinear 3-D time-history method was a huge improvement over the Response Spectrum (by definition, linear) approach, it nevertheless was limited inasmuch as only one rack could be modeled in any simulation. The analyst had to *assume* the behavior of the adjacent racks. Models, which postulated a priori the behavior of the contiguous racks in the vicinity of the subject rack (rack being analyzed), were developed and deployed in safety analyses. Two most commonly used models were the so-called "opposed phase" model and the "in-phase" model, the former used almost exclusively to predict inter-rack impacts until 1985. Holtec Position Paper WS-115 (proprietary), included herein as Attachment 1A, provides a summary description of these early single rack 3-D models.

The inadequacy of the single rack models (albeit nonlinear) to predict the response of a grouping of submerged racks arrayed in close proximity became an object of prolonged intervenors' contention in the reracking of PG&E's Diablo Canyon units in 1986-87. Holtec, with active participation from the USNRC, developed a 2-D multi-rack model for the Diablo Canyon racks; this model helped answer intervention issues, permitting PG&E to rerack. USNRC experts testified in support of the veracity of the 2-D multi-rack dynamic models at the ASLB hearings in Pismo Beach, California in June 1987.

The Diablo Canyon intervention spurred Holtec to develop what later came to be known as the 3-D Whole Pool Multi-Rack (WPMR) analysis. A key ingredient in the WPMR analysis is quantification of the hydrodynamic coupling effect that couples the motion of *every* rack with *every* other rack in the pool. In 1987, Dr. Burton Paul (Professor Emeritus, University of Pennsylvania) developed a fluid mechanics formulation using Kelvin's recirculation theorem that provided the fluid coupling matrix (2n x 2n for a pool containing n racks).

As an example, refer to Figure RAI 1.1, where an array of N (N = 16) two-dimensional bodies (each with two degrees of freedom) is illustrated. The dynamic equilibrium equation for the i-th mass in the x-direction can be written as:

$$\left[m_i + M_{ii}\right]\ddot{x}_i + \sum_{j=1}^{N}\left[M_{ij}\ddot{x}_j + N_{ij}\ddot{y}\right] = Q_{x_i}(t)$$

In the above equation, m_i is the mass of body i (i = 1,2...N), and $\ddot{x}_i$ is the x-direction acceleration vector of body i. M_{ij} and N_{ij} denote the "virtual" mass effects of body j on body i in the two directions of motion. The second derivative of y with respect to time represents the acceleration in the y-direction.

The terms M_{ij} are functions of the shape and size of the bodies (and the container boundary) and, most important, the size of the inter-body gaps. M_{ij} are *analytically* derived coefficients. Q_{x_i} represents the so-called Generalized force that may be an amalgam of all externally applied loads on the mass i in the x-direction. The above equilibrium equation for mass i in x-direction translational motions can be written for all degrees of freedom and for all masses. The resulting second order matrix differential equation contains a fully populated mass matrix (in contrast, dynamic equations *without* multi-body fluid coupling will have only diagonal non-zero terms).

The above exposition explains the inclusion of fluid coupling in a multi-body fluid coupled problem using a simplified planar motion case. This explanation provides the building blocks to explain the more complicated formulation needed to simulate freestanding racks. Dr. Paul's formulation is documented in a series of four reports written for PG&E in 1987. References [4] through [7], included herein as Attachments 1B, 1C, 1D, and 1E, contains the formulation information. The Paul multi-body fluid coupling theory conservatively assumes the flow of water to be irrotational (inviscid) and assumes that no energy losses (due to form drag, turbulence, etc.) occur. The USNRC personnel reviewed this formulation in the course of their audit of the Diablo Canyon rerack (ca. 1987) and subsequently testified in the ASLB hearings on this matter, as stated above.

While the ASLB, USNRC, and Commission consultants (Brookhaven National Laboratory and Franklin Research Center) all endorsed the Paul multi-body coupling model as an appropriate and conservative construct, the theory was still just a theory. Recognizing this perceptual weakness, Holtec and Northeast Utilities undertook an experimental program in 1988 to benchmark the theory. The experiment consisted of subjecting a scale model of racks (from one to four at one time in the tank) to a two-dimensional excitation on a shake table at a QA qualified laboratory in Waltham, Massachusetts.

The Paul multi-body coupling formulation, coded in QA validated preprocessors to DYNARACK, was compared against the test data (over 100 separate tests were run). The results, documented in Holtec Report HI-88243, were previously provided to the Commission. The experimental benchmark work validated Paul's fluid mechanics model and showed that the theoretical model (which neglects viscosity effects) is consistently bounded by the test data. This experimentally verified multi-body fluid coupling is the central underpinning of the DYNARACK WPMR solution that has been employed in every license application since Chinshan (1989). The DYNARACK 3-D WPMR solution has been found to predict much greater rack displacements and rotations than the previously used 3-D single rack results [8] (see Attachment 1F, included herein).

In general, the advance from linearized analyses (response spectrum) in the late 1970s to the single rack 3-D analyses until the mid-1980s and, finally, to the 3-D WPMR analysis in the past eleven years has, at each technology evolution stage, led to some *increase* in the computed rack response. The stresses and displacements computed by the DYNARACK 3-D WPMR analysis for the Vermont Yankee racks, in other words, may be larger (and therefore more conservative) than the docketed work on similar instances from 15 years ago. The conservatism's built into the WPMR solution arises from several simplifying assumptions explicitly intended to establish an upper bound on the results, namely:

i. In contrast to the single rack 3-D models, the fluid forces on every rack in the pool consist of the aggregate of fluid coupling effects from *all other* racks located in the pool. No assumptions on the motion of racks need be made a priori; the motion of each rack in the pool is a result of the analysis.

ii. The fluid coupling terms are premised on classical fluid mechanics; they are not derived from empirical reasoning. Further, fluid drag and viscosity effects, collectively referred to as "fluid damping", are neglected. In short, while the transfer of fluid kinetic energy to the racks helps accentuate their motion, there is *no* subtraction of energy through damping or other means.

iii. In the Vermont Yankee rack simulations, the dynamic model for the fuel assemblies in a rack assumes that *all* fuel assemblies within a rack move in unison. Work in quantifying the effect of discordant rattling of fuel assemblies within a rack in other licensing applications by Holtec has shown that the "unified motion" assumption exaggerates the rack response by 25% to 60%, depending on the rack geometry details and earthquake harmonics.

iv. The rack-to-rack and rack-to-wall gaps are taken as the *initial* nominal values. During the earthquake, these gaps will in fact change through the time-history duration. Strictly speaking, the fluid coupling matrix should be recomputed at each time-step with the concomitant gap distribution. The inversion of the mass matrix at each time-step (there are over four million time-steps in a typical WPMR run) would, even today, mandate use of a supercomputer. Fortunately, neglect of this so-called nonlinear fluid coupling effect is a conservative assumption. This fact is rigorously proven in a peer reviewed paper by Drs. Soler and Singh entitled "Dynamic Coupling in a Closely Spaced Two-Body System Vibrating in a Liquid Medium: The Case of Fuel Racks", published in 1982. The only docket where recourse to the nonlinear fluid coupling was deemed essential was Vogtle Unit 2 (in 1988) where the margin inherent in the nonlinear fluid effect, published in the above mentioned paper, was reaffirmed.

Nonlinear fluid coupling effects due to the use of current gaps at each time instant are not employed in this present application which imputes over 15% margin (in Holtec's analysts' estimate) in the computed rack response. In summary, the WPMR analysis utilizes a fluid coupling formulation that is theoretically derived (without empiricism) and experimentally validated. The assumptions built in the DYNARACK formulation are aimed to demonstrably exaggerate the response of all racks in the pool simulated in one comprehensive model. The DYNARACK fluid coupling model is neither approximate or empirical. A further elaboration of the details of the structural model used for the spent fuel racks and a mathematical explanation of the manner in which fluid coupling is considered in the solution is provided below.

DYNARACK, developed in the late 1970s and continuously updated since that time to incorporate technology advances such as multi-body fluid coupling, is a Code based on the Component Element Method (CEM). The chief merit of the CEM is its ability to simulate friction, impact, and other nonlinear dynamic events with accuracy. The high-density racks designed by Holtec International are ideally tailored for the CEM-based Code because of their honeycomb construction (HCC). Through the interconnection of the boxes, the HCC rack essentially simulates a multi-flange beam. The beam characteristics of the rack (including shear, flexure, and torsion effects) are appropriately modeled in DYNARACK using the classical CEM "beam spring". However, the rack is not rendered into a "stick" model, as implied by the staff's RAI. Rather, each rack is modeled as a prismatic 3-D structure with support pedestal locations and the fuel assembly aggregate locations set to coincide with their respective C.G. axes. The rattling between the fuel and storage cells is simulated in exactly the same manner as it would be experienced in nature: namely, impact at any of the four facing walls followed by rebound and impact at the opposite wall. Similarly, the rack pedestals can lift off or slide as the instantaneous dynamic equilibrium would dictate throughout the seismic event. The rack structure can undergo overturning, bending, twist, and other dynamic motion modes as determined by the interaction between the seismic (inertia) impact, friction, and fluid coupling forces. Hydrodynamic loads, which can be quite significant, are included in a comprehensive manner, as we explain in more detail below.

As explained in the foregoing, the fluid coupling effect renders the mass matrix into a fully populated matrix. Modeling the fuel rack as a multi-degree of freedom structure, the following key considerations are significant:

i. Over 70% of the mass of the loaded rack consist of fuel assemblies, which are unattached to the rack, and resemble a loose bundle of slender thin-walled tubes (high mass, low frequency).

ii. In honeycomb construction (HCC) racks, as shown in a 1984 ASME paper [9], the rack behaves like a stiff elongated box beam (End Connected Construction racks, built 20 years ago and now obsolescent behave as a beam and bar assemblage).

Since the racks under inertial loading have overall structural characteristics of a multi-flange beam, it is computationally wasteful (and, as we explain later, numerically hazardous) to model such a structure as a plate assemblage. The DYNARACK dynamic model preserves the numerical stability of the physical problem by representing the rack structure by an equivalent flexural and shear resisting "component element" (in the terminology of the Component Element Method [10]).

A detailed discussion of the formation of the fluid mass matrix is presented below.

The problem to be investigated is shown in Figure RAI 1.1, which shows an orthogonal array of sixteen rectangles which represent a unit depth of the sixteen spent fuel racks in the Vermont Yankee Spent Fuel Pool. The rectangles are surrounded by narrow fluid filled channels whose width is much smaller than the characteristic length or width of any of the racks. The spent fuel pool walls are shown enclosing the entire array of racks.

The dimensions of the channels are such that an assumption of uni-directional fluid flow in a channel is an engineering assumption consistent with classical fluid mechanics principles.

We consider that each rectangular body (fuel rack) has horizontal velocity components U and V parallel to the x and y axes, and that the channels are parallel to either the x or y axes. The pool walls are also assumed to move.

We conservatively assume that the channels are filled with an inviscid, incompressible fluid. Due to a seismic event, the pool walls, and the spent fuel racks are subject to inertia forces that induce motion to the rectangular racks and to the wall. This motion causes the channel widths to depart from their initial nominal values and causes flow to occur in each of the channels. Because all of the channels are connected, the equations of classical fluid mechanics can be used to establish the fluid velocity (and hence, the fluid kinetic energy) in terms of the motion of the spent fuel racks.

For the case in question, there are 40 channels of fluid identified. Figure RAI 1.2 shows a typical rack (box) with four adjacent boxes and fluid and box velocities identified. The condition of vanishing circulation around the box may be expressed as

$$\Gamma = \oint_c v_s ds = 0$$

or

$$\int_{-a/2}^{a/2} (u_B - u_T) d\xi + \int_{-b/2}^{b/2} (v_R - v_L) d\eta = 0$$

where the subscripts (L, R, B, T) refer to the left, right, bottom, and top channels, respectively; ξ, η are local axes parallel to x and y, and u, v are velocities parallel to ξ, η.

Continuity within each channel gives an equation for the fluid velocity as

$$w = w_m - (\frac{\dot{h}}{h})\, s$$

where w represents the velocity along the axis of a channel, w_m represents the mean velocity in the channel, s is either ξ or η, and $\dot{h}$ is the rate of increase of channel width. For example,

$$\dot{h}_R = U_R - U$$

From Figure RAI 1.2, four equations for u_B, u_T, v_R, v_L in terms of the respective mean channel velocities, can be developed so that the circulation equation becomes

$$a\,(U_{Bm} - U_{Tm}) + b\,(\nu_{Rm} - \nu_{Lm}) = 0$$

One such circulation equation exists for each spent fuel rack rectangle. We see that the velocity in any channel is determined in terms of the adjacent rack velocities if we can determine the mean fluid velocity in each of the 40 channels. Circulation gives 16 equations. The remaining equations are obtained by enforcing continuity at each junction as shown in Figure RAI 1.3. Enforcing continuity at each of the 25 junctions gives 25 equations of the general form

$$\Sigma\, h\, \sigma\, w \frac{1}{2} \Sigma\, L\, \dot{h}$$

where w is the mid-length mean velocity in a connecting channel of length L and $\dot{h}$ is the relative normal velocity at which the walls open. The summation covers all channels that meet at the node in question. The sign indicator $\sigma = \pm 1$ is associated with flow from a channel either into or out of a junction.

Therefore, there are a total of 25 + 16 = 41 equations which can be formally written; one circulation equation, however, is not independent of the others and reflecting the fact that the sum total of the 25 circulation equations must also equal zero, representing circulation around a path enclosing all racks. Thus, there are exactly 40 independent algebraic equations to determine the 40 unknown mean velocities in this configuration.

Once the velocities are determined in terms of the rack motion, the kinetic energy can be written and the fluid mass matrix identified using the Holtec QA-validated pre-processor program CHANBP6. The fluid mass matrix is subsequently apportioned between the upper and lower portions of the actual rack in a manner consistent with the assumed rack deformation shape as a function of height in each of the two horizontal directions. This operation is performed by the

Holtec QA-validated pre-processor code VMCHANGE. Finally, structural mass effects and the hydrodynamic effect from fluid within the narrow annulus in each cell containing a fuel assembly between fuel and cell wall is incorporated using the Holtec QA-validated pre-processor code MULTI155.

The initial inter-rack and rack-to-wall gaps are provided in Figure RAI 1.2. These gaps, which directly figure in the computation of fluid mass effects in fluid coupling matrix, are assumed to apply for the entire duration of the earthquake. In reality, the gaps change throughout the seismic event and a rigorous analysis would require that the mass matrix be recomputed at every time-step. Besides being numerically impractical, such refinement in the solution would reduce the conservatism in the computed results, as discussed earlier.

The time variations in the inter-rack and rack-to-wall gaps are, however, tracked for the duration of the earthquake. Closure of any gap at any location results in activation of the compression gap spring at that location. The loads registered in the gap spring quantify the collision force at that location. The fuel-to-storage cell rattling forces and rack pedestal-to-pool liner impact forces (in the event of pedestal lift-off) are typical examples of collision forces that are ubiquitous in rack seismic simulations. The nonlinear contact springs in DYNARACK simulate these "varying gap" events during seismic events using an unconditionally convergent algorithm.

In summary, the Whole Pool Multi-Rack (WPMR) analysis is a geometrically nonlinear formulation in all respects (lift-off, sliding, friction, impact, etc.), except in the computation of the fluid coupling matrix, which is based on the nominal (initial) inter-body gaps.

The modeling technique used (i.e. representation of the fuel rack and contained fuel by elastic beams and appropriate lumped masses) was chosen based on the application Codes, Standards and Specifications given in Section IV (2) of the NRC guidance on spent fuel pool modifications entitled, "Review and Acceptance of Spent Fuel Storage and Handling Applications," dated April 14, 1978. This reference states that "Design…may be performed based upon the AISC specification or Subsection NF requirements of Section III of the ASME B&PV Code for Class 3 component supports." The rack modeling technique is consistent with the linear support beam-element type members covered by these codes.

It is agreed that finite element models could be developed using plate and fluid elements, which may also provide satisfactory simulated behavior for a single rack. However, there is no known commercial finite element code which can treat multi-body fluid interaction correctly and sufficiently account for near and far field fluid effects involving many bodies (racks) in a closed pool. It is for this reason that the global dynamic analysis uses the formulation specifically developed and contained within DYNARACK.

Response 1b

Holtec Report HI-88243 by Dr. Burton Paul reports comparison of DYNARACK fluid coupling formulation with over 100 experiments carried out in an independent laboratory under a

10CFR50 Appendix B program. These tests were performed with the *sole* purpose of validating the multi-body fluid coupling formulation based on Kelvin's recirculation theorem in classical fluid mechanics. These experiments, to our knowledge, are the only multi-body fluid coupling tests conducted and recorded under a rigorous QA program. The participating bodies used in the tests were carefully scaled to simulate rectangular planform fuel racks. The tests were run with a wide range of seismic frequencies to sort out effects of spurious effects such as sloshing in the tank, and to establish that the fluid coupling matrix is (ought to be) independent of the frequency content of the impressed loading.

The University of Akron tests [3] performed some testing under the sponsorship of the predecessor company of U.S. Tool & Die, Inc. However, these tests were performed in the time when racks were still being analyzed using the Response Spectrum Method. We note that a theoretical model developed by Scavuzzo [2] is *exactly* that used in the Holtec WPMR analysis when the Holtec mass matrix is reduced to a single rectangular solid block surrounded by four rigid (pool) walls. That is, the work by Scavuzzo is a special case of a Holtec WPMR analysis for a spent fuel pool containing a single spent fuel rack.

The Holtec WPMR fluid mass matrix for many racks in the pool is obtained by applying the same classical principles of fluid continuity, momentum balance, and circulation, to a case of many rectangular bodies in the pool with multi-connected narrow fluid channels.

The experimental work performed by Scavuzzo, et al., does not attempt to model a free standing rack since many rack structures of that vintage were not free-standing. The experimental test is equivalent to a single spring-mass-damper subject to a forced harmonic oscillation while submerged. If one accepts the fact that the fluid model used by Scavuzzo is a limiting case of the more general Holtec formulation, then the good agreement of theory with experiment for the single "rack" modeled experimentally serves as additional confirmation that the Holtec theoretical hydrodynamic mass model, which is identical to the Scavuzzo model (for a simple rack) is reproducible by experiment.

We have utilized the data supplied by Scavuzzo to simulate the experiment using the pre-processor CHANBP6 and the solver DYNARACK. The results of this comparison have been incorporated into the Holtec validation manual for DYNARACK (HI-91700) as an additional confirmation of the fluid coupling methodology. This validation manual, along with additional supporting documentation and discussions, was presented to the NRC in April, 1992 under dockets 50-315 and 50-316 for the D.C. Cook station and also was submitted in the licensing for reracking of the Waterford 3 spent fuel pool. The submittal for Waterford contained the evaluation of the Scavuzzo theory and experiment and demonstrated that the WPMR general formulation was in agreement with the experimental work presented in [2].

Response 1c

Rack "CA" geometry is used to compute the requested rack stiffnesses. The DYNARACK computer code requires stiffness inputs in the x, y and vertical direction in order to perform the dynamic analysis.

Attachment 1G provides the calculations to develop the bending spring constants and the extensional stiffness appropriate to the directions requested. Attachment 1H provides similar calculations for the shear springs and the torsional spring based on the methods presented in the Holtec Position Paper WS-126 which is contained in Attachment 1J. The following table summarizes the results calculated in Attachments 1G and 1H.

SPRING CONSTANTS FOR RACK "CA"	
Item	**Value**
Bending spring associated with y deformation (rotation about x-x axis) (in.-lb)	4.614×10^{10}
Bending spring associated with x deformation (rotation about y-y axis) (in.-lb)	2.511×10^{10}
Extension spring vertical direction (in.-lb)	3.958×10^{7}
Shear spring associated with y deformation (lb./in.)	3.426×10^{6}
Shear spring associated with x deformation (lb/in.)	3.0×10^{6}
Torsion spring associated with twisting (in.-lb)	5.079×10^{8}

Response 1d

The requested data, which has been excerpted from Holtec International Report HI-981932, is provided in attachment 1K.

Response 1e

Holtec Report HI-981969 provides the details of the development of the time histories used for the Vermont Yankee pool from the design basis Response Spectra. Figures RAI 1.4-1.9, reproduced from that report, demonstrate the required enveloping of the target PSD, over the frequency range important to spent fuel racks (3-7Hz) by the PSD regenerated from the developed time histories.

References:

1. Deleted.

2. Scavuzzo, R.J., et al, "Dynamics Fluid Structure Coupling of Rectangular Modules in Rectangular Pools", ASME Publication PVP-39, 1979, pp. 77-87.

3. Radke, Edward F., "Experimental Study of Immersed Rectangular Solids in Rectangular Cavities," Project for Master Science Degree, The University of Akron, Ohio, 1978.

4. "Evaluation of Fluid Flow for In-Phase and Out-of-Phase Rack Motions", by B. Paul, Holtec Report HI-98113, April 1987.

5. "Study of Non-Linear Coupling Effects", by A.I. Soler, Holtec Report HI-87102.

6. "Estimated Effects of Vertical Flow Between Racks and Between Fuel Cell Assemblies", by B. Paul, Holtec Report HI-87114, April 1987.

7. "Fluid Flow in Narrow Channels Surrounding Moving Rigid Bodies", by B. Paul, Holtec Report HI-87112.

8. "Chin Shan Analyses Show Advantages of Whole Pool Multi-Rack Approach", Nuclear Engineering International, March 1991.

9. "Seismic Response of a Free Standing Fuel Rack Construction to 3-D Floor Motion", Nuclear Engineering and Design, 1984.

10. United States Patent No. 4,382,060, "Radioactive Fuel Cell Storage Rack", May 3, 1983.

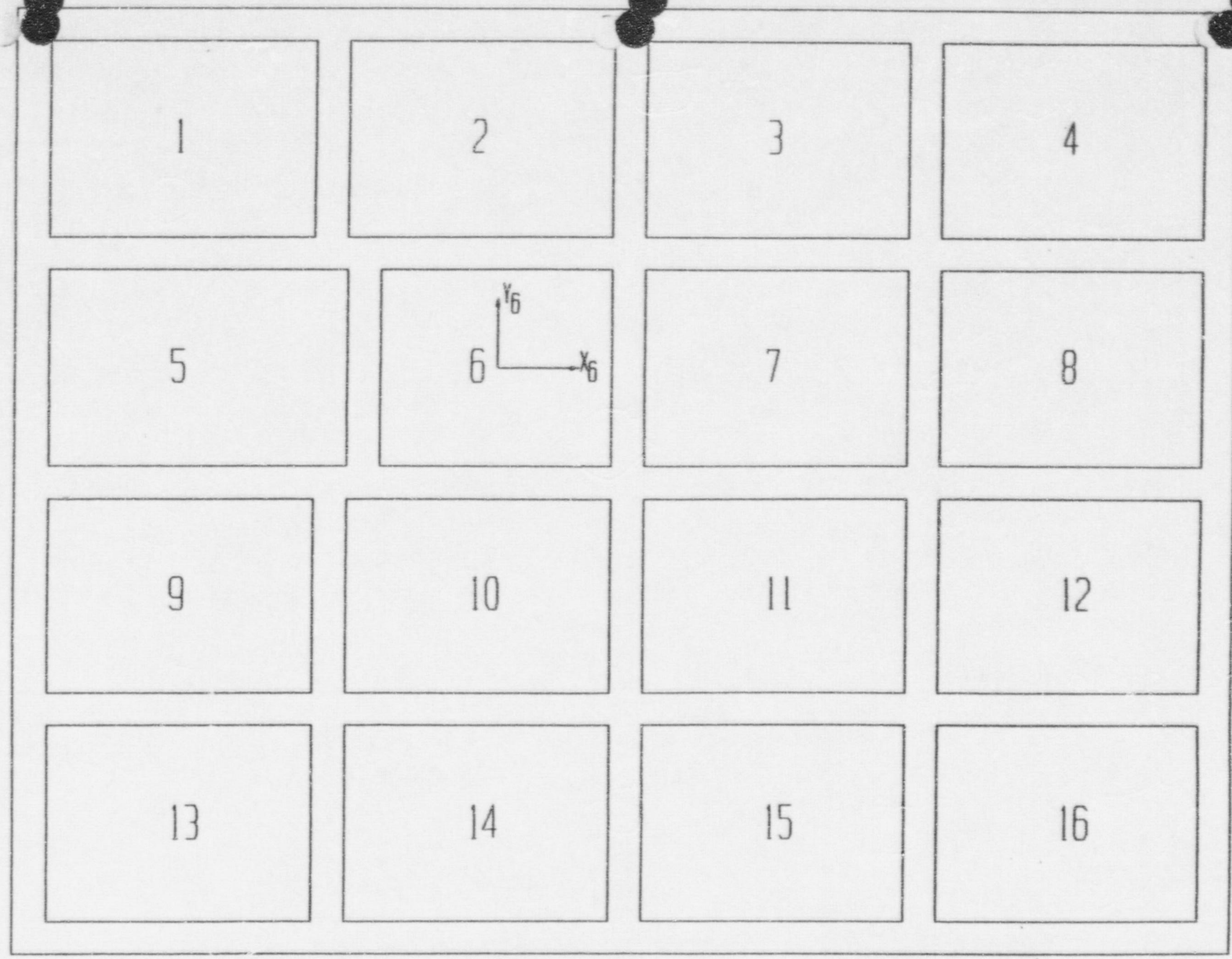


Figure RAI 1.1 PLANAR VIEW OF A 16 RACK ARRAY

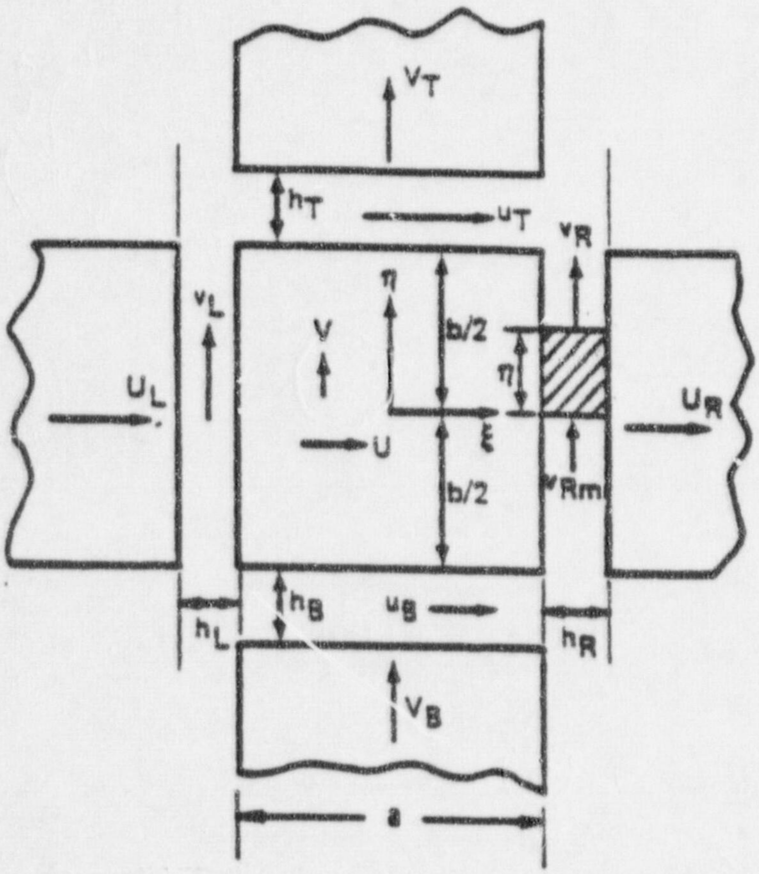


Figure RAI 1.2 FLOWS AROUND A TYPICAL CELL

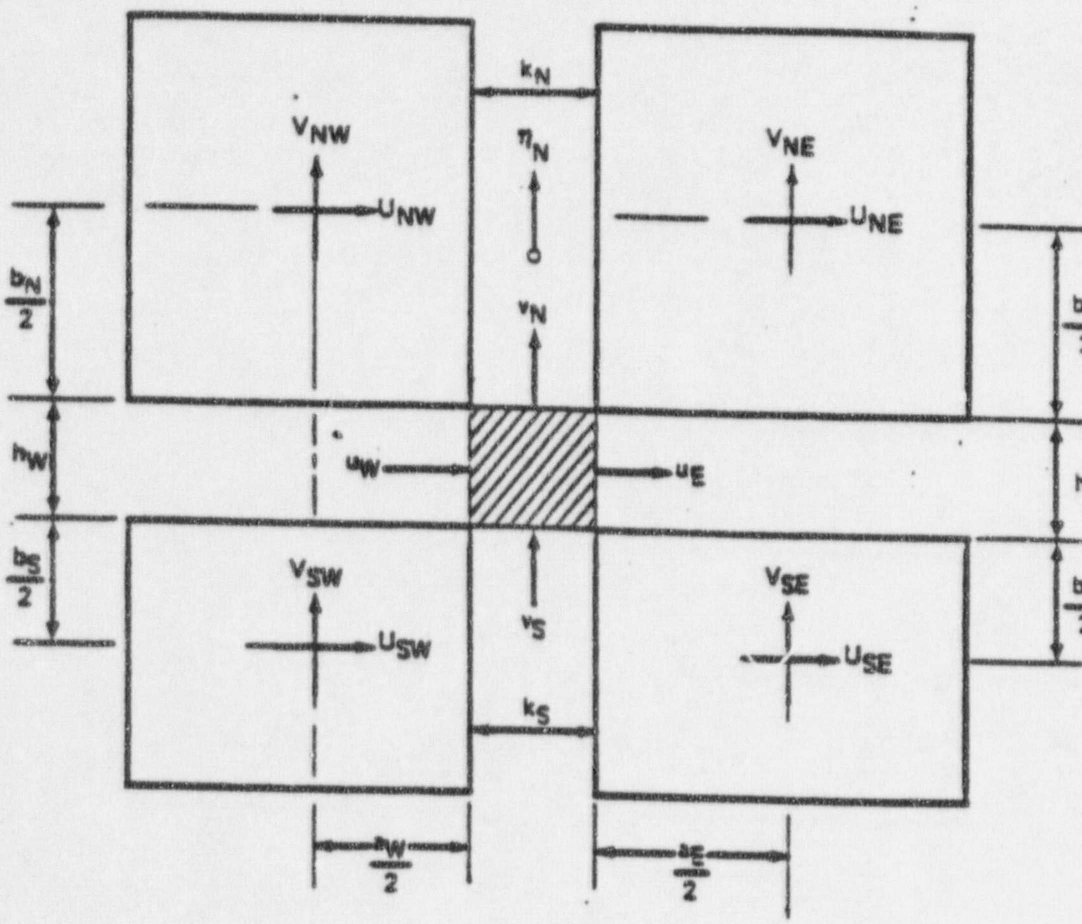
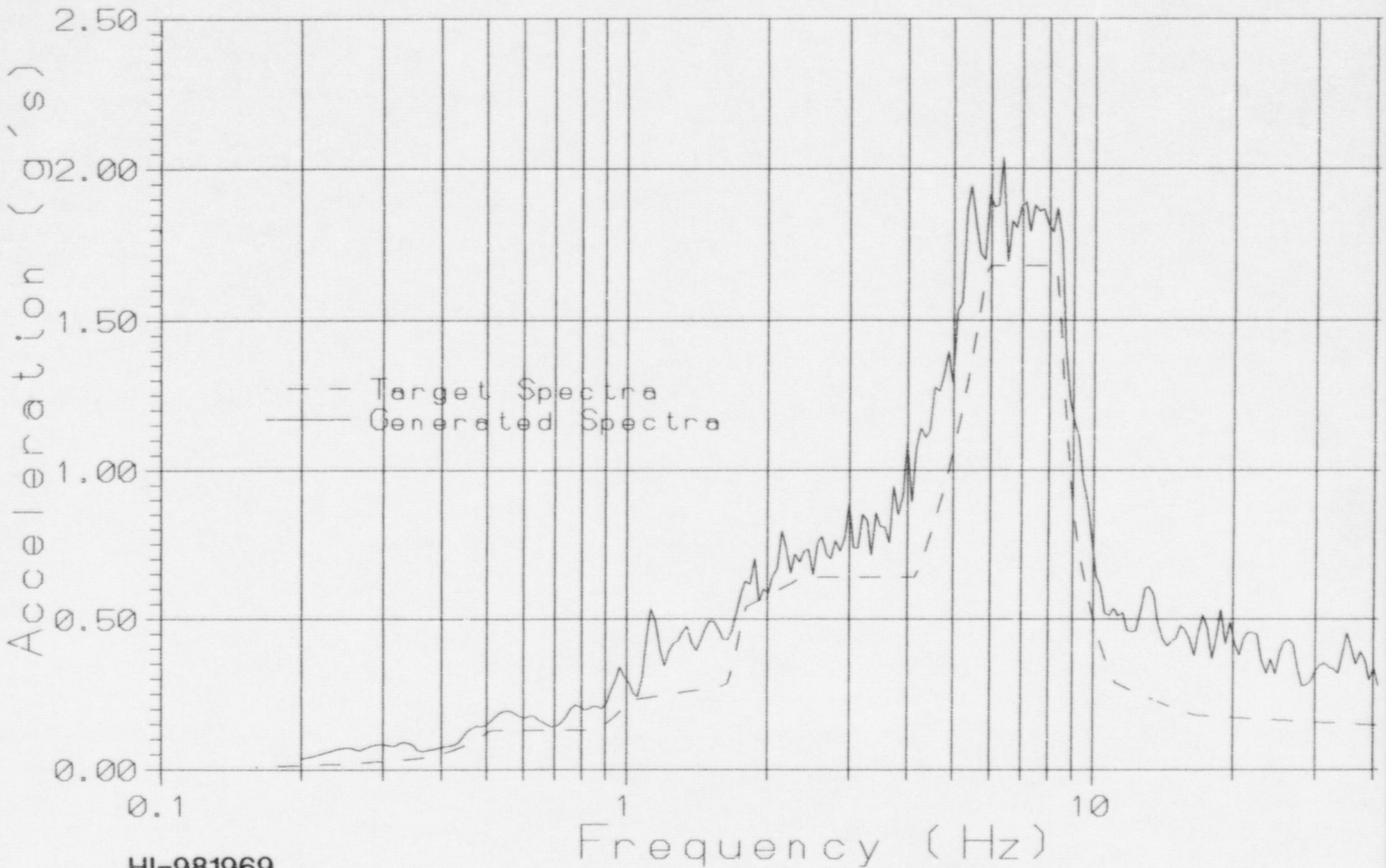


Figure RAI 1.3

FLOWS AT A CHANNEL INTERSECTION

Figure RAI 1.4 Vermont Yankee SFP Response Spectrum OBE-X direction (1% Damping)



HI-981969

Figure RAI 1.5

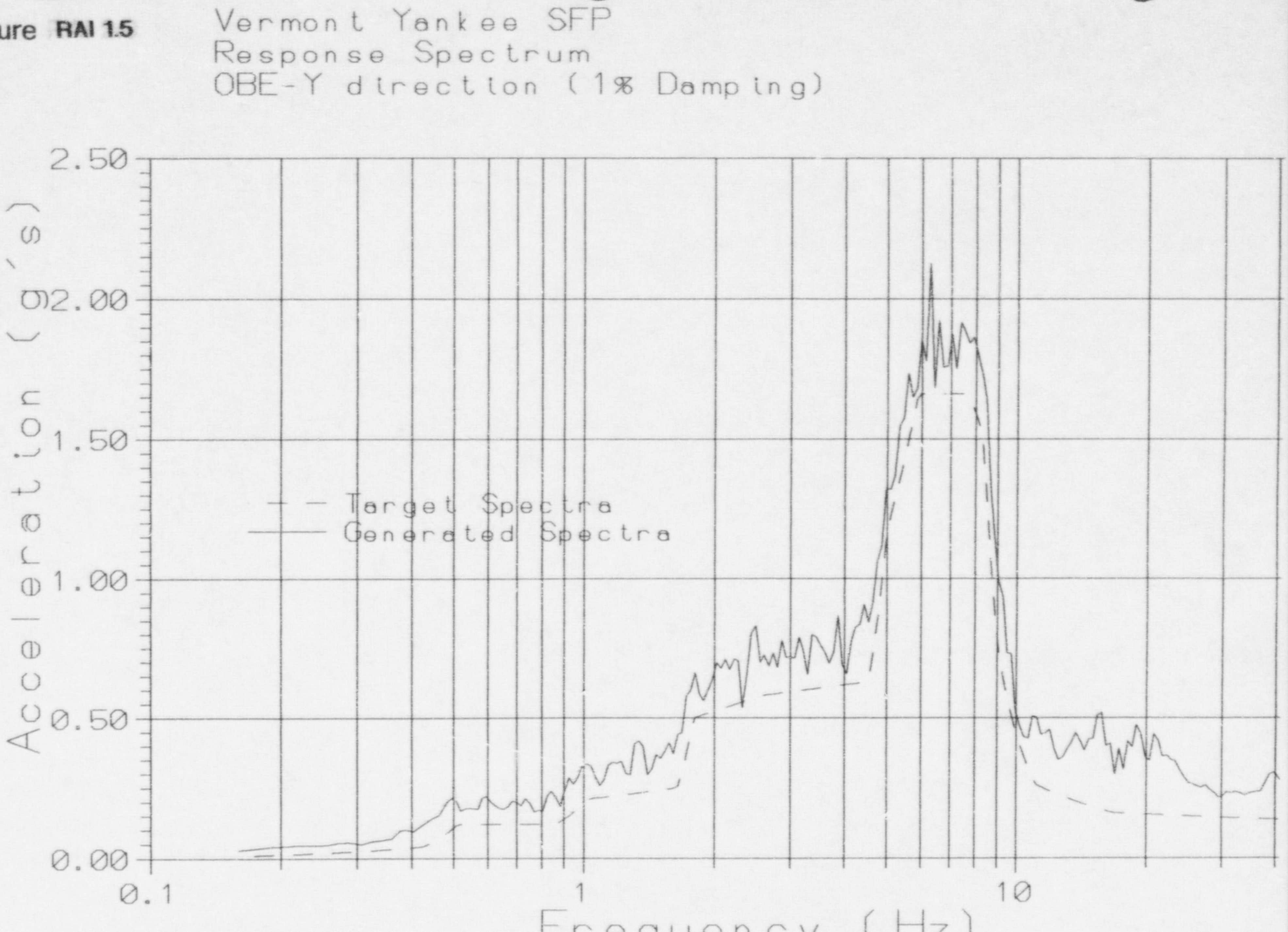


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Figure RAI 1.6

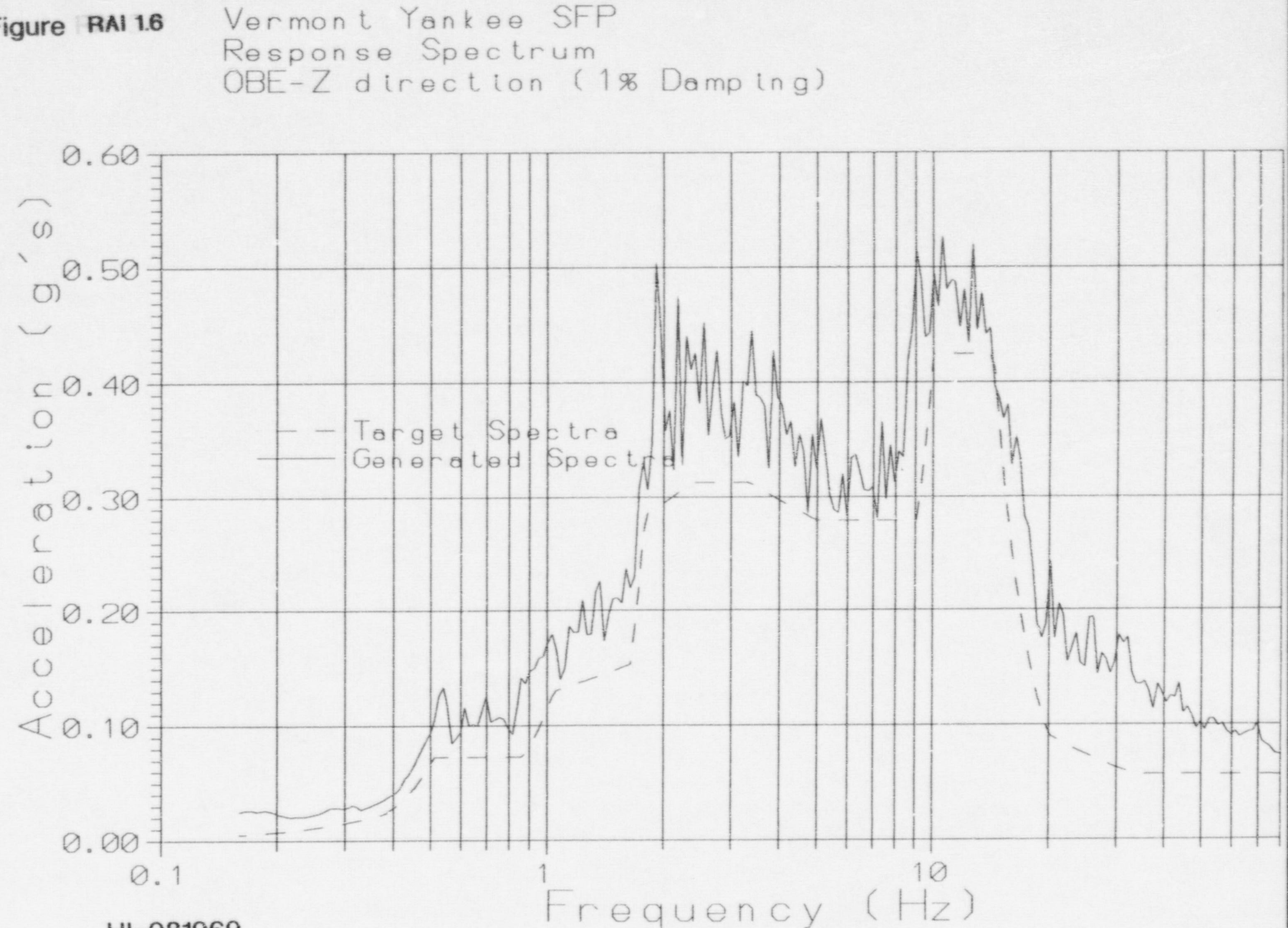

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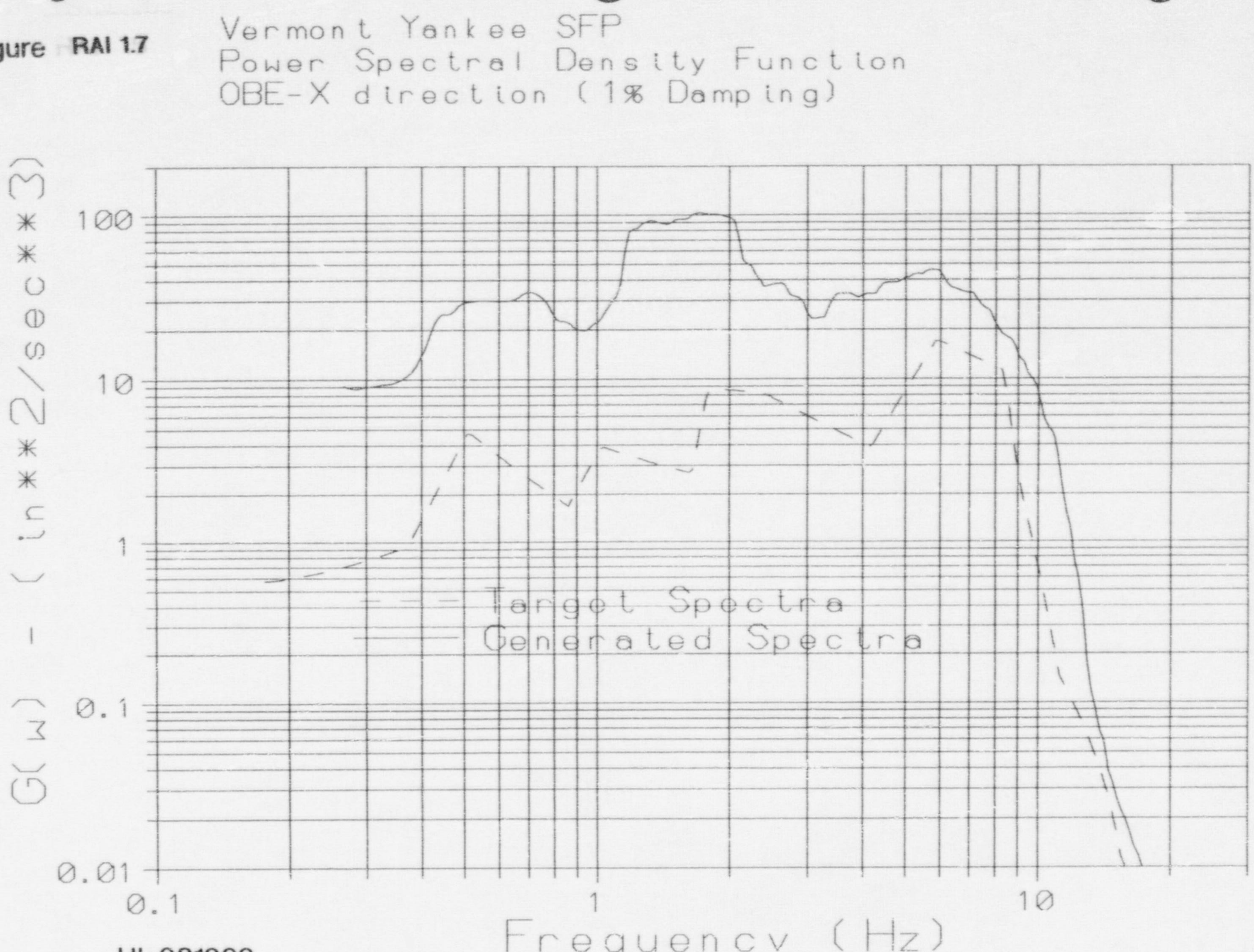
Figure RAI 1.7



HI-981969

Figure RAI 1.8

Vermont Yankee SFP
Power Spectral Density Function
OBE-Y direction (1% Damping)

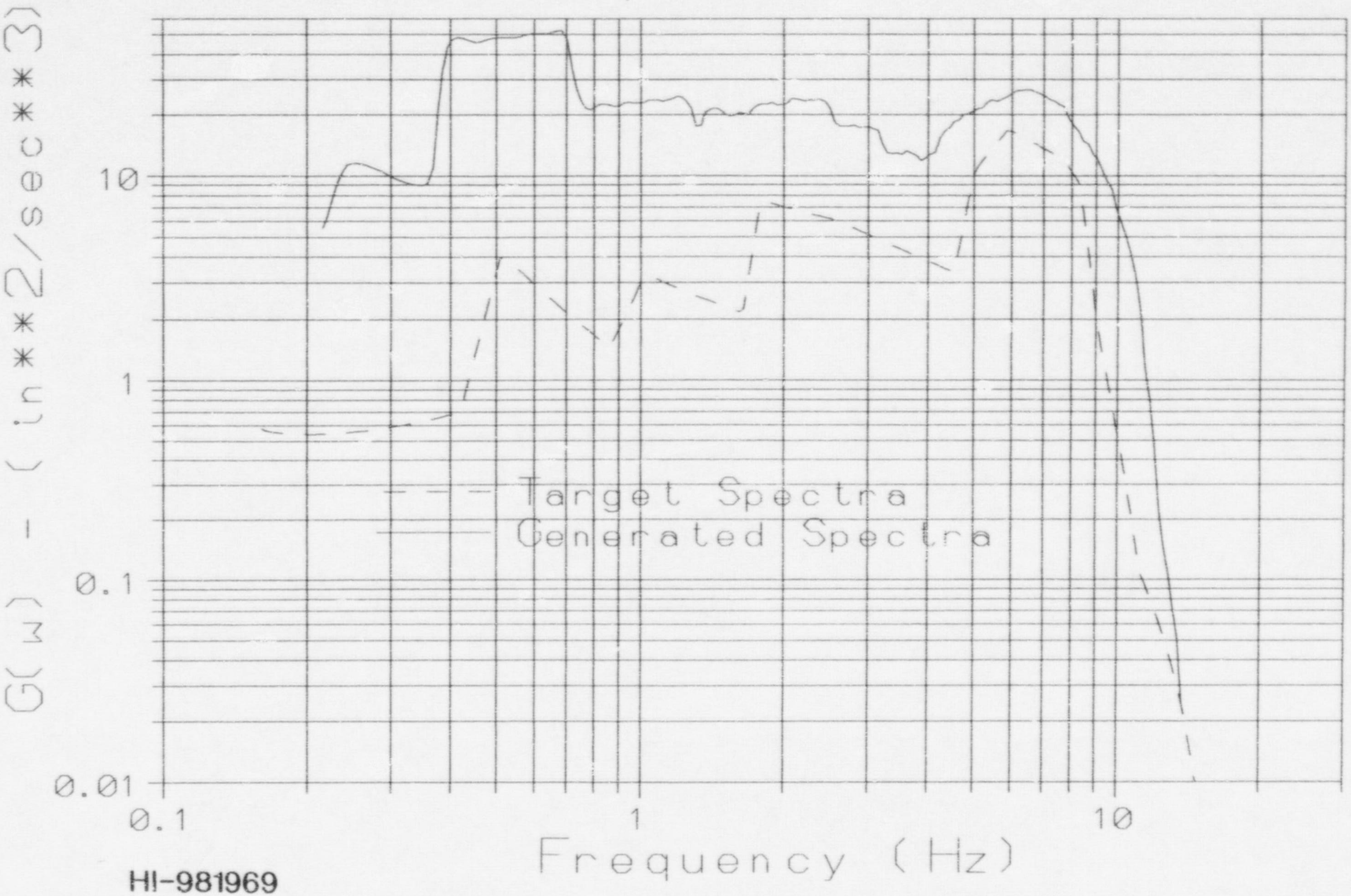
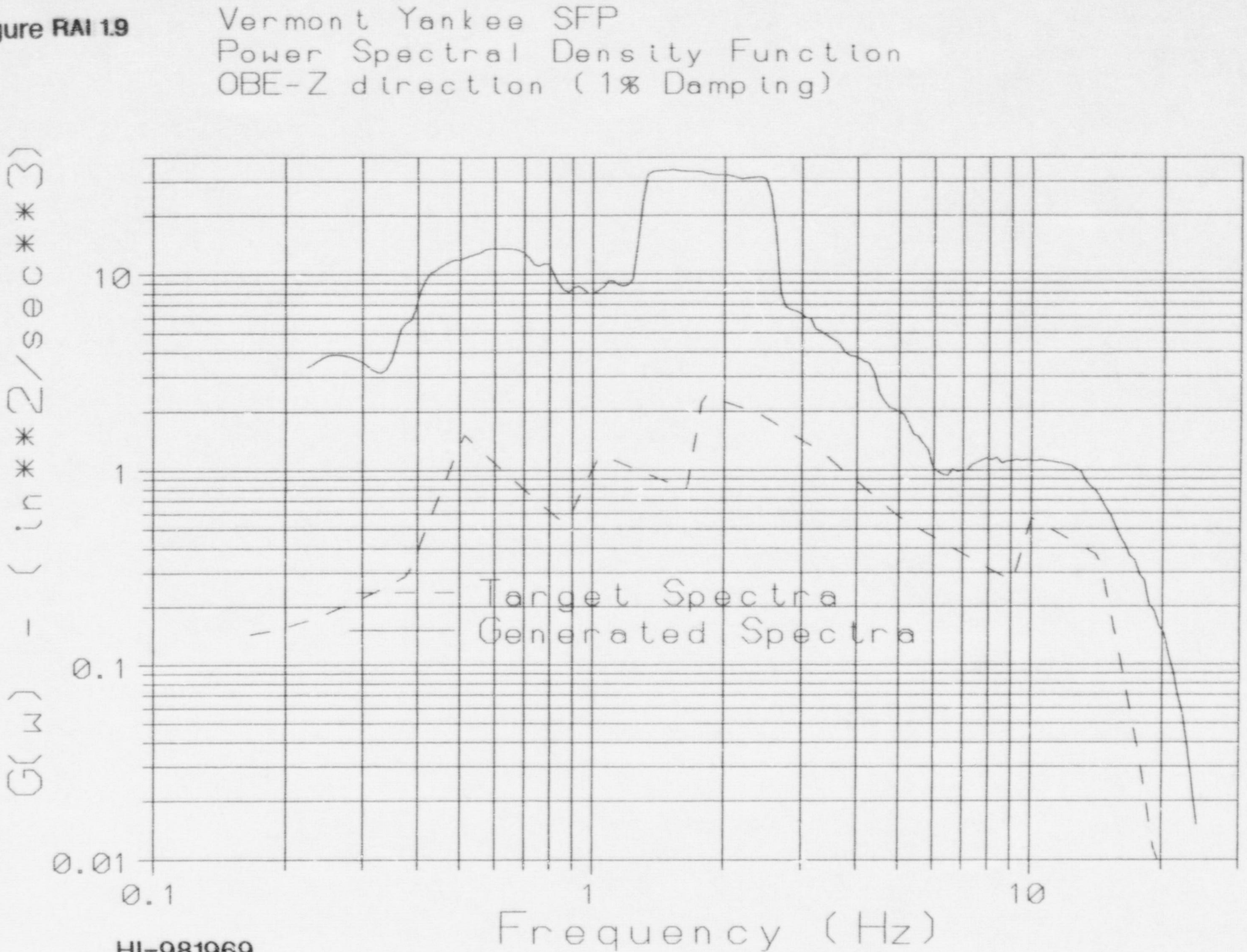


Figure RAI 1.9



HI-981969

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ATTACHMENT 1F

outside of the gamma shield. The neutron shield is enclosed by a thin outer shell. In addition to containing the resin, the aluminum also provides a conduction path for heat transfer from the cask body to the outer shell.

The cask is sealed using redundant metallic seals. The cask cavity is pressurized above atmospheric pressure with helium to preclude air inleakage in the event of seal failure.

The casks will be about 16.5ft tall and 8.5ft wide. A fully loaded cask will weigh around 120t. Four trunnions are attached to the cask body for lifting, tiedown and rotation. Two of the trunnions are located near the top of the body and two near the lower end. The lower trunnions may be used for rotating the unloaded cask between vertical and horizontal positions.

The long term cask surface temperature has been calculated as being 191°F. The short term temperature on hot, sunny days has been calculated as 233°F. The maximum external contact dose rate is estimated to be about 60mrem/h, with the dose rate at any location accessible to the public well within the allowable limits.

LICENSING

Work on the project began last autumn. The completion date depends largely on the licensing process. A safety-analysis report has been submitted by Northern States Power to the US NRC, whose review is expected to be completed around mid-April.

Chin Shan analyses show advantages of whole pool multi-rack approach

By K P Singh and A I Soler

Results from whole pool multi-rack (WPMR) analyses at Chin Shan and Oyster Creek point up the potential inadequacies of single rack 3D analyses, and show just how important it is to carry out WPMR simulations, despite their abstruseness and high cost.

Fuel storage racks are essentially thin-walled, cellular structures of prismatic cross-section. Although the details of design vary from one supplier to another, certain key physical attributes are common to all designs. For example, all racks feature square cells of sufficient opening size and height to enable insertion and withdrawal of the fuel assembly.

The cells (or "boxes") are arranged in a square (or rectangular) pattern and are fastened to each other using suitable connectors and welds. The array of cells is positioned in a vertical orientation and is supported off the pool slab surface by four or more support legs. The spent fuel pool is filled with the individual fuel racks. The plenum created by the support legs is essential for proper cooling of the fuel assemblies stored in the rack, which relies on natural convective cooling to extract the heat emitted by the spent fuel. However, it has the insalutary effect of making it kinematically less stable. Regulatory authorities require careful and comprehensive analysis of the response of the racks under the seismic motions postulated for the pool slab.

Non-linear structure. Such an analysis cannot be conducted in the manner of conventional structural analyses for power plants, because the classical approaches (viz the response spectrum method) are predicated on the assumption that the structure is linear. A fuel rack, however, is the epitome of a non-linear structure (defined as one in which the applied force does not have a linear relationship to the resulting displacement). The stored fuel assemblies, which constitute over 60 per cent of the weight of a fully loaded rack module, are free to rattle inside the storage cell during a seismic event. The rack module itself is not attached to the fuel pool slab. Furthermore, the Coulomb friction resisting the sliding of the rack module on the pool surface is, by definition, a non-linear force.

TIME INTEGRATION TECHNIQUES

In recognition of these highly non-linear attributes of the dynamic behaviour of fuel storage racks, their seismic simulation has been carried out using time integration techniques. The state-of-the-art analysis technique involves modelling a single rack module as a 3D structure with features to capture the fuel assembly rattling, module sliding, rocking and twisting motions.

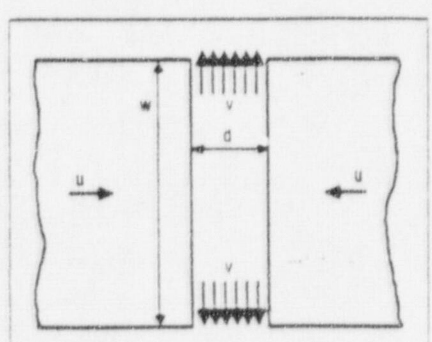
Despite the versatility of the 3D seismic model, the accuracy of the single rack simulations has been suspect due to one key element: namely, hydrodynamic participation of water around the racks. This effect is understood by considering the motion of water between large flat planes of width w at a (small) distance d apart, which are moving towards each other with velocity u. The moving planes, for simplicity of this illustration, are assumed to be infinitely long, such that the motion of water exiting the inter-plane space is in the plane of the page in the diagram below. For this geometry, the velocity of water v is computed by direct volume balance (continuity):

In a time interval dt,

$$w\,(2u)(dt) = 2vd\,(dt)$$

or $v/u = w/d$

This leads to the conclusion that the velocity of water exiting the fluid gap is w/d times the velocity of the approach plane. In a typical spent fuel pool, the racks are about 250cm (100in) wide, and are spaced at 4.5-7.5cm (2-3in)



▲ Two submerged parallel flat planes approaching each other.

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intervals. For the rack modules arranged in a typical spent fuel pool, w = 100in and d = 2in, so v/u = 50. Since kinetic energy is proportional to the square of velocity, the water exiting the inter-rack space will have 2500 times the specific kinetic energy of the moving rack.

This hydraulic energy is either drawn from or added to the moving rack, modifying its submerged motion in a significant manner. The dynamics of one rack, therefore, affects the motion of all others in the pool. A dynamic simulation which treats only one rack, or a small grouping of racks, therefore, is intrinsically inadequate to predict the motion of rack modules with any quantifiable level of accuracy.

EXPERIENCE IN TAIWAN

Taiwan — no stranger to seismic tremors — has three nuclear installations: Kuosheng, Maanshan and Chin Shan. Taiwan Power Company procured racks for the Chin Shan site from General Electric Company in 1986. These racks are of the so-called honeycomb construction, and were initially analysed by a single rack 2D seismic model. Recognizing the inadequacy of such a model to prognosticate the potential hazard of rack-to-rack (or rack-to-pool wall) collisions during a severe seismic event, Taiwan Power set out to determine the response to racks by a comprehensive whole pool analysis.

Under a consulting contract with Taiwan Power, Holtec International (USA) undertook to prepare a dynamic model of the entire assemblage of racks (a total of 14 modules) in the pool, with due consideration of fluid coupling effects. Holtec's code DYNARACK, which uses the component element method for non-linear dynamic analysis, and has been used in over a dozen fuel rack licensing projects, was used for this purpose.

The results of this first ever so-called whole pool multi-rack (WPMR) analysis provided further insight into the in-pool rack dynamic behaviour. Tracking of the inter-rack gap showed that the presence of water has the effect of injecting a certain symmetry into the motion of adjacent racks, although a certain amount of out-of-phase motion occurs.

Comparison with single rack 3D analyses, however, pointed to the rather unsettling conclusion that the single rack models do not bound the results of the whole pool simulations. In the Chin Shan analysis, the WPMR analyses yielded a maximum kinematic displacement of a rack in the pool — 8.5 times the single rack analysis prediction. The impact loads between rack support pedestals and the pool slab decreased slightly from the values obtained from the single rack analysis. In the Chin Shan analysis, the coefficient of friction between pedestal and slab was about 0.2. Even though the rack displacements relative to the slab showed a large increase over the single rack results, no rack-to-rack or rack-to-wall impacts were predicted.

OYSTER CREEK ANALYSES

Subsequent to the Chin Shan analysis, Holtec International completed some similar work for GPU Nuclear's Oyster Creek plant located near Toms River, New Jersey. The Oyster Creek analyses were performed using coefficients of friction of 0.2 and 0.8. In this case, the maximum displacement of any rack in the pool predicted by the WPMR analyses was 1.4 times the single rack analysis prediction. In this analysis, the pedestal to slab impact loads predicted by the WPMR analyses are slightly greater than the values obtained from the single rack analysis.

New storage technology at Greifswald

By W Fischer, S Standke, M Lein and K Hochstrate

The Greifswald site in the former GDR boasts a large interim fuel store (as well as four now-shut-down VVER-440s and four more in various stages of construction). In recent years the east Germans have been working towards expanding the capacity of the store by re-racking the ponds using locally-developed transport/storage baskets.

In 1985, the arrangements for shipping spent fuel from Rheinsberg (1 × VVER-70) and Greifswald (4 × VVER-440s) in eastern Germany back to the USSR were changed. Previously, the spent fuel had been returned after cooling for three years at the power stations. To meet the Soviet requirement for a five-year cooling time, the interim storage facility ZAB (Zwischenlager fur abgebrannte Brennstabe) was built at Greifswald.

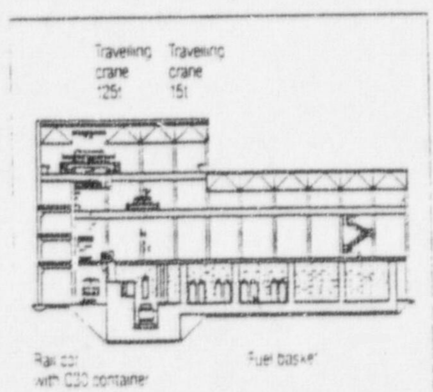
ZAB was designed by the Soviets, and the first stage has been built to hold 4500 undamaged spent fuel assemblies (550t heavy metal (HM)). The assemblies are stored in three ponds, each accommodating 52 baskets, with some specially designed baskets to accommodate assemblies with failed elements. A fourth pond acts as a reserve.

At the end of 1990, ZAB contained around 2400 spent fuel assemblies from the VVER-440s and the VVER-70. It is planned that a further 2300 fuel assemblies stored in Greifswald units 1 to 4 will be transferred to ZAB after a cooling time of three years. This transfer is to be performed using the TK-6 rail transporter developed at the German Fuel Institute, Freiberg. The TK-6 meets national and international transport regulations for Type B packages.

Soviet-developed [illegible] fuel baskets serve both as inserts for the TK-6 cask and as storage racks in the ZAB ponds. This avoids the need to handle single fuel assemblies in the ZAB facility. Each [illegible] basket contains 30 fuel assemblies (3.6tHM). The criticality safety of the [illegible] basket is achieved by having a centre-to-centre distance of 225mm between the fuel assemblies.

NEW BASKET

Work has been in progress to expand the



▲ Cross-section of the ZAB interim spent fuel store at Greifswald in eastern Germany.

W Fischer and S Standke are with Energiewerke Nord, Betrieb Bruno Leuschner [illegible] Greifswald, Germany. M Lein and K Hochstrate are with [illegible] Germany.

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ATTACHMENT 1G

ATTACHMENT 1G

Stiffness Calculations

The purpose of this document is to calculate the rack bending and extension spring constants, which characterize the Holtec fuel racks for Vermont Yankee Nuclear Station. The formulas are obtained from an article published in Nuclear Engineering and Design, which is titled "Seismic Response of a Free Standing Fuel Rack Construction to 3-D Floor Motion." The calculated values are used as inputs to the 22-DOF single rack analysis.

The following rack data, which is specific to rack "CA", are used in the calculations:

$b := 6.00 \cdot in$	Cell inner dimension
$t := 0.075 \cdot in$	Cell wall thickness (minimum)
$p := 6.218 \cdot in$	Cell pitch
$N_x := 14$	Number of storage cells in the x direction
$N_y := 19$	Number of storage cells in the y direction
$H := 168 \cdot in$	Height of the storage cells
$E := 27.6 \cdot 10^6 \cdot psi$	Young's modulus of cell material

First, the area and moment of inertia properties of the gross cross section are calculated. For a BWR fuel rack, any two adjacent storage cells share one wall. Therefore, the determination of the storage cell metal area and moment of inertia (per cell) are based on an effective cell wall thickness equal to one half of the actual wall thickness.

$t_e := \frac{t}{2}$ $\quad t_e = 0.037 \cdot in$ $\quad$ Effective Cell Wall Thickness

The cross sectional area and moments of inertia of a single storage cell are

$A_{cell} := 4 \cdot (b + t_e) \cdot t_e$ $\quad A_{cell} = 0.906 \cdot in^2$ $\quad$ Area of Single Cell

$I_{cell} := \frac{1}{12} \left[(b + 2 \cdot t_e)^4 - b^4 \right]$ $\quad I_{cell} = 5.502 \cdot in^4$ $\quad$ Moment of Inertia of Single Cell

The cross sectional area and moments of inertia of the gross cell structure are

$$A_{rack} := N_x \cdot N_y \cdot A_{cell}$$

$A_{rack} = 240.896 \cdot in^2$ Area of Gross Cross Section

$$n_0 := if\left(floor\left(\frac{N_x - 1}{2}\right) = \frac{N_x - 1}{2}, 1, 0.5\right) \qquad row := 0 .. \left(ceil\left(\frac{N_x - 1}{2}\right) - 1\right)$$

$$I_{row} := N_y \cdot \left[I_{cell} + A_{cell} \cdot \left[\left(n_0 + row\right) \cdot p\right]^2\right]$$

$$Iyy_{rack} := if\left[floor\left(\frac{N_x - 1}{2}\right) = \frac{N_x - 1}{2}, \left(N_y \cdot I_{cell} + 2 \cdot \Sigma I\right), 2 \cdot \Sigma I\right]$$

$Iyy_{rack} = 1.528 \cdot 10^5 \cdot in^4$ Y-Moment of Inertia of Gross Cross Section

$$n_0 := if\left(floor\left(\frac{N_y - 1}{2}\right) = \frac{N_y - 1}{2}, 1, 0.5\right) \qquad row := 0 .. \left(ceil\left(\frac{N_y - 1}{2}\right) - 1\right)$$

$$I_{row} := N_x \cdot \left[I_{cell} + A_{cell} \cdot \left[\left(n_0 + row\right) \cdot p\right]^2\right]$$

$$Ixx_{rack} := if\left[floor\left(\frac{N_y - 1}{2}\right) = \frac{N_y - 1}{2}, \left(N_x \cdot I_{cell} + 2 \cdot \Sigma I\right), 2 \cdot \Sigma I\right]$$

$Ixx_{rack} = 2.809 \cdot 10^5 \cdot in^4$ X-Moment of Inertia of Gross Cross Section

The following stiffness values are calculated based on the material and geometric properties of the rack

$K_{EXTENSION} := \frac{E \cdot A_{rack}}{H}$ $\qquad K_{EXTENSION} = 3.958 \cdot 10^{7} \cdot \frac{lbf}{in}$

$K_{BENDINGX} := \frac{E \cdot Ixx_{rack}}{H} \cdot \frac{1}{rad}$ $\qquad K_{BENDINGX} = 4.614 \cdot 10^{10} \cdot \frac{lbf \cdot in}{rad}$

$K_{BENDINGY} := \frac{E \cdot Iyy_{rack}}{H} \cdot \frac{1}{rad}$ $\qquad K_{BENDINGY} = 2.511 \cdot 10^{10} \cdot \frac{lbf \cdot in}{rad}$

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ATTACHMENT 1H

ATTACHMENT 1H

DYNARACK SHEAR DEFORMATION CALCULATION IN ACCORDANCE WITH HOLTEC WS-126 - CALCULATION OF COMPOSITE SHEAR DEFORMATION CONSTANT FOR RACK "CA", VYNPS

Input

Height of Storage Cell $L_{total} := 168 \cdot in$

Length of intermittant weld $L_1 := 8 \cdot in$

Total Length Between Intermittant Welds $L_2 := \frac{(L_{total} - 6 \cdot L_1)}{5}$ $L_2 = 24 \cdot in$

$x_1 := \frac{L_1}{L_{total}}$ $x_1 = 0.048$

$x_2 := \frac{L_1 + L_2}{L_{total}}$ $x_2 = 0.19$ $x_3 := \frac{L_2}{L_{total}}$ $x_3 = 0.143$

$$A2 := x_1^2 + (x_2 + x_1)^2 - x_2^2 + (2 \cdot x_2 + x_1)^2 - (2 \cdot x_2)^2 + (3 \cdot x_2 + x_1)^2 - (3 \cdot x_2)^2 \ldots + (4 \cdot x_2 + x_1)^2 - (4 \cdot x_2)^2 + (5 \cdot x_2 + x_1)^2 - (5 \cdot x_2)^2$$

$A2 = 0.286$

$$B2 := x_2^2 - x_1^2 + (2 \cdot x_2)^2 - (x_1 + x_2)^2 + (3 \cdot x_2)^2 - (2 \cdot x_2 + x_1)^2 + (4 \cdot x_2)^2 \ldots + (-1) \cdot (3 \cdot x_2 + x_1)^2 + (5 \cdot x_2)^2 - (4 \cdot x_2 + x_1)^2$$

$B2 = 0.714$

We can now compute the effective shear deformation for each of two directions, using the information calculated in Attachment A.

Shear Deformation Coefficient for the "y" direction deformation (Attachment A)

Moment of inertia of rack cross section about the x-x axis $I_{xx} := 280900 \cdot in^4$

Metal area of rack cross section $A_t := 240.896 \cdot in^2$

Moment of inertia of a single cell $I := 5.502 \cdot in^4$

Number of cells in rack $n := 14 \cdot 19$ $n = 266$

$\alpha := 2.276$ (from calculation file "sheardef.mcd")

$$\phi_H := 31.2 \cdot \alpha \cdot \frac{I_{xx}}{A_t} \cdot \frac{1}{L_{total}^2} \qquad \phi_H = 2.934$$

This is the shear deformation coefficient if the entire rack were of honeycomb construction

For an individual fuel cell in an end connected region

$$\phi_e := 31.2 \cdot \frac{6}{5} \cdot \frac{I \cdot n}{A_t} \cdot \frac{1}{L_2^2} \qquad \phi_e = 0.395$$

Then the effective shear deformation coefficient for the assemblage of individual fuel cells is

$$\phi_2 := \frac{(1 + \phi_e) \, I_{xx}}{n \cdot I} - 1 \qquad \phi_2 = 266.727$$

$$\phi_{stary} := \phi_H \cdot A2 + \phi_2 \cdot B2 \cdot x_3^2$$

The effective shear deformation coefficient, for the full length composite section, is

$$\phi_{stary} = 4.726$$

Shear Deformation Coefficient for the "x" direction deformation

Moment of inertia of rack cross section about the y-y axis $I_{yy} := 152800 \cdot in^4$

Metal area of rack cross section $A_t := 240.896 \cdot in^2$

Moment of inertia of a single cell $I := 5.502 \cdot in^4$

Number of cells in rack $n := 14 \cdot 19$ $n = 266$

$\alpha := 2.245$ (from calculation file "sheardef.mcd")

$$\phi_H := 31.2 \cdot \alpha \cdot \frac{I_{yy}}{A_t} \cdot \frac{1}{L_{total}^2} \qquad \phi_H = 1.574$$

This is the shear deformation coefficient if the entire rack were of honeycomb construction

For an individual fuel cell in an end connected region

$$\phi_e := 31.2 \cdot \frac{6}{5} \cdot \frac{I \cdot n}{A_t} \cdot \frac{1}{L_2^2} \qquad \phi_e = 0.395$$

Then the effective shear deformation coefficient for the assemblage of individual fuel cells is

$$\phi_2 := \frac{\left(1 + \phi_e\right) I_{yy}}{n \cdot I} - 1 \qquad \phi_2 = 144.634$$

$$\phi_{starx} := \phi_H \cdot A2 + \phi_2 \cdot B2 \cdot x_3^2$$

The effective shear deformation coefficient, for the full length composite section, is

$$\phi_{starx} = 2.558$$

DYNARACK TORSIONAL INERTIA PROPERTY CALCULATION IN ACCORDANCE WITH HOLTEC WS-126

For the effective torsional rigidity, we use the formula in WS-126. For the VYNPS CA Rack

$J_1 := 170800 \cdot in^4$ (from calculation file "sheardef.mcd")

The torsional inertia of the rack considered as a series of end connected tubes is

$J_2 := 5820 \cdot in^4$ (from calculation file "sheardef.mcd")

Therefore, following WS-126, we have the composite moment of inertia as

$$J := \frac{J_1}{\left(A2 + \frac{J_1}{J_2} \cdot B2\right)} \qquad J = 8.038 \cdot 10^3 \, in^4$$

SHEAR AND TORSION SPRING CONSTANTS

Young's Modulus $E := 27600000 \cdot psi$ $H := 168 \cdot in$

Poisson's Ratio $\nu := 0.3$

Using results from Attachment A

Shear Spring Rate for y-direction deformation

$$K_{shear\_y} := \left(4.614 \cdot 10^{10} \cdot in \cdot lbf\right) \cdot \frac{12}{H^2 \cdot \left(1 + \phi_{stary}\right)} \qquad K_{shear\_y} = 3.426 \cdot 10^6 \, \frac{lbf}{in}$$

Shear Spring Rate for x-direction deformation

$$K_{shear\_x} := \left(2.511 \cdot 10^{10} \cdot in \cdot lbf\right) \cdot \frac{12}{H^2 \cdot \left(1 + \phi_{starx}\right)} \qquad K_{shear\_x} = 3 \cdot 10^6 \, \frac{lbf}{in}$$

Torsional Spring Rate $$K_{torsion} := \frac{E}{2 \cdot (1 + \nu)} \cdot \frac{J}{H} \qquad K_{torsion} = 5.079 \cdot 10^8 \, lbf \cdot in$$

HOLTEC POSITION PAPER WS-126

ATTACHMENT 1J

THEORETICAL BASIS FOR SHEAR AND TORSIONAL SPRING CONSTANTS IN A HOLTEC SPENT FUEL RACK

HOLTEC POSITION PAPER WS-126

Author: Alan I. Soler, Ph.D.

Revision 0: October 23, 1998

Scope

The Holtec Spent Fuel Racks are modeled as a linearly elastfc structure in the dynamic simulation of a seismic event. The methodology for calculation of spring constants for the dynamic model has been developed for more than a decade. While the calculation of spring constants for bending and extension is well understood and simple to explain, the calculation of the spring constants for shear effects and for torsion merits additional comments. While the development is set forth in appropriate Holtec theory reports, the purpose of this position paper is to set down the theoretical basis in a form that is easily communicated to the general public on a "need-to-know" basis.

Evaluation of shear deformation in a composite beam representing a fuel rack

The purpose of this development is to demonstrate how the effective shear deformation coefficient for a composite beam representing the Holtec spent fuel storage rack is developed. Modeling the rack as a composite short beam section of length L, area moment of inertia I, and metal cross section area A, the complementary energy U is written as

$$U = \int_0^L \left(\frac{M^2}{2EI} + \frac{\alpha V^2}{2GA} \right) dx$$

where M(x) = bending moment

V(x) = shear force

and x represents a length coordinate defined along the neutral axis of the beam.

E, A are the Young's Modulus and Shear Modulus, respectively. Let us consider a beam that has uniform A and I but unknown shear deformation constant α along the beam. That is, in general, write

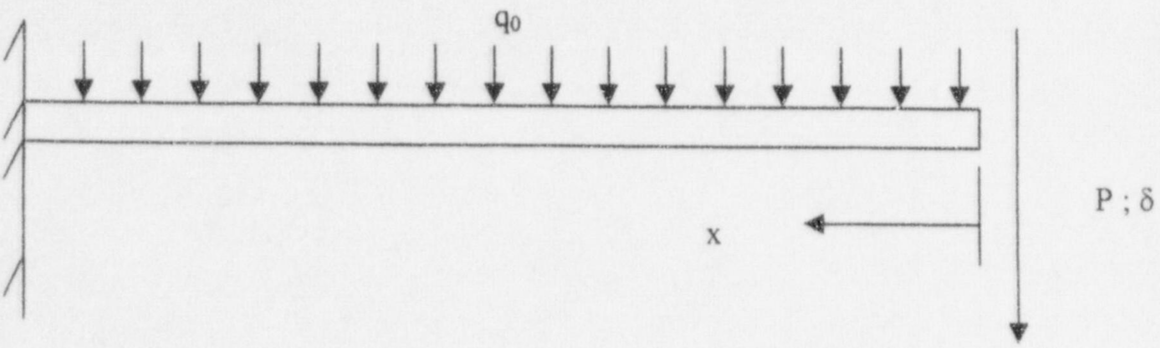
$$U = \frac{1}{2EI}\int_0^L \left(M^2 + \alpha \frac{EI}{GA} V^2 \right) dx$$

Define $\phi_i = \frac{12}{L_i^2}\alpha_i \frac{EI}{GA} = \frac{12}{L_i^2}\varphi_i$

where i = 1, 2 represent distinct regions along the length of the beam where α_i has two distinct values α_1, α_2. In particular, in region 1 of length L_1, we have fully welded honeycomb construction where $\alpha_1 \approx 1.6\text{-}12/5$ which is appropriate for a rectangular shaped multi-cell grid beam. In region 2 of length L_2, we have an effective shear deformation coefficient α_2 that represents the assemblage of individual boxes. The complementary energy U can now be written as:

$$U = \frac{I}{2EI}\int \left[M^2(x) + \varphi(x) V^2(x) \right] dx$$

We consider a uniformly loaded cantilever beam with an additional concentrated load P at the tip and determine the tip deflection δ using Castigliano's Theorem.



Noting that

$M(x) = Px + q_0x^2/2 \quad ; V(x) = P + q_0x$

$$\delta = \left.\frac{\partial U}{\partial P}\right|_{P=0} = \frac{1}{EI}\int_0^L \left[M \left.\frac{\partial M}{\partial P}\right|_{P=0} + \varphi V \left.\frac{\partial V}{\partial P}\right|_{P=0} \right] dx$$

$$\frac{\partial M}{\partial P} = x \qquad \frac{\partial V}{\partial P} = 1$$

so that, in general

$$\delta = \frac{1}{EI}\int_0^L \left[M(x)x + \varphi(x)V(x) \right]\Big|_{P=0} dx$$

We first compute this for a beam where there is a uniform connectivity so that $\varphi(x) = \theta$. The solution is:

$$\delta = \frac{q_0 L^4}{8EI}\left[1 + \frac{4\theta}{L^2}\right]$$

Note : that increase in φ leads to lower natural frequency.

We see that in general, the term dealing with shear deformation is the second term. Thus, the uniform beam of length L, for this special case,

$$\frac{q_0 L^2}{2\,EI}\theta = \int^L \varphi(x)\frac{q_0}{EI}x\,dx$$

or

$$\frac{\theta L^2}{2} = \int_0^L \varphi(x)\,x\,dx$$

Of course, for $\varphi(x) = \theta$, the above relationship is an <u>identity</u>. However, we now postulate that for the composite beam consisting of 6 sections of length L_1 and 5 sections of length L_2, the <u>same</u> relation can be used to define an <u>effective</u> θ for a uniform beam and thus define an effective $\phi = (12/L^2)\,\theta$.

Then in general, the effective ϕ is defined as

$$\phi \frac{L^4}{24} = \int_0^L \varphi(x) x dx$$

or in terms of a coordinate $z = \frac{x}{L}$

$$\phi \frac{L^2}{24} = \int_0^1 \varphi(z) z dz$$ where L = total length of beam

Now, in each of the sections (i.e. honeycomb or end-connected) we have

$$\varphi_1 = \frac{L_1^2}{12}\phi_1 \qquad or \qquad \varphi_2 = \frac{L_2^2}{12}\phi_2$$

so that

$$\phi = 2\int_{L_1} \phi_1 \left(\frac{L_1}{L}\right)^2 zdz + 2\int_{L_2} \phi_2 \left(\frac{L_2}{L}\right)^2 zdz$$

where

$\int_{L_i} (\ )dz$ means the totality of the integrals evaluated over all of the sections of length L_i.

We note that the paper "Seismic Response of a Free Standing Fuel Rack Construction to 3-D Floor Motion", by Soler and Singh (Nuclear Engineering Design, Vol. 80 (1984) pp. 315-329) provides the appropriate equations for ϕ_1 and ϕ_2. The work in that paper assumed that shear deformation effects could be neglected in the so-called "end-connected" construction because that construction was present over the entire length of the rack. In the Holtec rack construction, the individual regions between the honeycomb welded sections can have a much more important shear deformation effect since the ratio of characteristic cross section length to length of end-connected construction is much larger. Modifying the formulas in the paper so as to include shear deformation in the individual segments of end-connected construction leads to the following results for ϕ_1 and ϕ_2:

$$\phi_1 = \frac{12EI^*}{GA^*L_1^2}\alpha \qquad \alpha = 6/5$$

$$\phi_2 = \frac{(1+\phi_1)I^*}{nI} - 1$$

where I^* = total moment of inertia about the centroid of n fuel cells.

I = moment of inertia about the center of a single fuel cell (or opening)

n = number of assemblies in the rack

$A^* = nA$; A = area of a single cell

If we define

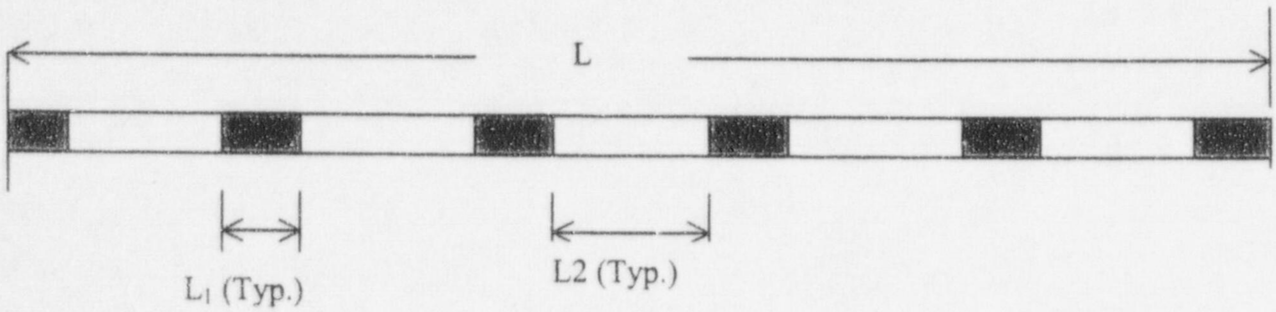
$\phi_H = \frac{12EI^*}{GA^*L^2}\alpha$ = shear deformation coefficient if the entire rack were honeycomb welded construction, then

$$\phi = 2\int_{L_1} \phi_H z dz + 2\int_{L_2} \phi_2 \left(\frac{L_2}{L}\right)^2 z dz$$

Now consider the spent fuel rack where we have six honeycomb (welded) sections and five end-connected sections. For the purpose of this analysis, we assume an average L_2 (the bottom section has a smaller unconnected length) and define

$$x_1 = L_1 / L \qquad x_2 = \frac{L_1 + L_2}{L} \qquad \textit{where}$$

the following sketch shows a representation of the fuel rack as a single beam with regions of honeycomb (welded) construction and end-connected individual fuel cells identified:



Define geometric quantities A and B so that

$$A/2 = \int_0^{x_1} zdz + \int_{x_2}^{x_1+x_2} zdz + \int_{2x_2}^{2x_2+x_1} zdz + \int_{3x_2}^{3x_2+x_1} zdz + \int_{4x_2}^{4x_2+x_1} zdz + \int_{5x_2}^{5x_2+x_1} zdz$$

$$B/2 = \int_{x_1}^{x_2} zdz + \int_{x_2+x_1}^{2x_2} zdz + \int_{2x_2+x_1}^{3x_2} zdz + \int_{3x_2+x_1}^{4x_2} zdz + \int_{4x_2+x_1}^{5x_2} zdz$$

Integrating yields the following results for A and B

$$A = x_1^2 + (x_1+x_2)^2 - x_2^2 + (2x_2+x_1)^2 - (2x_2)^2 + (3x_2+x_1)^2 - (3x_2)^2 + (4x_2+x_1)^2 - (4x_2)^2 + (5x_2+x_1)^2 - (5x_2)^2$$

$$B = x_2^2 - x_1^2 + (2x_2)^2 - (x_2+x_1)^2 + (3x_2)^2 - (2x_2+x_1)^2 + (4x_2)^2 - (3x_2+x_1)^2 + (5x_2)^2 - (4x_2+x_1)^2$$

Note that in the limit as $L_1 = 0$, and $L_2 = L/5$ - $A=0$ and $B=1$.

In the limit $L_1 = L/6$ and $L_2 = 0$. - $A = 1$ and $B = 0$.

Therefore, in terms of the known values for

$$\phi_H , \phi_2$$

the effective shear deformation coefficient for the composite beam representing the spent fuel rack is expressible in the final form

$$\phi = \phi_H A + \phi_2 \left({L_2}/{L} \right)^2 B$$

The determination of the effective shear deformation value for the rack in terms of the rack geometry and proportion of honeycomb and end-connected construction enables the appropriate spring constant to be calculated per the formulas in the reference text "Component Element Method in Dynamics", by S. Levy and J. Wilkerson, McGraw-Hill, 1976. This is implemented in the Holtec computer code for simulating the dynamics of spent fuel racks.

Evaluation of effective Polar Moment of Inertia in a composite beam representing a fuel rack

To compute the appropriate torsion spring constant to represent the resistance of the spent fuel rack to a twist about its longitudinal axis, the effective Polar Moment of Inertia needs to be computed. Consider the complementary energy U for the torsional behavior of the composite beam-like structure representing the spent fuel rack:

$$U = \int_0^L \frac{T^2(x)}{2GJ} dx$$

where T(x) is the twisting moment acting on the beam. The variable x is the coordinate defined along the length of the beam in the same manner as was done for the shear deformation calculation above. Let m_0 be a distributed uniform twisting moment (per unit length of beam) and T_0 be a concentrated twisting moment at the tip of the beam (x=0). Using Castigliano's Theorem, the angle of twist at the tip of the beam is obtained as

$$\theta = \int_0^L \frac{T(x)}{GJ(x)} dx = \frac{m_0}{G} \int_0^L \frac{x}{J(x)} dx$$

For the beam with constant J, the following relation is an identity

$$\frac{L^2}{2J} = \int_0^L \frac{x}{J(x)} dx$$

For a beam with variable J(x), the above relation can be used to define an effective uniform J = J_e, to give the same angle of twist at the top of the beam

$$\frac{L^2}{2J_e} = \int_0^L \frac{x}{J(x)}dx$$

As was done previously in the development of the effective shear deformation value, we introduce dimensionless variable z=x/L. Then the effective Polar Moment of Inertia J_e can be written in the form

$$\frac{1}{2J_e} = \int_{L_1} \frac{zdz}{J_1} + \int_{L_2} \frac{zdz}{J_2}$$

where the integral notation represents integration over the appropriate sections of the composite structure representing the spent fuel rack. We recognize the integrals as the functions A and B defined previously. Therefore, J_e is given in terms of the individual properties J_1 and J_2 (for the different construction along the length of the beam representing the spent fuel rack) as:

$1/J_e = A/J_1 + B/J_2$

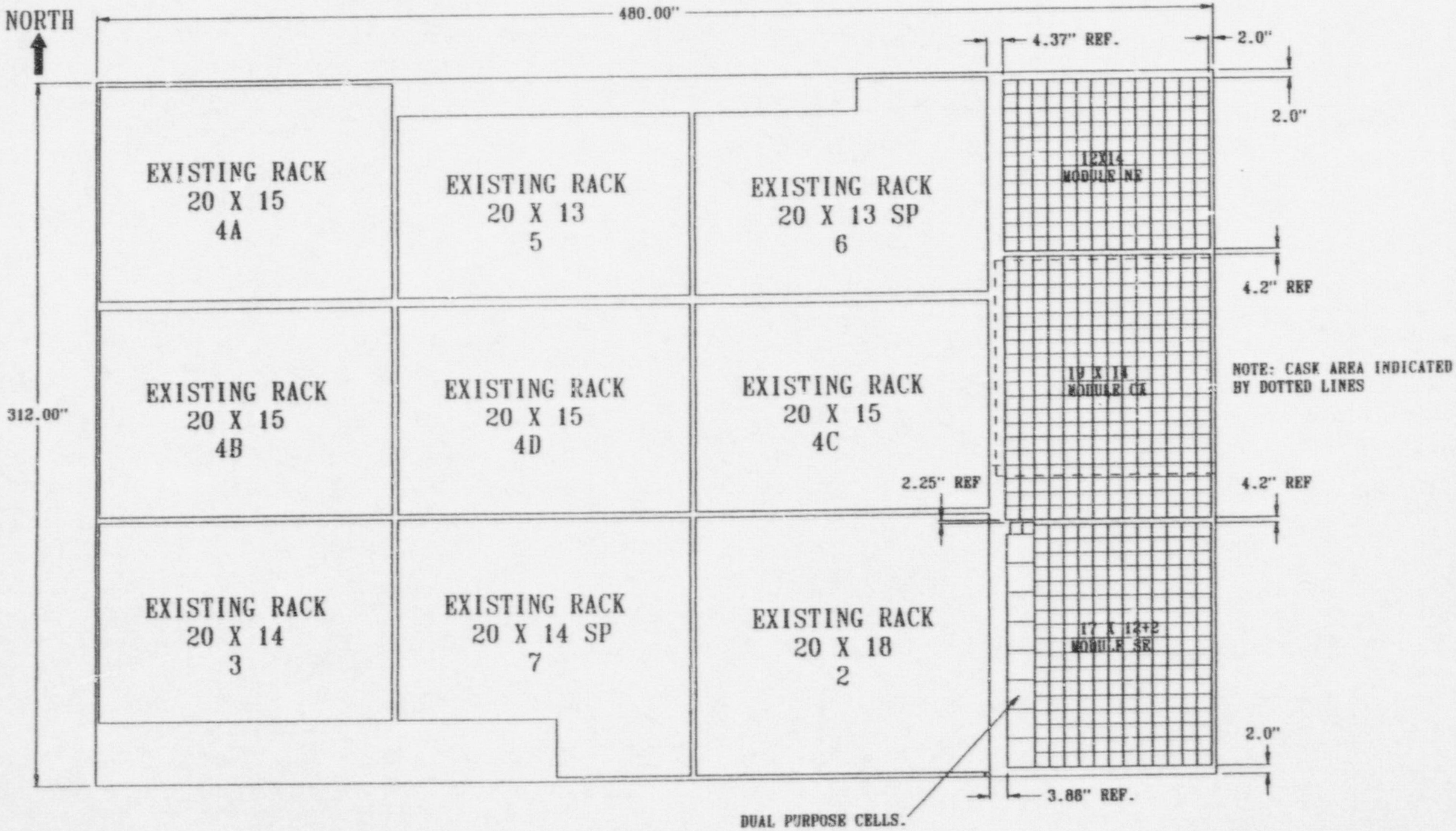
Therefore, the effective Polar Moment of Inertia is written in the form

$$J_e = \frac{J_1}{\left(A + \frac{J_1}{J_2}B\right)}$$

With the effective Polar Moment of Inertia developed, the appropriate twisting spring rate is formed using the methodology in the text "Component Element Method in Dynamics" and implemented in the Holtec computer code for dynamics of spent fuel racks.

RAI RESPONSES
for
VERMONT YANKEE NUCLEAR STATION
RERACKING PROJECT

ATTACHMENT 1K



BOX I.D. = 6.0
BOX WALL THK. = 0.075
INNER SHEATHING THK. = 0.035
PITCH = 6.218

NEW RACKS : 640 CELLS
DUAL PURPOSE CELLS: 8
TOTAL CELL COUNT:672.

FIGURE 2.1: POOL LAYOUT FOR VERMONT YANKEE

RACK	WEIGHT(lb.)
MODULE CA	25,860#
MODULE SE	23,800#
MODULE NE	16,810#

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Table 2.2

NUMBER OF STORAGE CELLS

MODULE I.D.	QTY.	NUMBER OF CELLS		Total Per Rack
		North-South Direction	East-West Direction	
CA	1	19	14	266
SE	1	17	12 + 2 cells	238 *
NE	1	12	14	168
Existing Racks in Pool	9	–	–	2,683
TOTAL:	**12**	–	–	**3,355**

* Includes use of eight dual-purpose storage cells for fuel.

Table 2.3 **MODULE DIMENSIONS AND WEIGHTS FOR VY RERACK**			
Module I.D.	Dimension (inches)†		Shipping Weight in Pounds
	North-South	East-West	
NE	74.991	87.427	16,810
CA	118.517	87.427	25,860
SE	108.2	87.916	23,800

† Nominal rectangular planform dimensions.

Docket No. 50-271
BVY 99-115

Attachment 2

Vermont Yankee Nuclear Power Station

Response to Request for Additional Information Regarding
Spent Fuel Pool Storage Capacity Expansion
(Civil and Mechanical Engineering Considerations)

Response to Question 2

Question 2a:

You indicated that the design conditions described in SRP Section 3.8.4 and ACI 349-80 were used as a guidance in the calculations of the spent fuel pool (SFP) capacity. With respect to the SFP capacity calculations using the ANSYS computer code presented in Chapter 8 of the submittal:

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

Response 2a:

The structural analysis that was performed to determine the capacity of the spent fuel pool (SFP) structure did not employ use of the ANSYS computer program. The ANSYS computer code was used in the fuel rack pedestal bearing pad analysis, as described in Section 8.5 of the submittal. With the exception of the structural reanalysis performed by EBASCO in 1984, all follow-on analyses addressing the SFP capacity, including the structural analysis of record, utilize hand analysis techniques. The computer analysis performed by EBASCO used the finite element program, NASTRAN, and provided the basis for the scope of the structural evaluation in the follow-on hand analyses.

The SFP slab is 26'-0" wide by 40'-0" long and is approximately 4'-1" thick. It is supported on the north side by the 4'-5" (minimum) thick reactor shield wall and on the south side by a deep, 5'-6" thick east-west concrete tie beam, which also serves as support for the south wall of the SFP. The east and west sides of the SFP slab, and the east-west tie beam, are supported by a pair of deep north-south concrete beams, 6'-0" thick, which, in turn, are supported by the reactor shield wall and the 3'-2" thick south exterior concrete wall of the building. The SFP is 39'-0 ¾" deep.

Concrete floor slabs at elevations 303.00', 318.67', and 345.67' are contiguous with the SFP structure.

The interior of the pool is constructed with a stainless steel liner anchored to the concrete walls and floor by stiffeners. The liner is designed as a leaktight membrane and is not relied upon as a structural member. The plate thickness of the floor liner is 1/4 inch, and the plate thickness of the wall 3/16 inch. The spacing for horizontal and vertical stiffeners on wall liners is 6'-6" and 1'-8", respectfully. On the floor liner, the stiffeners are only in the north-south direction having a spacing of 4'-5" of 5'-7." Beneath the floor liner there is an 11" thick grout pad on top of the rough concrete surface at El. 305.50'.

Question 2b:

Provide the mesh used in the analysis.

Response 2b:

As noted in the response to part a) above, computer analysis techniques were not employed in the SFP capacity analysis of record. The SFP reanalysis performed by EBASCO in 1984 using the finite element program NASTRAN provided the basis for the scope of the follow-on structural analysis. The methodology employed in the structural analysis to determine the SFP capacity is contained in Section 3.3.3.6.2 of Vermont Yankee's licensing submittal Spent Fuel Storage Rack Replacement Report for installation of the NES racks (dated April 1986) and is reprinted in part below. (References noted below will be provided if requested.)

"The basic approach in determining fuel rack load capacity was to isolate the controlling element of the load path and then determine the load capacity (in terms of ksf fuel rack floor load) with respect to this element. The strength design method for reinforced concrete was used in conjunction with conventional structural analysis procedures to determine capacities. Section strengths were determined using the methods and procedures contained in Reference 31. The allowable limit loads were converted to actual rack loads using the load equations contained in Reference 9.

The impact loading associated with predicted rocking motion of the racks was included as part of the seismic loading considerations. Determination of impact energy of the fuel racks was based on maximum predicted gaps between the fuel rack support legs and the supporting surface of the fuel pool floor. Gap data were obtained from Table 3-8, which considered an equivalent fuel rack load of 2.87 ksf. This data was a direct product of the NES Report identified as Reference 34. The impact energy applied to the slab was determined using vectorial partitioning of the total racking energy with respect to normal impact on the slab (impact energy based on velocity component normal to the slab).

The kinetic energy transmitted into the slab during impact and the resulting structural response was determined using the methods and procedures contained in References 31, 32, and 33.

To account for the nonconcurrent aspects of the impacts from different racks (Reference 34), the slab kinetic energies from each impact were combined using the square-root-sum-of-squares (SRSS) method to determine the maximum concurrent kinetic energy of the slab.

Kinetic energy was then equated to strain energy to determine the maximum structured response. This enabled determination of the differential uniform load capacity required for impact effects.

The fuel rack load capacity was then determined from the controlling load equation considering the fuel rack dead and seismic (with impact) loads to be proportional to the mass of the fuel racks."

The methodology described above was taken directly from Vermont Yankee's licensing submittal for installation of the NES racks in 1986. Although the methodology remained essentially the same for the structural analysis in support of the proposed installation of the Holtec racks, it is important to note two differences from the above methodology: 1) the equivalent fuel rack load was taken as 2.93 ksf (due to the weight of fuel being conservatively taken as 700 lbs, compared to a fuel weight of 670 lbs used in the 1986 submittal), and 2) impact energies from individual racks were conservatively taken as acting concurrently and, consequently, cumulative impact effects were determined by means of absolute summation.

Question 2c:

Identify and describe the boundary conditions used in the mesh.

Response 2c:

In the hand evaluation performed, the slab boundaries were taken as fixed due to the high relative stiffness of the supporting concrete beams and walls.

Question 2d:

Provide the material properties used in the analysis.

Response 2d:

The properties of the reinforced concrete SFP used in the analysis are the same as the properties identified in Section 2.3.3.2 of Vermont Yankee's licensing submittal Spent Fuel Storage Rack Replacement Report for installation of the NES racks (dated April 1986) and is reprinted in part below.

"The following symbols are used to describe the material properties used in the structural analysis:

f'_c = Compressive strength of concrete

γ = Specific weight

μ = Poisson's ratio

α = Coefficient of linear expansion

E = Modulus of elasticity

f_Y = Yield stress

A. Concrete

f'_c = 4000.0 psi * (all members except SFP slab)
(f'_c = 6400.0 psi - SFP slab)*
γ = 150.0 pcf

$$\mu = \frac{(f'_c)^{1/2}}{350} = 0.181$$

E = 3.834×10^3 ksi *

α = 5.5×10^{-6} per °F

B. Reinforcing Steel

γ = 490 pcf

f_Y = 40.0 ksi

$\mu = 0.3$

$E = 29 \times 10^3$ ksi

$\alpha = 6.5 \times 10^{-6}$ per °F

* For the fuel pool floor the in place concrete test data shown in Table 2-2 may be used."

Table 2-2 documents the testing program results that were the basis to upgrade compressive strength properties of the SFP slab. Table 2-2 is attached to this response.

Question 2e:

Describe the applied loading conditions including the magnitudes, and indicate their locations in the mesh.

Response 2e:

The SFP structure is subjected to dead load, live load from the contiguous floors, hydrostatic load from a water depth of approximately 35'-9", the weight of the high density spent fuel racks plus fuel, and the effects of OBE and SSE seismic loads, as well as thermal loads. The loads are essentially unchanged from those described in Vermont Yankee's licensing submittal Spent Fuel Storage Rack Replacement Report, dated April 1986, which was prepared in support of the proposed installation of the NES racks. The loads and load combinations are defined in Sections 2.3.3.3 and 2.3.3.4, respectively, of that report and are reprinted in part below. (References noted below will be provided if requested.)

"2.3.3.3 Design Loads

A. Dead Load (D)

Dead loads consist of the dead weight of concrete, grout steel liner, fuel racks, fuel, and any equipment permanently attached to the pool.

Hydrostatic pressure loads acting on walls and floor shall be included in this category. It should be noted that the hydrostatic load acting on the north wall of the pool is dependent on the operation condition.

Dead loads distributed from the contiguous floors and beams to the pool walls shall be considered in the analysis. An 80 psf intensity on all floors connected to the pool shall be used to account for piping weights as documented in Reference 13.

B. Live Load (L)

Live loads are random, temporary load conditions during maintenance and operation. They shall include the following loads:

- Weight of fuel cask

- Weight of control rod storage racks and stored control rods
- Weight of refueling bridge and service platform

Live loads distributed from the contiguous floors and beams shall be calculated based on the following load densities:

- El. 345.67' 500.0 psf
- El. 318.67' 200.0 psf
- El. 303.00' 200.0 psf

Values of load densities are obtained from Reference 13.

C. Normal Operating Thermal Load (T_o)

These thermal loads are generated under normal operating or shutdown conditions. The following temperature data shall be used in the calculation of thermal gradients through walls and floor:

- Inside drywell temperature 135°F
- Pool water temperature 150°F
- Room temperature 60°F
- Outside ambient temperature:

  Summer 100°F

  Winter 0°F

D. Accident Thermal Loads (Ta)

These thermal loads are due to the thermal conditions generated by the postulated accidents. The thermal accident temperature for the spent fuel pool is 212°F throughout the whole pool.

E. Seismic Loads (E, E')

E = Seismic loads generated by Operating Basis Earthquake (OBE)

E' = Seismic loads generated by Safe Shutdown Earthquake (SSE)

The SSE seismic loads are assumed to be twice that of the OBE seismic loads. The maximum accelerations for OBE are given in Figures 2-6 through 2-8. The vertical seismic effects shall be considered either upward or downward to result in the worst loading in the load combinations.

The hydrodynamic loads of pool water acting on pool walls shall be calculated in accordance with Chapter 6 of Reference (14).

The vertical seismic loads due to the refueling bridge and service platform and the vertical seismic loads distributed from the contiguous floors and beams shall be included in the analysis.

The seismic loads due to equipment mounted on pool walls shall be calculated using 150 percent of the peak floor response spectra.

The effects of three components of earthquake shall be combined by the SRSS method. Amplified response spectra provided by YAEC shall be used to determine the appropriate acceleration values for computing seismic loads.

The effect of rack impact on the pool floor during a seismic event should also be considered in the analysis. These loads should be combined with the loads due to vertical seismic floor acceleration.

2.3.3.4 Load Combinations

The following load combinations shall be considered in the spent fuel pool analysis (Paragraph 3.0, Section 3.8.4 of NUREG-0800[9]:

A. Concrete Pool Structure

1. Service Load Conditions

a. $1.4D + 1.7L$

b. $1.4D + 1.7L + 1.9E$

c. $(0.75)\ (1.4D + 1.7L + 1.7T_o)$

d. $(0.75)\ (1.4D + 1.7L + 1.9E + 1.7T_o)$

e. $1.2D + 1.9E$

2. Factored Load Conditions

a. $D + L + T_o + E'$

b. $D + L + T_a$

c. $D + L + T_a + 1.25E'$

Where any load reduces the effects of other loads, the corresponding coefficients for that load should be taken as 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with other loads. Otherwise, the coefficient for that load should be taken as zero."

Question 2f:

Explain how the interface between the liner and the concrete slab is modeled, and also, how the liner anchors are modeled. Provide the basis for using such modeling with respect to how they accurately represent the real structure behavior.

Response 2f:

In the structural analysis to determine the SFP capacity, no credit was taken for either the pool liner, and its anchorage to the concrete structure, or the 11" thick grout pad.

Question 2g:

Provide the calculated governing factors of safety in tabular form for the axial, shear, bending, and combined stress conditions.

Response 2g:

The controlling element in the structural analysis of the SFP structure is shear in the slab for the load combination involving OBE loading. Based upon the maximum loading from the proposed Holtec rack configuration, the minimum factor of safety is conservatively calculated to be 1.07. With respect to flexure, the factor of safety in the slab is conservatively calculated as 1.08. Wall stresses are governed by the south wall of the SFP. The factors of safety based upon shear and flexure for the south wall are 1.46 and 1.30, respectively.

These factors of safety are summarized below.

SFP Structural Element	Factor of Safety	
	Shear	Flexure
Slab	1.07	1.08
Walls (governed by south wall)	1.46	1.30

Question 2h:

What is the maximum bulk pool temperature at a full-core offload during a refueling outage? If the temperature exceeds 150° F, provide the following:

i) Describe the details of the SFP structural analysis including the material properties (i.e., modulus of elasticity, shear modulus, poisson's ratio, yield stress and strain, ultimate stress and strain, compressive strength) used in the analysis for the reinforced concrete slab and walls, liner plate, welds and anchorage in the analysis.

ii) ACI Code 349 limits a concrete temperature up to 150° F for normal operation or any other long-term period. Provide technical justifications for exceeding the required temperature of 150° F.

Response 2h:

The maximum bulk pool temperature at a full core offload during a refueling outage is 150° F.

An evaluation of the SFP slab was also made for an accident temperature of 212°F. Based upon data extracted from published test results, the strength in the top one third of the slab was reduced by 20% to account for the increased temperature in excess of 150° F. The slab was still found to be acceptable. However, it is noted that scenarios that could result in spent fuel pool bulk water temperatures greater than 150°F are beyond our licensing Basis.

Docket No. 50-271
BVY 99-115

Attachment 1A

Vermont Yankee Nuclear Power Station

Response to Request for Additional Information Regarding
Spent Fuel Pool Storage Capacity Expansion
(Civil and Mechanical Engineering Considerations)

Holtec Position Paper WS-115, Revision 1, 3D Single Rack Analysis of Fuel Racks

Proprietary Information Enclosed