From: George Hubbard
To: GBK Glenn Kelly
Date: Wednesday, July 26, 2000 01:52 PM
Subject: Aee attached

4234

Appendix 5 Enhanced Seismic Checklist and Supporting Stakeholder Documentation

Appendix 5 contains the following sub-sections:

Appendix 5a Original NEI Screening Criteria, August 18, 1999

Alan Nelson
SENIOR PROJECT MANAGER
PLANT SUPPORT
NUCLEAR GENERATION DIVISION

Mr. Richard Dudley
Project Manager
U.S. Nuclear Regulatory Commission
Mail Stop 11 D19
Washington, DC 20555-0001

Dear Mr. Dudley:

On July 15-16, 1999, the NRC held a workshop on spent fuel accidents at decommissioning plants. During the course of the workshop, presentations by the NRC and the industry concluded that spent fuel pools possess substantial capability beyond their design basis to with stand seismic events but that variations in seismic capacity existed due to plant specific designs and locations.

The consensus was that the risk was low enough that precise quantification was not necessary to support exemption requests but that this needed to be confirmed on a plant specific basis with deterministic criteria. It was recommended that a simple spent fuel pool (SFP) vulnerability check list be developed to provide additional assurance that no beyond-design-basis seismic structural vulnerabilities exist at decommissioning plants. Enclosed for your review is the "Seismic Screening Criteria For Assessing Potential Pool Vulnerabilities At Decommissioning Plants."

Please contact me at (202) 739-8110 or by e-mail (apn@.nei.org) if you have any questions or if a meeting should be scheduled to discuss the enclosed seismic checklist.

Sincerely,

Alan Nelson

APN:tnb

Enclosure

Seismic Screening Criteria

For

Assessing Potential Fuel Pool Vulnerabilities

At

Decommissioning Plants

August 18, 1999

Background

To increase the efficiency and effectiveness of decommissioning regulations, the NRC staff has engaged in rulemaking activities that would reduce the need to routinely process exemptions once a plant is permanently shut down. With this goal in mind, members of the NRC staff, industry representatives and other stakeholders held a two-day workshop on risk related spent fuel pool accidents at decommissioning plants.

At this workshop, based upon presentations by the NRC staff (Goutam Bagchi et al.) and the nuclear industry (T. O'Hara - DE&S), it was concluded that a large seismic event (in the range of three times the design level earthquake) would represent a risk of exceeding the structural capacity of the spent fuel pool and thus potentially result in draining the pool.

Although the methodologies presented by the NRC staff and the industry differed somewhat, they both concluded that, in general, spent fuel pools possess substantial capacity beyond their design basis but that variations in seismic capacity existed due to plant specific details (i.e. " Differences in seismic capacity due to spent fuel location and other details.").

The consensus was that the risk was low enough that precise quantification was not necessary to support exemption requests but that this needed to be confirmed on a plant specific basis with deterministic criteria. It was recommended that a simple spent fuel pool (SFP) vulnerability check list be developed to provide additional assurance that no beyond-design-basis seismic structural vulnerabilities exist at decommissioning plants. The following pages provide the proposed structural vulnerability check list/screening criteria.

Purpose of Checklist

As discussed briefly in the "Background" section, the purpose of this checklist is to identify and evaluate specific seismic characteristics which might result in a specific spent fuel pool from not being capable of withstanding, without catastrophic failure, a beyond-design-basis seismic event equal in magnitude to approximately three times its design basis. Completion of the requirements will be performed by a qualified seismic engineer. This effort will include a thorough SFP walkdown and a review of appropriate SFP design drawings.

DRAFT CHECKLIST

Item 1:

Requirement: Assure Adequate Ductility of Shear Wall Structures

Basis: The expert panel involved with the development of Reference 1 concluded that, " For the Category 1 structures which comply with the requirements of either ACI 318-71 or ACI 349-76 or later building codes and are designed for an SSE of at least 0.1g pga, as long as they do not have any special problems as discussed below, the HCLPF capacity is at least 0.5g pga." This conclusion was based upon the assumption that the shear wall structure will respond in a ductile manner. The "special problems" cited deal with individual plant details, which could prevent a particular plant from responding in the required ductile fashion. Examples cited in Reference 1 included an embedded structural steel frame in a common shear wall at a plant (which was assumed to fail in brittle manner due to a potential shear failure of the attached shear studs) and large openings in a " crib house" roof which could interrupt the continuity of the structural slab.

Other examples which could impact the ductility of the spent fuel pool structure include large openings which are not adequately reinforced or reinforcing bars that are not sufficiently embedded to prevent a bond failure before the yield capacity of the steel is reached.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 2:

Requirement: Assure Design adequacy of Diaphragms (including roofs)

Basis: In the design of many nuclear power plants, the seismic design of roof and floor diaphragms has often not received the same level of attention as have the shear walls of the structures. Major cutouts for hatches or for pipe and electrical chases may pose special problems for diaphragms. Since more equipment tends to be anchored to the diaphragm compared to shear walls, moderate amounts of damage may be more critical for the diaphragm compared to the same amount of damage in a wall.

Based upon the guidance provided in Reference 1, diaphragms for Category I structures designed for a SSE of 0.1g or greater do not require an explicit evaluation provided that: (1) the diaphragm loads were developed using dynamic analysis methods; (2) they comply with the ductility detailing requirements of ACI 318-71 or ACI 349-76 or later editions. Diaphragms which do not comply with the above ductility detailing or which did not have loads explicitly calculated using dynamic analysis should be evaluated for a beyond-design-basis seismic event in the 0.45-0.5g pga range.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 3:

Requirement: Verify the Adequacy of Structural Steel (and Concrete) Frame Construction

Basis: At a number of older nuclear power plants, the walls and roof above the top of the spent fuel pool are constructed of structural steel. These steel frames were generally designed to resist hurricane and tornado wind loads, which exceeded the anticipated design basis seismic loads. A review of these steel (or possibly concrete) framed structures should be performed to assure that they could resist the seismic forces resulting from a beyond-design-basis seismic event in the 0.45-0.5g pga range. Such a review of steel structures should concentrate on structural detailing at connections. Similarly, concrete frame reviews should concentrate on the adequacy of the reinforcement detailing and embedment.
Failure of the structural steel superstructure should be evaluated for its potential impact on the ability of the spent fuel pool to continue to successfully maintain its water inventory for cooling and shielding of the spent fuel.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 4:

Requirement: Verify the Adequacy of Spent Fuel Pool Penetrations

Basis: The seismic and structural adequacy of any spent fuel pool (SFP) penetrations whose failure could result in the draining or syphoning of the SFP must be evaluated for the forces and displacements resulting from a beyond-design-basis seismic event in the 0.45-0.5g pga range. Specific examples include SFP gates and gate seals and low elevation SFP penetrations, such as, the fuel transfer chute/tube and possibly piping associated with the SFP cooling system. Failures of any penetrations, which could lead, to draining or siphoning of the SFP, should be considered.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 5:

Requirement: Evaluate the Potential for Impacts with Adjacent Structures

Basis: Structure-to-structure impact may become important for earthquakes significantly above the SSE, particularly for soil sites. Structures are usually conservatively designed with rattle space sufficient to preclude impact at the SSE level but there are no set standards for margins above the SSE. In most cases, impact is not a serious problem but, given the potential for impact, the consequences should be addressed. For impacts at earthquake levels below 0.5g pga, the most probable damage includes the potential for electrical equipment malfunction and for local structural damage. As cited previously, these levels of damage may be found to be acceptable or to result in the loss of SFP support equipment. The major focus of this impact review is to assure that the structure-to-structure impact does not result in the inability of the SFP to maintain its water inventory.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 6:

Requirement: Evaluate the Potential for Dropped Loads

Basis: A beyond-design-basis seismic event in the 0.45-0.5g pga range has the potential to cause the structural collapse of masonry walls and/or equipment supports systems. If these secondary structural failures could result in the accidental dropping of heavy loads which are always present (i.e. not loads associated with cask movements) into the SFP, then the consequences of these drops must be considered. As in previous evaluations, the focus of the drop consequence analyses should consider the possibility of draining the SFP. Additionally, the evaluation should evaluate the consequences of any resulting damage to the spent fuel or to the spent fuel storage racks.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 7:

Requirement: Evaluation of Other Failure Modes

Basis: Experienced seismic engineers should review the geotechnical and structural design details for the specific site and assure that there are not any design vulnerabilities which will not be adequately addressed by the review areas listed above. Soil-related failure modes including liquefaction and slope instability should be screened by the approaches outlined in Reference 1 (Section 7 & Appendix C).

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 7: Required Documentation

A simple report describing the results of the seismic engineer's walkdown and drawing review findings is judged to provide sufficient documentation to rule out a beyond-design-basis seismic event as a significant risk contributor to a decommissioned nuclear power plant.

References:

1. "A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)," (EPRI NP-6041-SL), August 1991

2. "Seismic Discussion Session," Workshop on Risk Related to Spent Fuel Pool Accidents at Decommissioning Plants, Stuart Richards, Goutam Bagchi and Gareth Parry, July 16, 1999

3. "Draft Technical Study of Spent Fuel Pool Accidents for Decommissioning Plants," by USNRC Technical Working Group - Vonna Ordaz et al., dated June 1999

4. "Risk Informed Decommissioning Emergency Planning," EPRI/NEI Project by Tom O'Hara (presented July 16, 1999)

5b Craig Memo to Holahan Forwarding Kennedy Report, November 19, 1999.

<u>Comments Concerning Seismic Screening
And Seismic Risk of Spent Fuel Pools for
Decommissioning Plants</u>

by

Robert P. Kennedy

October 1999

prepared for

Brookhaven National Laboratory

1. Introduction

I have been requested by Brookhaven National Laboratory, in support of the Engineering Research Applications Branch of the Nuclear Regulatory Commission, to review and comment on certain seismic related aspects of References 1 through 4. Specifically, I was requested to comment on the applicability of using seismic walkdowns and drawing reviews conducted following the guidance provided by seismic screening tables (seismic check lists) to assess that the risk of seismic-induced spent fuel pool accidents is adequately low. The desire is to use these seismic walkdowns and drawing reviews in lieu of more rigorous and much more costly seismic fragility evaluations. It is my understanding that the primary concern is with a sufficiently gross failure of the spent fuel pool so that water is rapidly drained resulting in the fuel becoming uncovered. However, there may also be a concern that the spent fuel racks maintain an acceptable geometry. It is also my understanding that any seismic walkdown assessment should be capable of providing reasonable assurance that seismic risk of a gross failure of the spent fuel pool to contain water is less than the low 10^{-6} mean annual frequency range. My review comments are based upon these understandings.

2. Background Information

The NRC Draft Technical Study of Spent Fuel Pool Accidents (Ref. 1) assumes that spent fuel pools are seismically robust. Furthermore, it is assumed that High-Confidence-Low-Probability-of Failure (HCLPF) seismic capacity of these pools is in the range of 0.4 to 0.5g peak ground acceleration (PGA). This HCLPF capacity (C_{HCLPF}) corresponds to approximately a 1% mean conditional probability of failure capacity ($C_{1\%}$), i.e.:

$$C_{HCLPF} \quad C_{1\%} \qquad (1)$$

as shown in Ref. 10.

In Ref. 5, detailed seismic fragility assessments have been conducted on the gross structural failure of spent fuel pools for two plants: Vermont Yankee (BWR), and Robinson (PWR). The following HCLPF seismic capacities are obtained from the fragility information in

Ref. 5:

$$\begin{aligned} &\text{Vermont Yankee (BWR):} && C_{HCLPF} = 0.48g \text{ PGA} \\ &\text{Robinson (PWR):} && C_{HCLPF} = 0.65g \text{ PGA} \end{aligned} \quad (2)$$

These two fragility estimates provide some verification of the HCLPF capacity assumption of 0.4 to 0.5g PGA used in Ref. 1.

I am confident that a set of seismic screening tables (seismic check lists) can be developed to be used with seismic walkdowns and drawing reviews to provide reasonable assurance that the HCLPF capacity of spent fuel pools is at least in the range of 0.4 to 0.5g PGA for spent fuel pools that pass such a review. However, in order to justify a HCLPF capacity in the range of 0.4 to 0.5g PGA, these screening tables will have rather stringent criteria so that I am not so confident that the vast majority of spent fuel pools will pass the screening criteria. The screening criteria (seismic check lists) summarized in Ref. 4 provides an excellent start. The subject of screening criteria is discussed more thoroughly in Section 3.

Once the HCLPF seismic capacity (C_{HCLPF}) has been estimated, the seismic risk of failure of the spent fuel pool can be estimated by either rigorous convolution of the seismic fragility (conditional probability of failure as a function of ground motion level) and the seismic hazard (annual frequency of exceedance of various ground motion levels), or by a simplified approximate method. This subject is discussed more thoroughly in Ref. 10.

A simplified approximate method is used in Ref. 1 to estimate the annual seismic risk of failure (P_F) of the spent fuel pool given its HCLPF capacity (C_{HCLPF}). The approach used in Ref. 1 is that:

$$P_F = 0.05\ H_{HCLPF} \quad (3)$$

where H_{HCLPF} is the annual frequency of exceedance of the HCLPF capacity. Ref. 1 goes on to state that for most Central and Eastern U.S. (CEUS) plants, the mean annual frequency of exceeding 0.4 to 0.5g PGA is on the order of or less than $2x10^{-5}$ based on the Ref. 8 hazard curves. Thus, from Eqn. (3), the annual frequency of seismic-induced gross failure (P_F) of the spent fuel pool is on the order of $1x10^{-5}$ or less for most CEUS plants.

Unfortunately, the approximation of Eqn. (3) is unconservative for CEUS hazard curves that have shallow slopes. By shallow slopes, I mean that it requires more than a factor of 2 increase in ground motion to correspond to a 10-fold reduction in the annual frequency of exceedance. For most CEUS sites, Ref. 8 indicates that a factor of 2 to 3 increase in ground motion is required to reduce the hazard exceedance frequency from $1x10^{-5}$ to $1x10^{-6}$. Over this range of hazard curve slopes, Eqn. (3) is always unconservative and will be unconservative by a factor of 2 to 4. Therefore, a HCLPF capacity in the range of 0.4 to 0.5g PGA is not sufficiently high to achieve a spent fuel pool seismic risk of failure on the order of $1x10^{-6}$ or less for most

CEUS plants. However, HCLPF capacities this high are sufficiently high to achieve seismic risk estimates less than $3x10^{-6}$ for most CEUS plants based upon the Ref. 8 hazard curves. This subject is further discussed in Section 4.

In lieu of using a simplified approximate method, Ref. 2 has estimated the seismic risk of spent fuel pool failure by rigorous convolution of the seismic fragility and seismic hazard estimates for the 69 CEUS sites for which seismic hazard curves are given in Ref. 8. Ref. 2 has divided the sites into 26 BWR sites and 43 PWR sites.

For the 26 BWR sites, Ref. 2 used the fragility curve defined in Ref. 5 for Vermont Yankee with the following properties:

<u>BWR Sites</u>
Median Capacity $C_{50} = 1.4$ PGA
HCLPF Capacity $C_{HCLPF} = 0.48g$ PGA (4)

Using the Ref. 8 seismic hazard estimates and the Eqn. (4) fragility, Ref. 2 obtained spent fuel pool mean annual failure probabilities ranging from $12.0x10^{-6}$ to $0.11x10^{-6}$ and averaging $1.6x10^{-6}$ for the 26 BWR sites. In my judgment, seismic screening criteria (seismic check lists) can be developed which are sufficiently stringent so as to provide reasonable assurance that the seismic capacity of spent fuel pools which pass the seismic screening roughly equals or exceeds that defined by Eqn. (4). With such a fragility estimate, based on the Ref. 8 seismic hazard estimates, for most CEUS sites, the estimated spent fuel pool seismic-induced failure probability will be less than $3x10^{-6}$ as further discussed in Section 4.

For the 43 PWR sites, Ref. 2 used the fragility curve defined in Ref. 5 for Robinson with the following properties:

<u>PWR Sites</u>
Median Capacity $C_{50} = 2.0$ PGA
HCLPF Capacity $C_{HCLPF} = 0.65g$ PGA(5)

Using the Ref. 8 seismic hazard estimates and the Eqn. (5) fragility, Ref. 2 obtained spent fuel pool mean annual failure probabilities ranging from $2.5x10^{-6}$ to $0.03x10^{-6}$ and averaging $0.48x10^{-6}$ for the 43 PWR sites. A fragility curve as high as that defined by Eqn. (5) is necessary to achieve an estimated spent fuel pool seismic-induced failure probability as low as $1x10^{-6}$ for nearly all CEUS sites. However, I don't believe realistic seismic screening criteria can be developed which are sufficiently stringent to provide reasonable assurance that the Eqn. (5) seismic fragility is achieved. In my judgment, a more rigorous seismic margin evaluation performed in accordance with the CDFM method described in Refs. 6 or 7 would be required to justify a HCLPF capacity as high as that defined by Eqn. (5).

3. Development and Use of Seismic Screening Criteria

Screening criteria are very useful to reduce the number of structure, system, and component (SSC) failure modes for which either seismic fragilities or seismic margin HCLPF capacities need to be developed. Screening criteria are presented in Ref. 6 for SSCs for which failures might lead to core damage. These screening criteria were established by an NRC sponsored "Expert Panel" based upon their review of seismic fragilities and seismic margin HCLPF capacities computed for these SSCs at more than a dozen nuclear power plants, and their review of earthquake experience data. These screening criteria were further refined in Ref. 7.

The screening criteria of Refs. 6 and 7 are defined for two seismic margin HCLPF capacity levels which will be herein called Level 1 and Level 2. Refs. 6 defines these two HCLPF capacity levels in terms of the PGA of the ground motion. However, damage to critical SSCs does not correlate very well to PGA of the ground motion. Damage correlates much better with the spectral acceleration of the ground motion over the natural frequency range of interest which is generally between 2.5 and 10 Hz for nuclear power plant SSCs. For this reason, Ref. 7 defines these same two HCLPF capacity levels in terms of the peak 5% damped spectral acceleration (PSA) of the ground motion. The two HCLPF capacity screening levels defined in Refs 6 and 7 are:

	HCLPF Screening Levels	
	Level 1	Level 2
PGA (Ref. 6)	0.3g	0.5g
PSA (Ref. 7)	0.8g	1.2g

These two definitions (PGA and PSA) are consistent with each other based upon the data upon which these screening levels are based. However, in my judgment, it is far superior to use the Ref. 7 PSA definition for the two screening levels when convolving a fragility estimate with CEUS seismic hazard estimates. For these CEUS seismic hazard estimates from Ref. 8, the ratio PSA/PGA generally lies in the range of 1.8 to 2.4 which is lower than the PSA/PGA ratio of the data from which the screening tables were developed. A more realistic and generally lower estimate of the annual probability of failure will result when the seismic fragility is defined in terms of PSA and convolved with a PSA hazard estimate in which the PSA hazard estimate is defined in the 2.5 to 10 Hz range.

In the past, a practical difficulty existed with defining the seismic fragility in terms of PSA instead of PGA. The Ref. 8 PSA hazard estimates are only carried down to 10^{-4} annual frequency of exceedance whereas the PGA hazard estimates are extended down to about 10^{-6}. Since it is necessary for the hazard estimate to be extended to at least a factor of 10 below the annual failure frequency being predicted, it has not been practical to use the PSA seismic fragility definition with the Ref. 8 hazard estimates. However, this difficulty has been overcome by Ref. 9 prepared by the Engineering Research Applications Branch of the Nuclear Regulatory Commission which extends the PSA seismic hazard estimates also down to 10^{-6}. Ref. 9 is attached herein as Appendix A.

In order to achieve a seismic induced annual failure probability P_F in the low 10^{-6} range for nearly all of the CEUS spent fuel pools with the Ref. 8 hazard estimates, it is necessary to apply the Level 2 screening criteria of Refs. 6 or 7, i.e., screen at a HCLPF seismic capacity of 1.2g PSA (equivalent to 0.5g PGA). The seismic screening criteria presented in Ref. 4 is properly based upon screening to Level 2. Furthermore, Ref. 4 appropriately summarizes the guidance presented in Ref. 7 for screening to Level 2. In general, I support the screening criteria defined in Ref. 4. However, I do have three concerns which are discussed in the following subsections.

3.1 Out-of-Plane Flexural and Shear Failure Modes for Spent Fuel Pool Concrete Walls and Floor

The screening criteria for concrete walls and floor diaphrams were developed to provide seismic margin HCLPF capacities based upon in-plane flexural and shear failures of these walls and diaphrams. For typical auxiliary buildings, reactor buildings, diesel generator buildings, etc., it is these in-plane failure modes which are of concern. For normal building situations, seismic loads are applied predominately in the plane of the wall or floor diaphram. Out-of-plane flexure and shear are not of significant concern. As one the primary authors of the screening criteria in both Refs. 6 and 7, I am certain that these screening criteria do not address out-of-plane flexure and shear failure modes.

For an aboveground spent fuel pool in which the pool walls (and floor in some cases) are not supported by soil backfill, it is likely that either out-of-plane flexure or shear will be the expected seismic failure mode. These walls and floor slab must carry the seismic-induced hydrodynamic pressure from the water in the pool to their supports by out-of-plane flexure and shear. It is true that these walls and floor are robust (high strength), but they may not be as ductile for out-of-plane behavior as they are for in-plane behavior. For an out-of-plane shear failure to be ductile requires shear reinforcement in regions of high shear. Furthermore, if large plastic rotations are required to occur, the tensile and compression steel needs to be tied together by closely spaced stirrups. I question whether such shear reinforcement and stirrups exist at locations of high shear and flexure in the spent fuel pool walls and floor. As a result, I suspect that only limited credit for ductility can be taken.

Without taking credit for significant ductility, it is not clear to me that spent fuel pool walls and floors not supported by soil can be screened at a seismic HCLPF capacity level as high as 1.2g PSA (equivalent to 0.5g PGA). I am aware of only one seismic fragility analysis having been performed on such unsupported spent fuel pool walls. That analysis was the Vermont Yankee spent fuel pool analysis reported in Ref. 5 for which the reported seismic HCLPF capacity was 0.48g PGA. A single analysis case does not provide an adequate basis for establishing a screening level for all other cases, particularly when the computed result is right at the desired screening level. The screening criteria in Refs 6 and 7 are based upon the review of many cases at more that a dozen plants.

In my judgement, it will be necessary to have either seismic fragility or seismic margin HCLPF computations performed on at least six different aboveground spent fuel pools with walls not supported by soil before out-of-plane flexure and shear HCLPF capacity screening levels can be established for such spent fuel pools.

3.2 Spent Fuel Pool Racks

I don't know whether a gross structural failure of the spent fuel racks is of major concern. This is a topic outside of my area of expertise. However, if such a failure is of concern, no seismic HCLPF capacity screening criteria is available for such a failure. The screening criteria of Refs. 6 and 7 were never intended to be applied to spent fuel pool racks. Since I have never seen a seismic fragility or seismic margin HCLPF capacity evaluation of a spent fuel pool rack, I have no basis for deciding whether these racks can be screened at a seismic HCLPF capacity as high as 1.2g PSA (equivalent to 0.5g PGA).

3.3 Seismic Level 2 Screening Requirements

In order to screen at a seismic HCLPF capacity of 1.2g PSA (0.5g PGA), the Level 2 screening criteria for concrete walls and diaphrams requires that such walls and diaphrams essentially comply with the ductile detailing and rebar development length requirements of either ACI 318.71 or ACI 349.76 or later editions. It is not clear to me how many CEUS spent fuel pool walls and floors essentially comply with such requirements since earlier editions of these codes had less stringent requirements. Therefore, it is not clear to me how many spent fuel pool walls and floors can actually be screened at Seismic Level 2 even for in-plane flexure and shear failure mode.

4. Seismic Risk Associated With Screening Level 2

4.1 Simplified Approaches for Estimating Seismic Risk Given the HCLPF Capacity

As mentioned in Section 2, the seismic risk of failure of the spent fuel pool can be estimated by either rigorous convolution of the seismic fragility and the seismic hazard, or by a simplified approximate method. The simplified approximate method defined by Eqn. (3) was used in Ref. 1. However, as also mentioned in Section 2, this approximate method understates the seismic risk by a factor of 2 to 4 for typical CEUS hazard estimates.

Ref. 10 presents an equally simple approach for estimating the seismic risk of failure of any component given its HCLPF capacity C_{HCLPF} and a hazard estimate. This approach tends to introduce from 0% to 25% conservative bias to the computed seismic risk when compared with rigorous convolution. Given the HCLPF capacity C_{HCLPF} this approach consists of the following steps:

Step 1: Estimate the 10% conditional probability of failure capacity $C_{10\%}$ from:

$$C_{10\%} = F_{\beta} C_{HCLPF} \quad (6)$$

$$F_\beta = e^{1.044\beta}$$

where β is the logarithmic standard deviation of the fragility estimate and 1.044 is the difference between the 10% non-exceedance probability (NEP) standard normal variable (-1.282) and the 1% NEP standardized normal variable (-2.326). F_β is tabulated below for various fragility logarithmic standard deviation β values.

β	Median/CDFM Capacity ($C_{50\%}/C_{CDFM}$)	F_β=($C_{10\%/CHCLPF}$)
0.3	2.01	1.37
0.4	2.54	1.52
0.5	3.20	1.69
0.6	4.04	1.87

For structures such as the spent fuel pool, β typically ranges from 0.3 to 0.5. Ref. 10 shows that over this range of β, the computed seismic risk is not very sensitive to β. Therefore, I recommend using a midpoint value for β of 0.4.

Step 2: Determine hazard exceedance frequency $H_{10\%}$, that corresponds to $C_{10\%}$ from the hazard curve.

Step 3: Determine seismic risk P_F from:

$$P_F = 0.5\, H_{10\%} \quad (7)$$

Table 1 presents the Peak Spectral Acceleration PSA seismic hazard estimates from Ref. 8 and 9 (LLNL93 results) for the Vermont Yankee and Robinson sites. In order to accurately estimate the seismic risk for a seismic HCLPF capacity C_{HCLPF} of:

$$C_{HCLPF} = 1.2\text{g PSA} = 1176 \text{ cm/sec}^2 \text{ PSA} \qquad (8)$$

associated with Screening Level 2 for the Vermont Yankee site by rigorous convolution, it is necessary to extrapolate the Ref. 9 hazard estimates down to the $2\text{x}10^{-8}$ exceedance frequency. Also, intermediate values in Table 1 have been obtained by interpolation.

Table 2 compares the seismic risk of spent fuel pool failure for these two sites as estimated by the following three methods:

1. Ref. 1 simplified approach, i.e., Eqn. (3).

2. Ref. 10 simplified approach, i.e., Steps 1 through 3 above.

3. Rigorous convolution of the hazard and fragility estimates.

For all three approaches the Screening Level 2 HCLPF capacity defined by Eqn. (8) was used. In addition, for both the Ref. 10 and rigorous convolution approaches, a fragility logarithmic standard deviation β of 0.4 was used.

From Table 2, it can be seen that the Ref. 1 method (Eqn. (3)) underestimates the seismic risk by factors of 2.3 and 3.5 for Vermont Yankee and Robinson, respectively. The simplified approach recommended in Ref. 10 and described herein overestimates the seismic risk by 20% and 5% respectively for these two cases. These results are consistent with the results I have obtained for many other cases.

4.2 Estimated Seismic Risk of Spent Fuel Pools Screened at Screening Level 2 Using Mean LL93 Hazard Estimates from Ref. 8 and 9

Using the Ref. 10 simplified approach described in the previous subsection, I have estimated the spent fuel pool seismic risk of failure corresponding to Screening Level 2 for all 69 CEUS sites with LLNL93 seismic hazard estimates defined in Refs. 8 and 9. These sites are defined in terms of an NRC site number code (OCSP_) used in Ref. 9. For each site, I assumed that the HCLPF capacity C_{HCLPF} was defined by Eqn. (8). A total of 35 of the 69 sites had estimated seismic risks of spent fuel pool failure associated with Screening Level 2 of greater than 1x10-6. The estimated seismic risk of 26 of these sites exceeded 1.25×10^{-6}. These 26 sites with their estimated seismic risk corresponding to Screening Level 2 are listed in Table 3. As can be seen in Table 3, only 8 of the 69 sites had estimated seismic risks of spent fuel pool failure exceeding 3×10^{-6}. One of these sites is Shoreham at which no fuel exists.

It should be noted that the seismic risks of spent fuel pool failure tabulated in Table 3 are based on the assumption that the HCLPF capacity of the spent fuel pool exactly equals the Screening Level 2 HCLPF capacity of 1.2g PSA (equivalent to 0.5g PGA). In actuality, spent fuel pools which pass the appropriately defined screening criteria are likely to have capacities higher than the screening level capacity. Therefore these are upper bound seismic risk estimates for spent fuel pools that pass the to-be established screening criteria. Furthermore, the simplified approach used to estimate the seismic risks in Table 3 overestimates these risks by 0% to 25%.

4.3 Estimated Seismic Risk of Spent Fuel Pools Screened at Screening Level 2 Using Mean EPRI89 Hazard Estimates

Following the exact same Ref. 10 simplified approach which I followed for the LLNL93 hazard estimates, Ref. 11 provides the corresponding seismic risk of spent fuel pool failure estimates based upon EPRI89 hazard estimates for 60 of the 69 CEUS sites. Table 3 shows the corresponding seismic risk computed in Ref. 11 for the EPRI89 hazard estimates.

From Table 3, it can be seen that the EPRI89 hazard estimates produce generally much lower seismic risk estimates corresponding to Screening Level 2 than do the LLNL93 hazard estimates. Based on the EPRI89 hazard estimates, only one site has a seismic risk exceeding 1×10^{-6}. Only three other sites have seismic risks exceeding 0.5×10^{-6}. Table 3 includes all sites for which the computed seismic risk exceeds 0.5×10^{-6} based on the mean EPRI89 hazard estimates.

5. Conclusions

If based on the mean LLNL93 hazard estimates (Ref. 8 and 9) it is acceptable to have up to a mean $3x10^{-6}$ annual seismic risk of spent fuel pool failure at the screening level, then Screening Level 2 defined in Section 3 represents a practical screening level. Only 8 of the 69 sites have computed seismic risks greater than $3x10^{-6}$ at this screening level. Screening Level 2 is set at a peak 5% damped spectral acceleration (PSA) level of 1.2g (equivalent to a PGA level of 0.5g).

Based on the mean EPRI89 hazard estimates (Ref. 11), Screening Level 2 would generally result in seismic risk of spent fuel pool failure estimates less than $0.5x10^{-6}$ for spent fuel pools which passed the screening criteria. Only 4 out of 60 sites have computed seismic risks greater than $0.5x10^{-6}$ at this screening level.

The screening criteria given in Refs. 4 and 7 represent a good start on developing screening criteria for spent fuel pools at Screening Level 2. However, I have three significant concerns which are discussed in Sections 3.1 through 3.3. In my judgment, a detailed fragility review of a few spent fuel pools will be necessary in order to address my concerns. These reviews should concentrate on aboveground spent fuel pools with walls not backed by soil backfill. I believe these reviews need to be performed before a set of screening criteria can be finalized at Screening Level 2.

References

1. *Preliminary Draft Technical Study of Spent Fuel Pool Accidents for Decommissioning Plants,* Nuclear Regulatory Commission, June 16, 1999

2. *Draft EPRI Technical Report: Evaluation of Spent Fuel Pool Seismic Failure Frequency in Support of Risk Informed Decommissioning Energy Planning*, Duke Engineering and Services

3. *A Review of Draft NRC Staff Report: Draft Technical Study of Spent Fuel Pool Accidents for Decommissioning Plants,* NEI, August 27, 1999

4. *Seismic Screening Criteria for Assessing Potential Fuel Pool Vulnerabilities at Decommissioning Plants,* NEI, August 18, 1999

5. *Seismic Failure and Cask Drop Analyses of the Spent Fuel Pools at Two Representative Nuclear Power Plants,* NUREG/CR-5176, Prepared for Nuclear Regulatory Commission, January 1989

6. *An Approach to the Quantification of Seismic Margins in Nuclear Power Plants,* NUREG/CR-4334, Prepared for Nuclear Regulatory Commission, August 1985

7. *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1),* (EPRI NP-6041-SL), August 1991

8. *Revised Livermore Seismic Hazard Estimates for 69 Nuclear Power Plant Sites East of the Rocky Mountains,* NUREG-1488, Nuclear Regulatory Commission, October 1993

9. *Extension to Longer Return Periods of LLNL Spectral Acceleration Seismic Hazard Curves for 69 Sites,* provided by Engineering Research Applications Branch, Nuclear Regulatory Commission, September, 1999

10. Kennedy, R.P., *Overview of Methods for Seismic PRA and Margin Assessments Including Recent Innovations,* CSNI Seismic Risk Workshop, Tokyo, Japan, August 1999

11. Personal Communication from Tom O'Hara, Duke Engineering and Services to Robert Kennedy, October 19, 1999

Table 1
Seismic Hazard Estimates for Peak Spectral Acceleration for PSA
From Refs. 8 and 9 (LLNL 93 Results)

	Peak Spectral Acceleration PSA ($cm/sec.^2$)		
Exceedance Frequency H	Vermont Yankee	Robinson	
$1x10^{-3}$	93	232	
$5x10^{-4}$	151	369	
$2x10^{-4}$	246	676	
$1x10^{-4}$	354	991	
$5x10^{-5}$	501	1349	*
$2x10^{-5}$	759	2054	*
$1x10^{-5}$	1058	2801	
$5x10^{-6}$	1396	3915	*
$2x10^{-6}$	1884	6096	*
$1x10^{-6}$	2308	8522	
$5x10^{-7}$	2661	--	**
$2x10^{-7}$	3330	--	**
$1x10^{-7}$	3802	--	**
$5x10^{-8}$	4266	--	**
$2x10^{-8}$	5248	--	**

* By Interpolation

** By Extrapolation

Table 2
Comparison of Seismic Risk Estimated by Various Approaches

$C_{HCLPF} = 1.2g$ PSA, $\beta = 0.4$

	Computed Seismic Risk P_F (to be multiplied by 10^{-6})		
Site	Ref. 1 Method Eqn. (3)	Ref. 10 Method Steps 1 through 3	Rigorous Convolution
Vermont Yankee	0.38	1.07	0.89
Robinson	3.7	13.6	13.0

Table 3
<u>Seismic Risk Associated With Screening Level 2</u>

C_{HCLPF} = 1.2g Peak Spectral Acceleration

Site Number	Annual Seismic-Induced Probability of Failure P_F (to be multiplied by 10^{-6})	
	LLNL93 Hazard	EPRI89 Hazard
36	13.6	0.14
18	8.3	1.9
25	6.6	0.57
8	5.5	0.21
43	4.5	0.12
59	4.4	*
21	4.2	*
62	4.1	*
27	2.9	0.38
49	2.8	0.27
40	2.5	0.10
16	2.5	0.14
38	2.3	0.21
63	2.2	0.06
54	2.2	0.26
19	1.8	0.17
32	1.8	0.17
28	1.7	0.04
4	1.6	*
50	1.5	0.20
44	1.5	*
20	1.5	0.55
31	1.4	0.06
39	1.4	0.14
14	1.3	0.60
13	1.3	0.33

Not Available

5c Huffman Memo to Richards with Staff Evaluation of Screening Criteria, December 3, 1999

December 3, 1999

MEMORANDUM TO: Stuart A. Richards, Director
Project Directorate IV & Decommissioning
Division of Licensing Project Management
Office of Nuclear Reactor Regulation

FROM: William C. Huffman, Project Manager/S/ P. RAY FOR
Decommissioning Section
Project Directorate IV & Decommissioning
Division of Licensing Project Management
Office of Nuclear Reactor Regulation

SUBJECT: SCREENING CRITERIA FOR ASSESSING POTENTIAL SEISMIC VULNERABILITIES OF SPENT FUEL POOLS AT DECOMMISSIONING PLANTS

The staff is in the process of preparing a final draft of its technical study on spent fuel pool accident risks at decommissioning plants. This final draft will be issued for public comment in early January 2000. Included in this report will be a discussion on risks from a large seismic event that exceeds the structural capacity of the spent fuel pool to the extent that a catastrophic failure occurs. Such a failure would result in rapid draining of the spent fuel pool with no capability of retaining water even if reflooded. The staff has previously acknowledged that spent fuel pools are inherently robust and can withstand loads substantially beyond those for which they were designed. Consequently, they have a significant seismic capacity. To take credit for the seismic design margins existent in spent fuel pools, the staff sought an appropriate method to identify potential structural vulnerabilities without having to perform a detailed fragility review. At a public workshop conducted on July 15-16, 1999, development of a simple spent fuel pool seismic screening checklist was proposed as way of assessing the seismic vulnerabilities of spent fuel pools without performing quantifying analyses. In a letter to the staff dated August 18, 1999, the Nuclear Energy Institute (NEI) proposed a "seismic checklist" for screening potential spent fuel pool structural vulnerabilities on a plant-specific basis. Based on the staff's recent input to the final draft report, the use of a checklist is considered to be an excellent approach to plant-specific seismic assessments; however, some deficiencies have been identified in the checklist proposed by NEI. The nature of the deficiencies with the current version of the checklist was generally discussed in a public meeting with NEI and other stakeholders on November 19, 1999. NEI indicated that it needed additional details on the staff's findings relative to the checklist in order to propose effective improvements.

The Attachment to this memorandum contains additional details on the deficiencies the staff has found with use of the current seismic checklist. Copies of this memorandum with the attached information will be provided to NEI and all other interested stakeholders in an effort to

further the dialogue relating to the seismic checklist and support the development of additional modifications that will resolve the deficiencies currently identified.

For comments to be considered for the draft report that will be issued in January 2000 for public comment, written comments must be received by the staff no later than December 13, 1999. Comments received after December 13, 1999, will be addressed in the final report that will be issued in early April 2000. The NRC staff contact for public comments is Mr. William Huffman. Mr. Huffman can be reached at (301) 415-1141.

Attachment: As stated

cc w/att: See next page

further the dialogue relating to the seismic checklist and support the development of additional modifications that will resolve the deficiencies currently identified.

For comments to be considered for the draft report that will be issued in January 2000 for public comment, written comments must be received by the staff no later than December 13, 1999. Comments received after December 13, 1999, will be addressed in the final report that will be issued in early April 2000. The NRC staff contact for public comments is Mr. William Huffman. Mr. Huffman can be reached at (301) 415-1141.

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Mr. Ralph Beedle
Senior Vice President
and Chief Nuclear Officer
Nuclear Energy Institute
1776 I Street, NW, Suite 400
Washington, DC 20006-3708

Mr. Alex Marion, Director
Programs
Nuclear Energy Institute
1776 I Street, Suite 400
Washington, DC 20006-3708

Mr. David Modeen, Director
Engineering
Nuclear Energy Institute
1776 I Street, NW, Suite 400
Washington, DC 20006-3708

Mr. Anthony Pietrangelo, Director
Licensing
Nuclear Energy Institute
1776 I Street, NW, Suite 400
Washington, DC 20006-3708

Mr. Jim Davis, Director
Operations
Nuclear Energy Institute
1776 I Street, NW, Suite 400
Washington, DC 20006-3708

Ms. Lynnette Hendricks, Director
Plant Support
Nuclear Energy Institute
1776 I Street, NW, Suite 400
Washington, DC 20006-3708

Mr. H. A. Sepp, Manager
Regulatory and Licensing Engineering
Westinghouse Electric Company
P.O. Box 355
Pittsburgh, PA 15230-0355

Mr. Charles B. Brinkman, Director
Washington Operations
ABB-Combustion Engineering, Inc.
12300 Twinbrook Parkway, Suite 330
Rockville, MD 20852

Mr. Michael Meisner
Maine Yankee Atomic Power Co.
321 Old Ferry Road
Wiscassett, ME 04578-4922

Mr. Paul Blanch
Energy Consultant
135 Hyde Road
West Hartford, CT 06117

New England Coalition on Nuclear
Pollution
P. O. Box 545
Brattleboro, VT 05302

Mr. Ray Shadis
Friends of the Coast
P. O. Box 98
Edgecomb, ME 04556

Mr. David Lochbaum
Union of Concerned Scientists
1616 P St. NW, Suite 310
Washington, DC 20036

Mr. Paul Gunter
Nuclear Information Resource Service
1424 16th St. NW, Suite 404
Washington, DC 20036

Mr. Peter James Atherton
P.O. Box 2337
Washington, DC 20013

Mr. H. G. Brack
Center for Biological Monitoring
P.O. Box 144
Hull's Cove, ME 04644

Ms. Deborah B. Katz
Citizen's Awareness Network
P. O. Box 3023
Charlemont, MA 01339-3023

Mr. Robert Holden
National Congress of American Indians
Suite 200

1301 Connecticut Ave, NW
Washington DC, 20036

Structural Failure Modes

Amongst the various ways a pool structure can fail, the only failure modes that are of concern are those that involve pool floor slab failure, failure of side walls at the bottom of the pool or at the bottom corners. It is important to ensure that the structural integrity assessment is based on realistic failure modes for catastrophic loss of structural integrity. This should take into account physical interactions with adjacent structures and equipment.

For PWR spent fuel pools, the pool floor slab is not likely to fail except through the effect of local concrete spalling due to foundation uplift and impact with the subgrade or adjacent structures. Failure of walls in partially embedded pools is not likely. Bending moment capacity of the pool walls is very much dependent on reinforcing patterns and the walls are generally reinforced in an orthotropic pattern, such that the resistance in the horizontal and vertical directions are unequal. The resistance is also unequal between one wall and another wall. This requires a case by case assessment of the bending capacity of walls.

For BWR spent fuel pools, the floor slab, walls and supporting columns and shear walls need scrutiny to determine the critical failure mode. As in the case of PWR spent fuel pools, the effect of adjacent structures and equipment on structural failure needs to be evaluated.

The stainless steel liner plate is used to assure leak-tightness; cracks in the welded seams are not likely to lead to catastrophic loss of water inventory unless there is a simultaneous massive failure of the concrete structure.

The emphasis here is that spent fuel pool structures not only vary in layout and elevation between PWRs and BWRs, they can also vary within each group. The process of realistic assessment of structural capacity of pool structures begins with a methodical consideration of likely failure modes associated with a catastrophic loss of integrity.

The efforts involved in the assessment of seismic capacity of pool structures typically consist of the following:

- Inspect the pool structure and its vicinity and note:

    - physical condition such as cracking and spalling of concrete, signs of leakage or leaching and separation of pool walls from the grade surface, potential for piping connections, either buried underground or above ground, to fail due to a large seismic excitation or interaction with adjacent equipment, and cause drainage of the pool below the safety level of the pool water,

    5. arrangement and layout of supporting columns and shear walls, assessment of other loads from tributary load areas carried by the supporting structure of the pool, as-built dimensions and mapping of any existing structural cracks,

    6. adjacent structures that can impact the pool structure both above and below the grade surface, supporting arrangement for superstructure and crane and potential for failure of the superstructure and the crane, potential impact from heavy objects that can drop in the pool structure and the corresponding drop heights.

ATTACHMENT

- Seismic capacity assessments of the pool structure typically consist of the following:

- review existing layout drawings and structural dimensions and reconcile the differences, if any, between the as-built and as designed information and consider the effects of structural degradation as appropriate,

- from design calculations determine the margin to failure and assess the extrapolated multiple of SSE level that the pool structure could survive, determine whether or not design dynamic response analysis including soil-structure interaction effects are still applicable at the capacity level seismic event; if not, conduct a new analysis using properties of soil at higher strain levels and reduced stiffness of cracked reinforced concrete,

- determine the loads from pool structure foundation uplift and from impact of pool structure with adjacent structures during the capacity level seismic event, determine loads from the impact of a spent fuel rack on the pool floor and the side walls and determine the loads from dropping of heavy objects from the collapse of a superstructure or the overhead crane,

- determine a list of plausible failure modes; failure of side walls due to the worst loading from the capacity level earthquake in combination with fluid hydrostatic and sloshing head and dynamic earth pressure as appropriate, failure of the pool floor slab in flexure and bending due to loads from the masses of water and the spent fuel and racks, local failure by punching shear due to impact between structures and the spent fuel racks or dropping of heavy objects,

- the assessments to determine the lowest structural capacity can be based on ultimate strength of reinforced concrete structures due to flexure, shear and punching shear. When conducting a yield line analysis, differences in flexural yield capacities for the negative and positive bending moments in two orthogonal directions influence the crack patterns, and several sets of yield lines may have to be investigated to obtain the lowest capacity. For heterogeneous materials, the traditional yield line analysis provides upper bound solutions; consequently, considerable skill is needed to determine the structural capacity based on the yield lines that approximate the lower bound capacity.

Although the inspection of the pool structure is an essential part of establishing that the structure is in sound condition, some of the other attributes of a detailed capacity evaluation, as discussed above, may only be undertaken for plants that do not pass simple examination using a seismic checklist. Such an effort may be necessary for plants in high seismic hazard areas.

Other Considerations

NRC sponsored studies have treated the assessment of seismic capacity of spent fuel pools relying on the seismic margins method to determine the high confidence of low probability (less than 5% failure) of failure (HCLPF). The HCLPF value for a structural failure may well be unrealistic and unnecessarily conservative in terms of an instantaneous loss of water inventory.

This point needs to be emphasized because the shear and moment capacity of the walls and slabs are determined by using upper limits of allowable stresses. In the study which resulted in NUREG/CR 4982, the seismic capacities were based on the Oyster Creek reactor building and a shear wall from the Zion auxiliary building. For elevated pool structures, the Oyster Creek estimate may be an acceptable approximation, but the Zion shear wall may be too highly simplified to substitute for the catastrophic failure of the spent fuel pool structure. However, it is important to emphasize that out of plane loading on the pool walls from the hydrostatic head of the pool water can lead to flexure and shear-induced failures. Relatively low margin on allowable out-of-plane shear strength combined with the uncertainty of the extent to which reinforcement details ensure ductile behaviors make it imperative to ensure that seismic capacities of the pool walls and slab elements are adequate. The stainless steel pool liner was not designed to resist any structural load; nevertheless, it can provide substantial water-retaining capacity near the bottom half of the pool where structural deformations are likely to be low from seismic loading (this is due to the aspect ratio of the pool walls which are thick and form a deep box shape) except in a highly unlikely failure mode, such as puncturing the pool slab or the wall near the bottom of the pool.

For PWR pools that are fully or partially embedded, an earthquake motion that could cause a catastrophic failure is very high and is not a credible event. However, interaction with adjacent structures and equipment may have to be evaluated to determine the structural capacity on a case-by-case basis.

For BWR pools, the seismic capacity is likely to be somewhat less than that of a PWR pool and can vary significantly from one plant to another. This is because for most BWR pools that are at higher elevation there is amplification of seismic motion, and the pool floor may not be supported on the subgrade. Shear failure of the pool floor can occur at a relatively lower level of seismic input for BWR pools. More important, a combination of the hazard and the spent fuel pool structural capacity can bring down the likelihood of a catastrophic structural failure to a negligible risk. On the other hand, plant-specific hazard and seismic fragility of spent fuel pools can combine to produce a risk that needs to be examined on a case-by-case basis.

Using the data from NUREG-1488 (new Lawrence Livermore National Laboratory (LLNL) data) for currently operating plants in the eastern and central United States, the mean probability of exceedance (POE) of the peak ground acceleration values for the SSE were examined. The plant grouping approach, Reduced Scope, Focused Scope, Full Scope, etc., used in NUREG-1407, "Procedural and Submittal Guidance for the Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities" Final Report was also reviewed. The objective of plant grouping for IPEEE was to put plants into groups with similar seismic vulnerability; consequently, it was useful to look at these plant groups. However, the evaluation in this draft study is driven by the 1993 LLNL seismic hazard results, and it was determined that, except for a small number of plants, the POEs for SSE are lower than $1X10^{-4}$ per reactor year and for three times the SSE, the POEs are below $1X10^{-5}$. For these plants, the likelihood of a catastrophic pool structure failure at a HCLPF value of three times the SSE should be less than $5X10^{-7}$. This makes the simplifying assumption that the conditional probability of failure (POF) or reaching the end state of a structure is $5X10^{-2}$. In this approach there is confidence that the seismic hazard is low (at three times the SSE) and there is also a plant specific structural assessment of the HCLPF value which is more than or equal to three times the SSE.

For spent fuel pools located at sites that meet the HCLPF value of three times the SSE, a catastrophic structural failure from an earthquake much larger than the design basis SSE is not credible. However, this approach may not be feasible at sites where the likelihood of the spent fuel pool structure failure due to beyond design basis earthquake is higher. For such sites in the eastern United States, a more detailed examination of the probability of the earthquake, a realistic assessment of the ground motion caused by the event at the site and the structural capacity of the spent fuel pool structure may be necessary.

NEI Draft Seismic Checklist

The draft checklist provided in an NEI letter to the staff postmarked August 18, 1999, includes seven elements that identify areas of potential weaknesses. The use of such a checklist would ensure that potential vulnerabilities are either rectified or mitigation measures are put in place. The checklist is quite comprehensive. But it can be improved by taking into account out-of-plane shear capacity of shear walls such as those that form the pool when they are not backed up by backfill. Other considerations might include pre-existing degradation of concrete and the liner plate. With minor modifications the checklist can be finalized.

Kennedy Report

As a part of an independent technical review, Dr. Robert P. Kennedy was requested to conduct this review. This review activity was supported by the Office of Nuclear Regulatory Research, Division of Engineering Technology. Dr. Kennedy attended the public workshop on July 16, 1999. The report does endorse the feasibility of the use of the seismic screening concept and identifies eight sites by site numbers for which seismically induced probability of failure (POF) is greater than $3x10^{-6}$ using the LLNL 93 Hazard. It is important to recognize that sites where POF is greater than $3x10^{-6}$, in addition to the use of the seismic checklist, an evaluation of the POF using plant-specific fragility information will be necessary. For all other sites, the use of the seismic checklist should be adequate. Appropriate excerpts of the Kennedy Report are contained in the Enclosure.

Recommendation

The following actions are recommended:

1. The seismic checklist should consider out of plane shear and flexure.

2. Identification of preexisting concrete and liner plate degradation be added to the checklist.

3. The checklist should be augmented to discuss potential mitigation measures for vulnerabilities that may be identified.

4. Higher seismic hazard sites in the Eastern U.S., should be further evaluated by the industry to determine (a) a list of such sites, (b) a credible ground motion description at which the seismic hazard frequency is low enough at these sites, and (c) plant specific seismic capacity evaluation using credible ground motion description at the site.

5. Proposed treatment of sites West of the Rocky Mountains

NOTE: Additional supplemental information from the Kennedy report is included in the following pages.

5d Nelson Letter to Huffman with Revised Criteria, December 13, 1999

NUCLEAR ENERGY INSTITUTE

Alan Nelson
SENIOR PROJECT
MANAGER,
PLANT SUPPORT
NUCLEAR
GENERATION
DIVISION

December 13, 1999

Mr. William C. Huffman
Project Manager
Decommissioning Section
Projects Directorate IV & Decommissioning
U.S. Nuclear Regulatory Commission
Mail Stop 11 D19
Washington, DC 20555-0001

Dear Mr. Huffman:

On July 15-16, 1999, the NRC held a workshop on spent fuel accidents at decommissioning plants. During the course of the workshop, presentations by the NRC and the industry concluded that spent fuel pools possess substantial capability beyond their design basis to withstand seismic events but that variations in seismic capacity existed due to plant specific designs and locations.

NEI forwarded "Seismic Screening Criteria for Assessing Potential Pool Vulnerabilities at Decommissioning Plants, to the NRC " August 18, 1999 for review and comment. Based on NRC review, the staff proposed additional details to the submitted checklist. Detailed NRC comments were made available on December 3, 1999 "Screening Criteria for Assessing Potential Seismic Vulnerabilities of Spent Fuel Pools at Decommissioning Plants."

Enclosed is the revised screening criteria addressing the December 3, 1999 NRC memorandum. We believe the revision addresses the deficiencies identified. We request that the revised checklist be considered as the NRC prepares its draft report to be issued in January 2000.

Please contact me at (202) 739-8110 or by e-mail (apn@nei.org) if you have any questions or if you would like to schedule a meeting to discuss industry's response to the staff's recommendations. .

Sincerely,

Alan Nelson
APN/dc
Enclosure

Seismic Screening Criteria

for

Assessing Potential Fuel Pool Vulnerabilities

at

Decommissioning Plants

December 13, 1999
Revision 1

Background

To increase the efficiency and effectiveness of decommissioning regulations, the NRC staff has engaged in rulemaking activities that would reduce the need to routinely process exemptions once a plant is permanently shut down. With this goal in mind, members of the NRC staff, industry representatives and other stakeholders held a two-day workshop on risk related spent fuel pool accidents at decommissioning plants.

At this workshop, based upon presentations by the NRC staff (Goutam Bagchi et al.) and the nuclear industry (T. O'Hara - DE&S), it was concluded that a large seismic event (in the range of three times the design level earthquake) would represent a risk of exceeding the structural capacity of the spent fuel pool and thus potentially result in draining the pool.

Although the methodologies presented by the NRC staff and the industry differed somewhat, they both concluded that, in general, spent fuel pools possess substantial capacity beyond their design basis but that variations in seismic capacity existed due to plant specific details (i.e. " Differences in seismic capacity due to spent fuel location and other details.").

The consensus was that the risk was low enough that precise quantification was not necessary to support exemption requests but that this needed to be confirmed on a plant specific basis with deterministic criteria. It was recommended that a simple spent fuel pool (SFP) vulnerability check list be developed to provide additional assurance that no beyond-design-basis seismic structural vulnerabilities exist at decommissioning plants. A draft seismic screening checklist was provided to the Staff by NEI in August 1999. Comments on this draft were discussed during a conference call held on December 7, 1999 and the following draft screening checklist has been revised to address the issues raised..

Purpose of Checklist

As discussed briefly in the "Background" section, the purpose of this checklist is to identify and evaluate specific seismic characteristics which might result in a specific spent fuel pool from not being capable of withstanding, without catastrophic failure, a beyond-design-basis seismic event equal in magnitude to approximately three times its design basis. Completion of the requirements will be performed by a qualified seismic engineer. This effort will include a thorough SFP walkdown and a review of appropriate SFP design drawings.

DRAFT CHECKLIST

Item 1:

Requirement: Identify Preexisting Concrete and Liner Plate Degradation

Basis: A detailed review of plant records concerning spent fuel pool concrete and liner plate degradation should be performed and supplemented by a detailed walkdown of the accessible portions of the spent fuel pool concrete and liner plate. The purpose of the records review and visual inspection activities is to accurately assess the material condition of the SFP concrete and liner in order to assure that these existing material conditions are properly factored into the remaining seismic screening assessments.

Design Feature: The material condition of the SFP concrete and liner, based upon the records review and the walkdown inspection, will be documented and used as an engineering input to the following seismic screening assessments.

Item 2:

Requirement: Assure Adequate Ductility of Shear Wall Structures

Basis: The expert panel involved with the development of Reference 1 concluded that, " For the Category 1 structures which comply with the requirements of either ACI 318-71 or ACI 349-76 or later building codes and are designed for an SSE of at least 0.1g pga, as long as they do not have any special problems as discussed below, the HCLPF capacity is at least 0.5g pga." This conclusion was based upon the assumption that the shear wall structure will respond in a ductile manner. The "special problems" cited deal with individual plant details which could prevent a particular plant from responding in the required ductile fashion. Examples cited in Reference 1 included an embedded structural steel frame in a common shear wall at the Zion plant (which was assumed to fail in brittle manner due to a potential shear failure of the attached shear studs) and large openings in a "crib house" roof (also at the Zion plant) which could interrupt the continuity of the structural slab.

Other examples which could impact the ductility of the spent fuel pool structure include large openings which are not adequately reinforced or reinforcing bars that are not sufficiently embedded to prevent a bond failure before the yield capacity of the steel is reached.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 3:

Requirement: **Assure Design adequacy of Diaphragms (including roofs)**

Basis: In the design of many nuclear power plants, the seismic design of roof and floor diaphragms has often not received the same level of attention as have the shear walls of the structures. Major cutouts for hatches or for pipe and electrical chases may pose special problems for diaphragms. Since more equipment tends to be anchored to the diaphragm compared to shear walls, moderate amounts of damage may be more critical for the diaphragm compared to the same amount of damage in a wall.

Based upon the guidance provided in Reference 1, diaphragms for Category I structures designed for a SSE of 0.1g or greater do not require an explicit evaluation provided that: (1) the diaphragm loads were developed using dynamic analysis methods; (2) they comply with the ductility detailing requirements of ACI 318-71 or ACI 349-76 or later editions. Diaphragms which do not comply with the above ductility detailing or which did not have loads explicitly calculated using dynamic analysis should be evaluated for a beyond-design-basis seismic event in the 0.45-0.5g pga range.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 4:

Requirement: **Verify the Adequacy of the SFP Walls and Floor Slab to Resist Out-of-Plane Shear and Flexural Loads**

Basis: For PWR pools that are fully or partially embedded, an earthquake motion that could cause a catastrophic out-of-plane shear or flexural failure is very high and is not a credible event. For BWR pools (and PWR pools that are not at least partially embedded), the seismic capacity is likely to be somewhat less and the potential for our-of-plane shear and/or flexural wall or base slab failure, at beyond-design-basis seismic loadings, is possible.

A structural assessment of the pool walls and floor slab out-of plane shear and flexural capabilities should be performed and compared to the realistic loads expected to be generated by a seismic event equal to approximately three times the site SSE. This assessment should include dead loads resulting from the masses of the pool water and racks, seismic inertial forces, sloshing effects and any significant impact forces.

Credit for out-of-plane shear or flexural ductility should not be taken unless the reinforcement associated with each failure mode can be shown to meet the ACI 318-71 or ACI 349-49 requirements.

Design Feature: Compliance with this design feature will be documented based upon a review of drawings (in the case of embedded or partially embedded PWR pools) or based upon a review of drawings coupled with the specified beyond-design-basis shear and flexural calculations outlined above.

Item 5:

Requirement: **Verify the Adequacy of Structural Steel (and Concrete) Frame Construction**

Basis: At a number of older nuclear power plants, the walls and roof above the top of the spent fuel pool are constructed of structural steel. These steel frames were generally designed to resist hurricane and tornado wind loads which exceeded the anticipated design basis seismic loads. A review of these steel (or possibly concrete) framed structures should be performed to assure that they can resist the seismic forces resulting from a beyond-design-basis seismic event in the 0.45-0.5g pga range. Such a review of steel structures should concentrate on structural detailing at connections. Similarly, concrete frame reviews should concentrate on the adequacy of the reinforcement detailing and embedment.

Failure of the structural steel superstructure should be evaluated for its potential impact on the ability of the spent fuel pool to continue to successfully maintain its water inventory for cooling and shielding of the spent fuel.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 6:

Requirement: **Verify the Adequacy of Spent Fuel Pool Penetrations**

Basis: The seismic and structural adequacy of any spent fuel pool (SFP) penetrations whose failure could result in the draining or syphoning of the SFP must be evaluated for the forces and displacements resulting from a beyond-design-basis seismic event in the 0.45-0.5g pga range. Specific examples include SFP gates and gate seals and low elevation SFP penetrations, such as, the fuel transfer chute/tube and possibly piping associated with the SFP cooling system. Failures of any penetrations which could lead to draining or syphoning of the SFP should be considered.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 7:

Requirement: **Evaluate the Potential for Impacts with Adjacent Structures**

Basis: Structure-to-structure impact may become important for earthquakes significantly above the SSE, particularly for soil sites. Structures are usually conservatively designed with rattle space sufficient to preclude impact at the SSE level but there are no set standards for margins above the SSE. In most cases, impact is not a serious problem but, given the potential for impact, the consequences should be addressed. For impacts at earthquake levels below 0.5g pga, the most probable damage includes the potential for electrical equipment malfunction and for local structural damage. As cited previously, these levels of damage may be found to be acceptable or to result in the loss of SFP support equipment. The major focus of this impact review is to assure that the structure-to-structure impact does not result in the inability of the SFP to maintain its water inventory.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 8:

Requirement: **Evaluate the Potential for Dropped Loads**

Basis: A beyond-design-basis seismic event in the 0.45-0.5g pga range has the potential to cause the structural collapse of masonry walls and/or equipment supports systems. If these secondary structural failures could result in the accidental dropping of heavy loads which are always present (i.e. not loads associated with cask movements) into the SFP, then the consequences of these drops must be considered. As in previous evaluations, the focus of the drop consequence analyses should consider the possibility of draining the SFP. Additionally, the evaluation should evaluate the consequences of any resulting damage to the spent fuel or to the spent fuel storage racks.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 9:

Requirement: **Evaluation of Other Failure Modes**

Basis Experienced seismic engineers should review the geotechnical and structural design details for the specific site and assure that there are not any design vulnerabilities which will not be adequately addressed by the review areas listed above. Soil-related failure modes including liquefaction and slope instability should be screened by the approaches outlined in Reference 1 (Section 7 & Appendix C).

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 10: **Potential Mitigation Measures**

Although beyond the scope of this seismic screening checklist, the following potential mitigation measures may be considered in the event that the requirements of the seismic screening checklist are not met at a particular plant.

a.) Delay requesting the licensing waivers (E-Plan, insurance, etc.) until the plant specific danger of a "zirc-fire" is no longer a credible concern.

b.) Design and install structural plant modifications to correct/address the identified areas of non-compliance with the checklist. (It must be acknowledged that this option may not be practical for significant seismic failure concerns.)

c.) Perform plant-specific seismic hazard analyses to demonstrate that the seismic risk associated with a catastrophic failure of the pool is at an acceptable level. (The exact "acceptable" risk level has not been precisely quantified but is believed to be in the range of 1.0E-06.)

Item 11: **Required Documentation**

A simple report describing the results of the seismic engineer's walkdown and drawing review findings is judged to provide sufficient documentation to rule out a beyond-design-basis seismic event as a significant risk contributor to a decommissioned nuclear power plant.

References:

1. "A Methodology for Assessment of Nuclear Power Plant Seismic Margin Revision 1)," (EPRI NP-6041-SL), August 1991

5e The "Industry Comments" Referred to in the December 28, 1999 Kennedy letter

Comments on NRC Draft Screening Criteria for Assessing Potential Seismic Vulnerabilities of Spent Fuel Pools at Decommissioning Plants - December 3, 1999 NRC Memorandum

Summary of NRC Draft

To increase the efficiency and effectiveness of decommissioning regulations, the NRC staff has engaged in rulemaking activities that would reduce the need to routinely process exemptions once a plant is permanently shut down. The December 3, 1999 memorandum from W. Huffman to S. Richards (Reference 1) provides a summary of the staff's current concerns regarding a screening criteria for assessing potential seismic vulnerabilities to spent fuel pools (SFP) at decommissioning plants. Attachments to this memorandum contain suggested enhancements to the proposed seismic checklist and also excerpts from an independent technical review by Dr. Robert Kennedy. The report by Kennedy endorsed the feasibility of the use of a seismic screening concept. The Kennedy report identified eight sites for which the seismically induced probability of SFP failure is greater than 3.0×10^{-6} using the LLNL 93 hazard data.

The seismic risk of failure of the spent fuel pool can be estimated by rigorously convolving a family of fragility curves with a family of seismic hazard curves (Reference 2), or by simplified approximation methods. Two simplified methods are described in the attachments to the December 3, 1999 memorandum (Reference 1).

The first simplified method was presented by the Staff in their preliminary draft of June 16, 1999 (Reference 3). This method is based on use of the SFP high confidence low probability of failure (HCLPF) value and the simplifying assumption that the conditional probability of SFP failure is about a factor of 20 less than the annual probability of exceeding the SFP HCLPF value. Given that the SFP HCLPF value is more than or equal to three times the SSE (and less than 10^{-5}) then the SFP failure frequency should be less than 5×10^{-7}. This simplified method is based on use of peak ground acceleration (PGA) curves.

The second simplified method was suggested by Kennedy and is based on use of spectral acceleration (S_a) rather than PGA. Kennedy states that damage to structures, systems, and components (SSCs) does not correlate well to PGA ground motions but correlates much better with spectral accelerations between 2.5 and 10 Hz at nuclear power plants. Based on previous studies Kennedy proposes to screen SFPs based on use of the peak spectral acceleration (PSA) HCLPF seismic capacity of 1.2g. This value is equivalent to 0.5g PGA. This simplified approach is based on calculating the 10% conditional probability of failure capacity ($C_{10\%}$) given the PSA value of 1.2g. Using Equation 6 in the Reference 1 attachment results in a $C_{10\%}$ S_a value of 1.82g. The annual probability of exceeding this value at 10, 5 and 2.5 Hz is then calculated using the LLNL hazard results. These value are then multiplied by 0.5 and the highest of the 10, 5, and 2.5 Hz results is used as the SFP failure probability. For example, the $C_{10\%}$ at 5 Hz is 1.82g or about 56.8 cm/sec spectral velocity. For LLNL site 1, the annual probability of exceeding 56.8 cm/sec is about 2.0×10^{-6}. This value is multiplied by 0.5 which results in a SFP failure probability for site 1 of about 1.0×10^{-6}. This same calculation is performed at 10 and 2.5 Hz.

Based on comparisons made by Kennedy he concludes that simplified method 1 (Reference 3) underestimates the seismic risk by factors of 2.3 and 3.5 for Vermont Yankee and Robinson respectively. Using simplified method 2 the seismic risk is overestimated by 20% and 5% respectively for these two cases.

Kennedy noted that in his judgement it will be necessary to have seismic fragility HCLPF computations performed on at least six different aboveground SFPs with walls not supported by soil before HCLPF screening levels can be established for these SFPs.

Recommendation Number 4 of the December 3, 1999 memorandum requested that industry provide input concerning:

f. the list of high hazard sites,
g. a credible ground motion description at which the seismic hazard frequency is low enough at these sites, and
h. plant specific seismic capacity evaluations using credible ground motion descriptions at these sites.

Recommendation Number 5 requests that industry propose treatment of sites West of the Rocky Mountains.

Preliminary Industry Comments

Industry concurs that use of a seismic screening checklist is an excellent approach to plant-specific seismic assessments. In addition, we will incorporate into our earlier seismic checklist those suggestions presented in Recommendation numbers 1, 2, and 3 to the December 3, 1999 memorandum.

With respect to the simplified methods to estimate seismic failure frequency of SFP failure the method proposed by Kennedy appears to be reasonable.

In the recommendations section of the 12/3/99 memorandum (Reference 1) some actions by industry are proposed. Recommendation Number 4.b requests that industry recommend a credible ground motion description at which the seismic hazard frequency is low enough at these " high" hazard sites. These "high" hazard sites were identified based on use of the Kennedy simplified SFP failure methodology and the LLNL 1993 hazard results. The response to Recommendation Numbers 4.a and 4.c are dependent on the resolution of 4.b.

Comments on Recommendation Number 4.b

1. Using the Kennedy simplified SFP failure methodology $C_{10\%}$ values are determined at 10, 5, and 2.5 Hz. At 5 Hz the spectral acceleration value is 1.82g or about 56.8 cm/sec.

2. The PSA values associated with these $C_{10\%}$ values are consistent with spectral values which describe the San Onofre and Diablo Canyon SSEs, i.e., large magnitude, near field earthquakes.

3. The issue of large earthquakes occurring near EUS NPPs was resolved by the Charleston Issue (SECY-91-135, Reference 4). As stated in SECY-91-135, "Large 1886 Charleston-size earthquakes, greater than or equal to magnitude 6.5, are not significant contributors to the seismic hazard for nuclear facilities along the eastern seaboard outside the Charleston region. This result is consistent with the results emerging from the ongoing studies of earthquake-induced liquefaction features along the eastern seaboard. These studies have found no evidence of large prehistoric earthquakes originating outside the South Carolina region. Thus the issue of the Charleston earthquake occurring elsewhere in the eastern seaboard is considered to be closed."

4. Credible, versus not credible in terms of annual probability, is typically associated with greater than about 10^{-6} (credible) and 10^{-6} or less (not credible). Within the context of the Kennedy simplified SFP failure methodology, if the annual probability of exceeding the screening level value (for example 56.8 cm/sec at 5 Hz) times 0.5 is less than 10^{-6}, then only the seismic checklist must be satisfied. Implicit in this approach is that the probabilistic estimates at the $C_{10\%}$ level are credible.

5. For a site to be screened out the $C_{10\%}$ value should be on the order of 10^{-6}. Figure 1

(attached) shows the 5 Hz spectral acceleration values associated with the 10^{-6} LLNL results at each of the 69 sites. As can be seen, for site number 36 (which in Table 3 of the Kennedy report is the site with the highest SFP failure frequency) the 10^{-6} spectral acceleration is about 7,700 cm/sec^2 or about 245 cm/sec. As stated previously, 57 cm/sec is consistent with 5 Hz spectral velocities associated with a magnitude 6.6 earthquake 8 km from the site (San Onofre SSE), therefore these predicted groundmotions must be associated with a very large earthquake, greater than magnitude 6.5, very near to the site – which is counter to the conclusions of SECY-91-135. Other values at other sites are equally incredible. Based on these results, it is concluded that the LLNL results, at the probability/ground motion levels of interest, are deterministically incredible and therefore their use in screening is questionable. Figure 2 (attached) shows the 5 Hz spectral acceleration values associated with the 10^{-6} EPRI results. As can be seen, the EPRI results, at the probability/ground motion levels of interest, are credible, and consistent with SECY-91-135.

6. Figure 3 (Figure 2 from NUREG-1488, Reference 5) illustrates the problems associated with the LLNL results at high ground motions/low annual probabilities. As can be seen from Figure 3, at high probabilities there is reasonable agreement between LLNL and EPRI. However, the slope of the LLNL results at high ground motions is too shallow. The effect of this shallow slope is to predict incredible ground motions at credible probability levels.

7. Based on this review, industry contends that it would be appropriate to only use EPRI results in the SFP seismic screening analysis. We believe this to be reasonable in light of the difficulties associated with the LLNL results at low probabilities. The effect of using only the EPRI results is shown in column 3 of Table 3 in the Kennedy report (Reference 1). As can be seen, only 1 plant would be required to perform further analyses. However, because both LLNL and EPRI are considered to provide valid results, it is proposed that the results from each study be geometrically averaged such that equal weight is provided the results from each study. Arithmetic averaging is considered unacceptable in light of the difficulties associated with the LLNL results. Figure 4 provides the results of geometrically averaging the LLNL and EPRI results.

Comments on Recommendation Number 4.a

Based on Figure 4 about 6 sites would be preliminarily screened in due to exceeding the 10^{-6} criterion. One of the 6 sites is Shoreham. If these screened in SFPs are above ground then further analyses will be required.

Comments on Recommendation Number 4.c

It is industry's understanding of Section 4.2 of the Kennedy report that given that a plant satisfies the seismic screening checklist then the SFP is likely to have a seismic capacity higher than the screening level capacity. If plant-specific information is conveniently available, additional seismic capacity values will be developed in a manner similar to that described in

NUREG/CR-5176.

Comments on Recommendation Number 5

A response to the NRC Recommendation Number 5 requesting industry to provide "Proposed treatment of sites West of the Rocky Mountains" will be provided later. However, as a result of detailed deterministic investigations at and around each site, a better understanding of the sources and causes of earthquakes is developed in the licensing of Western U.S. (WUS) plants. Therefore, it would be reasonable to describe the credible ground motion for WUS sites deterministically.

References:

1. Memorandum, W. Hauffman to S. A. Richards, USNRC, Screening Criteria for Assessing Potential Seismic Vulnerabilities of Spent Fuel Pools at Decommissioning Plants, December 3, 1999.

2. NUREG/CR-5176, Seismic Failure and Cask Drop Analyses of the Spent Fuel Pools at Two Representative Nuclear Power Plants, Lawrence Livermore National Laboratory, January 1989.

3. USNRC, Preliminary Draft Technical Study of Spent Fuel Pool Accidents for Decommissioning Plants, June 16, 1999.

4. SECY-91-135, Conclusions of the Probabilistic Seismic Hazard Studies Conducted for Nuclear Power Plants in the Eastern United States, May 14, 1991.

5. NUREG-1488, Revised Livermore Seismic Hazard Estimates for 69 Nuclear Power Plant Sites East of the Rocky Mountains, October, 1993.

5f December 28, 1999 Kennedy Letter

Structural Mechanics Consulting, Inc.

Robert P. Kennedy 18971 Villa Terrace, Yorba Linda, CA 92686 (714) 777-2163

December 28, 1999

Dr. Charles Hofmayer
Environmental & Systems Engineering Division
Brookhaven National Lab
Building 130, 32 Lewis Road Upton, NY 11973-5000

Subject: Additional Documents Concerning Seismic Screening and Seismic Risk of Spent Fuel Pools For Decommissioning Plants

Dear Dr. Hofmayer:

I have reviewed the December 3,1999 memorandum from W. Huffman to S. Richards entitled *Screening Criteria for Assessing Potential Seismic Vulnerabilities of Spent Fuel Pools at.Decommissioning Plants*. I have also reviewed the "Industry Comments" on the material presented in this memorandum. Lastly, I reviewed Revision I of the *Industiy Seismic Screening Criteria* dated December 13, 1999.

I concur with the adequacy of the *Industry Seismic Screening Criteria* presented in Revision I for the vast majority of Central and Eastern US (CEUS) sites. So long as Screening Items I through 9 are satisfied, the seismic risk of spent fuel pool failure to contain water for these sites should be so low as to not warrant further assessment. The addition of Screening Item 4 in Revision I removes my concern about the previous draft. For spent fuel pool walls and floor slab not supported by soil, Screening Item 4 requires a structural assessment of the pool walls and floor slab out-of-plane shear and flexural capabilities be performed and compared to the realistic demands expected to be generated by seismic input equal to approximately three times the site SSE input. In order to demonstrate a HCLPF capacity in excess of approximately 3 SSE, this assessment should be performed with the degree of conservatism defined for the Conservative Deterministic Failure Margin (CDFM) method in EPRI 6041.

Spent fuel pools at a few higher seismic hazard sites in the CEUS and all Western US sites should be further evaluated beyond this screening criteria. I concur with the approach presented on page 4 of the "Industry Comments" for defining these few higher seismic hazard CEUS sites. Based on Figure 4 of the "Industry Comments", it appears that no more than 4 CEUS sites (excluding Shoreham) would fall into this higher seismic hazard category.

Either Seismic Margin or Seismic Fragility HCLPF capacity estimates should be made for spent fuel pools at decommissioning plants in each of the following cases:

1. Out-of-plane flexural and shear capacity of aboveground spent fuel pool walls and floors not supported by soil.

2. Spent fuel pools which do not pass the Revision I *Industry Seismic Screening Criteria.*

3. A few higher seismic hazard CEUS sites and all Western sites.

For the above situations where HCLPF capacity assessments should be made, I understand that Goutam Bagehi and Bob Rothman of the NRC have recommended that a plant coming in for decommissioning which can show that their spent fuel pool structural resistance has a HCLPF value of 3*SSE for CEUS sites and 2*SSE for West Coast sites has demonstrated an adequately low seismic risk for their spent fuel pool. This recommended approach represents a reasonable engineering approach with which I concur.

I believe the approach outlined above is a practical approach for demonstrating the seismic risk of spent fuel pools at decommissioning plants is very low. Please contact me if you desire further discussion.

Sincerely

Robert Kennedy

cc. Mr. Goutam Bagchi
Dr. Nilesh Chokshi

5g Enhanced Seismic Checklist

Item 1:

Requirement: Identify Preexisting Concrete and Liner Plate Degradation

Basis: A detailed review of plant records concerning spent fuel pool concrete and liner plate degradation should be performed and supplemented by a detailed walkdown of the accessible portions of the spent fuel pool concrete and liner plate. The purpose of the records review and visual inspection activities is to accurately assess the material condition of the SFP concrete and liner in order to assure that these existing material conditions are properly factored into the remaining seismic screening assessments.

Design Feature: The material condition of the SFP concrete and liner, based upon the records review and the walkdown inspection, will be documented and used as an engineering input to the following seismic screening assessments.

Item 2:

Requirement: Assure Adequate Ductility of Shear Wall Structures

Basis: The expert panel involved with the development of Reference 1 concluded that, " For the Category 1 structures which comply with the requirements of either ACI 318-71 or ACI 349-76 or later building codes and are designed for an SSE of at least 0.1g pga, as long as they do not have any special problems as discussed below, the HCLPF capacity is at least 0.5g pga." This conclusion was based upon the assumption that the shear wall structure will respond in a ductile manner. The "special problems" cited deal with individual plant details which could prevent a particular plant from responding in the required ductile fashion. Examples cited in Reference 1 included an embedded structural steel frame in a common shear wall at the Zion plant (which was assumed to fail in brittle manner due to a potential shear failure of the attached shear studs) and large openings in a "crib house" roof (also at the Zion plant) which could interrupt the continuity of the structural slab.

Other examples which could impact the ductility of the spent fuel pool structure include large openings which are not adequately reinforced or reinforcing bars that are not sufficiently embedded to prevent a bond failure before the yield capacity of the steel is reached.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 3:

Requirement: Assure Design adequacy of Diaphragms (including roofs)

Basis: In the design of many nuclear power plants, the seismic design of roof and floor diaphragms has often not received the same level of attention as have the shear walls of the structures. Major cutouts for hatches or for pipe and electrical chases may pose special problems for diaphragms. Since more equipment tends to be anchored to the diaphragm compared to shear walls, moderate amounts of damage may be more critical for the diaphragm compared to the same amount of damage in a wall.

Based upon the guidance provided in Reference 1, diaphragms for Category I structures designed for a SSE of 0.1g or greater do not require an explicit evaluation provided that: (1) the diaphragm loads were developed using dynamic analysis methods; (2) they comply with the ductility detailing requirements of ACI 318-71 or ACI 349-76 or later editions. Diaphragms which do not comply with the above ductility detailing or which did not have loads explicitly calculated using dynamic analysis should be evaluated for a beyond-design-basis seismic event in the 0.45-0.5g pga range.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 4:

Requirement: Verify the Adequacy of the SFP Walls and Floor Slab to Resist Out-of-Plane Shear and Flexural Loads

Basis: For PWR pools that are fully or partially embedded, an earthquake motion that could cause a catastrophic out-of-plane shear or flexural failure is very high and is not a credible event. For BWR pools (and PWR pools that are not at least partially embedded), the seismic capacity is likely to be somewhat less and the potential for our-of-plane shear and/or flexural wall or base slab failure, at beyond-design-basis seismic loadings, is possible.

A structural assessment of the pool walls and floor slab out-of plane shear and flexural capabilities should be performed and compared to the realistic loads expected to be generated by a seismic event equal to approximately three times the site SSE. This assessment should include dead loads resulting from the masses of the pool water and racks, seismic inertial forces, sloshing effects and any significant impact forces.

Credit for out-of-plane shear or flexural ductility should not be taken unless the reinforcement associated with each failure mode can be shown to meet the ACI 318-71 or ACI 349-49 requirements.

Design Feature: Compliance with this design feature will be documented basedupon a review of drawings (in the case of embedded or partially embedded PWR pools) or based upon a review of drawings coupled with the specified beyond-design-basis shear and flexural calculations outlined above.

Item 5:

Requirement: Verify the Adequacy of Structural Steel (and Concrete) Frame Construction

Basis: At a number of older nuclear power plants, the walls and roof above the top of the spent fuel pool are constructed of structural steel. These steel frames were generally designed to resist hurricane and tornado wind loads which exceeded the anticipated design basis seismic loads. A review of these steel (or possibly concrete) framed structures should be performed to assure that they can resist the seismic forces resulting from a beyond-design-basis seismic event in the 0.45-0.5g pga range. Such a review of steel structures should concentrate on structural detailing at connections. Similarly, concrete frame reviews should concentrate on the adequacy of the reinforcement detailing and embedment.

Failure of the structural steel superstructure should be evaluated for its potential impact on the ability of the spent fuel pool to continue to successfully maintain its water inventory for cooling and shielding of the spent fuel.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 6:

Requirement: Verify the Adequacy of Spent Fuel Pool Penetrations

Basis: The seismic and structural adequacy of any spent fuel pool (SFP) penetrations whose failure could result in the draining or syphoning of the SFP must be evaluated for the forces and displacements resulting from a beyond-design-basis seismic event in the 0.45-0.5g pga range. Specific examples include SFP gates and gate seals and low elevation SFP penetrations, such as, the fuel transfer chute/tube and possibly piping associated with the SFP cooling system. Failures of any penetrations which could lead to draining or syphoning of the SFP should be considered.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

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Requirement: Evaluate the Potential for Impacts with Adjacent Structures

Basis: Structure-to-structure impact may become important for earthquakes significantly above the SSE, particularly for soil sites. Structures are usually conservatively designed with rattle space sufficient to preclude impact at the SSE level but there are no set standards for margins above the SSE. In most cases, impact is not a serious problem but, given the potential for impact, the consequences should be addressed. For impacts at earthquake levels below 0.5g pga, the most probable damage includes the potential for electrical equipment malfunction and for local structural damage. As cited previously, these levels of damage may be found to be acceptable or to result in the loss of SFP support equipment. The major focus of this impact review is to assure that the structure-to-structure impact does not result in the inability of the SFP to maintain its water inventory.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

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Requirement: Evaluate the Potential for Dropped Loads

Basis: A beyond-design-basis seismic event in the 0.45-0.5g pga range has the potential to cause the structural collapse of masonry walls and/or equipment supports systems. If these secondary structural failures could result in the accidental dropping of heavy loads which are always present (i.e. not loads associated with cask movements) into the SFP, then the consequences of these drops must be considered. As in previous evaluations, the focus of the drop consequence analyses should consider the possibility of draining the SFP. Additionally, the evaluation should evaluate the consequences of any resulting damage to the spent fuel or to the spent fuel storage racks.

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 9:

Requirement: Evaluation of Other Failure Modes

Basis: Experienced seismic engineers should review the geotechnical and structural design details for the specific site and assure that there are not any design vulnerabilities which will not be adequately addressed by the review areas listed above. Soil-related failure modes including liquefaction and slope instability should be screened by the approaches outlined in Reference 1 (Section 7 & Appendix C).

Design Feature: This design feature requirement will be documented based on a review of drawings and a SFP walkdown.

Item 10: Potential Mitigation Measures

Although beyond the scope of this seismic screening checklist, the following potential mitigation measures may be considered in the event that the requirements of the seismic screening checklist are not met at a particular plant.

a.) Delay requesting the licensing waivers (E-Plan, insurance, etc.) until the plant specific danger of a zirconium fire is no longer a credible concern.

b.) Design and install structural plant modifications to correct/address the identified areas of non-compliance with the checklist. (It must be acknowledged that this option may not be practical for significant seismic failure concerns.)

c.) Perform plant-specific seismic hazard analyses to demonstrate that the seismic risk associated with a catastrophic failure of the pool is at an acceptable level. (The exact "acceptable" risk level has not been precisely quantified but is believed to be in the range of 1.0E-06.)

We believe that use of the checklist and determination that the spent fuel pool HCLPF is sufficiently high will assure that the frequency of fuel uncovery from seismic events is less than or equal to 1×10^{-6} per year.

5h Other Seismic Stakeholder Interactions

1. A member of the public raised a concern about the potential effects of Kobe and Northridge earthquakes related to risk-informed considerations for decommissioning during the Reactor Decommissioning Public Meeting on Tuesday, April 13, 1999, in Rockville, MD.

Stakeholder Comment

"I guess I'd like to direct my questions to the seismological review for this risk-informed process. And first of all, did any of the NUREGs that you looked at take into account new information coming out of the Kobe and Northridge events? I think that what we need to be concerned with is dated information. Particularly as we are learning more about risks associated with those two particular seismological events that were never even considered when plants were sited; particularly, though I can't frame it in the seismological language, from a lay understanding, it's clear that new information was gained out of Kobe and Northridge events suggesting that you can have seismological effects of greater consequence farther afield than at the epicenter of the event."

Response

The two NUREGs mentioned by a member of the public were written in the middle and late 1980s and used probabilistic seismic hazard analyses performed for the NRC by Lawrence Livermore National Laboratory (LLNL) for nuclear power plants in the central and eastern U.S. Since then, LLNL has performed additional probabilistic hazard studies for central and eastern U.S. nuclear power plants for the NRC. The results of these newer studies indicated lower seismic hazards for the plants than the earlier studies estimated. If the probabilistic hazard studies were to be performed again, hazard estimates for most sites would probably be reduced further than the LLNL 1993 study due to: new methods of eliciting information, newer methods of sampling hazard parameters' uncertainties, better information on ground motion attenuation in the U.S. and a more certain understanding of the seismicity of the central and eastern U.S.

The design basis for each nuclear power plant took into account the effects of earthquake ground motion. The seismic design basis, called the safe shutdown earthquake (SSE), defines the maximum ground motion for which certain structures, systems, and components necessary for safe shutdown were designed to remain functional. The licensees were required to obtain the geologic and seismic information necessary to determine site suitability and provide reasonable assurance that a nuclear power plant could be constructed and operated at a site without undue risk to the health and safety of the public.

The information collected in the investigations was used to determine the earthquake ground motion at the site, assuming that the epicenters of the earthquakes are situated at the point on the tectonic structures or in the tectonic provinces nearest to the site. The earthquake which could cause the maximum vibratory ground motion at the site was designated the safe shutdown earthquake (SSE). This ground motion was used in the design and analysis of the plant.

The determination of the SSEs had to follow the criteria and procedures required by NRC regulations and apply a multiple hypothesis approach. In this approach, several different methods were applied to determine each parameter, and sensitivity studies were performed to account for the uncertainties in the geophysical phenomena. In addition, nuclear power plants have design margins (capability) well beyond the demands of the SSE. The ability of a nuclear power plant to resist the forces generated by the ground motion during an earthquake is thoroughly incorporated in the design and construction. As a result, nuclear power plants are able to resist earthquake ground motions well beyond their design basis and far above the ground motion that would result in severe damage to residential and commercial buildings designed and built to standard building codes.

Following large damaging earthquakes such as the Kobe and Northridge events, the staff reviewed the seismological and engineering information obtained from these events to determine if the new information challenged previous design and licensing decisions. The Kobe and Northridge earthquakes were tectonic plate boundary events occurring in regions of very active tectonics. The operating U.S. nuclear power plants (except for San Onofre and Diablo Canyon) are located in the stable interior portion of the North American tectonic plate. This is a region of relatively low seismicity and seismic hazard. Earthquakes with the characteristics of the Kobe and Northridge events will not occur near central and eastern U.S. nuclear power plant sites.

The ground motion from an earthquake at a particular site is a function of the earthquake source characteristics, the magnitude and the focal mechanism. It is also a function of the distance of the facility to the fault, the geology along the travel path of the seismic waves, and the geology immediately under the facility site. Two U.S. operating nuclear power plant sites can be considered as having the potential to be subjected to the near field ground motion of moderate to large earthquakes. These are the San Onofre Nuclear Generating Station (SONGS) near San Clemente and the Diablo Canyon Power Plant (DCPP) near San Luis Obispo. The seismic design of SONGS Units 2 and 3 is based on the assumed occurrence of a magnitude 7 earthquake on the Offshore Zone of Deformation, a fault zone approximately 8 kilometers from the site. The design of DCPP has been analyzed for the postulated occurrence of a magnitude 7.5 earthquake on the Hosgri Fault Zone, approximately 4 kilometers from the site. The response spectra, used for both the SONGS and the DCPP, was evaluated against the actual spectra of near field ground motions of a suite of earthquakes gathered on a worldwide basis.

The individual stated, "... it's clear that new information was gained out of Kobe and Northridge events suggesting that you can have seismological effects of greater consequence farther afield than at the epicenter of the event." A review of the strong motion data and the damage resulting from these events do not bear out the validity of this concern at SONGS and DCPP.

The staff assumes that the individual alluded to the fact that the amplitudes of the ground motion from the 1994 Northridge earthquake were larger in Santa Monica than those at similar and lesser distances from the earthquake source. The cause of the larger ground motions in the Santa Monica area is believed to be the subsurface geology along the travel path of the waves. One theory (Gao et al, 1996) is that the anomalous ground motion in Santa Monica is explained by focusing due to a deep convex structure (several kilometers beneath the surface) that focuses the ground motion in mid-Santa Monica. Another theory (Graves and Pitarka, 1998) is that the large amplitudes of the ground motions in Santa Monica from the Northridge earthquake are caused by the shallow basin-edge structure (1 kilometer deep) at the northern edge of the Los Angles Basin. This theory suggests that the large amplification results from constructive interference of direct waves with the basin-edge generated surface waves. Earthquake recordings at San Onofre and Diablo Canyon do not indicate anomalous amplification of ground motion. In addition, there have been numerous seismic reflection and refraction studies of the site areas for the site evaluations, and for petroleum exploration and geophysical research. They, along with other well-proven methods, were used to determine the nature of the geologic structure in the site vicinity, the location of any faults, and the nature of the faults. None of these studies have indicated anomalous conditions, like those postulated for Santa Monica, at either SONGS or DCPP. In addition, the empirical ground motion database used to develop the ground motion attenuation relationships contains events recorded at sites with anomalous, as well as typical ground motion amplitudes. The design basis ground motion for both SONGS and DCPP were compared to 84th percentile level of ground motion obtained using the attenuation relationships and the appropriate earthquake magnitude, distance and geology for each site. The geology of the SONGS and DCPP sites do not cause anomalous amplification, therefore, there is no "new information gained from the Kobe and Northridge events," which raises safety concerns for U.S. nuclear power plants.

In summary, earthquakes of the type that occurred in Kobe and Northridge are different from those that can occur near nuclear power plants in the central and eastern U.S. The higher ground motions recorded in the Santa Monica area from the Northridge earthquake were due to the specific geology through which the waves traveled. Improvements in our understanding of central and eastern U.S. geology, seismic wave attenuation, seismicity, and seismic hazard calculation methodology result in less uncertainty and lower hazard estimates today than have previous studies.

2. During the July workshop, members of the public raised concerns about the hazard of the fuel transfer tube interacting with the pool structure during a large earthquake. There was also another concern about the effect of aging on the spent fuel pool liner plate and the reinforced concrete pool structure.

   Transfer tubes are generally used in PWR plants where the fuel assembly exits the containment structure through the tube and enters the pool. These transfer tubes are generally located inside a concrete structure that is buried under the ground and attached to the pool structure through a seismic gap and seal arrangement. These layouts and arrangements can vary from one PWR plant to another, and the seismic hazard caused by transfer tubes should be examined on a case-by-case basis. This is included in the seismic checklist.

3. During the July workshop, members of the public raised concerns about the effect of aging on the spent fuel pool liner plate and the reinforced concrete pool structure.

   Irradiation-induced degradation of steel requires high neutron fluency, which is not present in the spent fuel pools. Operating experience has not indicated any degradation of liner plates or the concrete that can be attributed to radiation effects.

   With aging, concrete gains compressive strength of about 20% in an asymptotic manner and spent fuel pool structures are expected to have this increased strength at the time of their decommissioning. Degradation of concrete structures can be divided into two parts, long term and short term. The long-term degradation can occur due to freezing and thawing effects when concrete is exposed to outside air. This is the predominant long-term failure mode of concrete; observed on bridge decks, pavements, and structures exposed to weather. Degradation of concrete can also occur when chemical contaminants attack concrete. These types of degradation have not been observed in spent fuel pools in any of the operating reactors. Additionally, inspection and maintenance of spent fuel pool structures are within the scope of the maintenance rule, 10 CFR 50.65, and corrective actions are required if any degradation is observed. An inspection of the spent fuel pool structure to identify cracks, spalling of concrete, etc., is also recommended as a part of the seismic checklist. Significant degradation of reinforced concrete structures would take more than 5 years or so, the time necessary to lose decay heat in the spent fuel. Substantial loss of structural strength requires long-term corrosion of reinforcing steel bars and substantial cracking of concrete. This is not likely to happen because of inspection and maintenance requirements.

The short-term period of concern for the beyond-design-basis seismic event can be considered to last no more than several days. Any seepage of water during this time will not degrade the capacity of concrete. Degradation of concrete strength would require loss of cross-section of reinforcing bars due to corrosion, and a period of several days is too short to cause such a loss.

Degradation of the liner plate can occur due to cracks that can develop at the welded joints. Seepage of water through minute cracks at welded seams has been minimal and has not been observed at existing plants to cause structural degradation of concrete. Nevertheless, preexisting cracks would require a surveillance program to ensure that structural degradation is not progressing.

Based on the discussion above, it can be assumed that the spent fuel pool structure will be at its full strength at the initiation of a postulated beyond-design-basis event.