Appendix 2 Structural Integrity of Pool Structure

Introduction

As a part of the Generic Issue 82, "Beyond Design Basis Accidents in Spent Fuel Pools," the NRC has studied the hypothetical event of an instantaneous loss of spent fuel pool water. A study in support of this generic issue indicates that a key part of a plant-specific evaluation of the effect of such an event a realistic seismic fragility analysis of the spent fuel pool. The failure or the end state of concern is a catastrophic failure of the spent fuel pool leading to an almost instantaneous loss of all pool water and making the pool unable to retain any water if it were to be reflooded.

Spent fuel pool structures at nuclear power plants are constructed with thick reinforced concrete walls and slabs lined with thin stainless steel liners 1/8 to 1/4 inches thick, except at Dresden Unit 1 and Indian Point Unit 1. These two plants do not have liner plates. They were decommissioned more than 20 years ago and no safety-significant degradation of the concrete pool structure has been reported. The walls vary from 4.5 to 5 feet in thickness and the pool floor slabs are around 4 feet thick. The overall pool dimensions are typically about 50 feet long by 40 feet wide and 55 to 60 feet high. In boiling water reactor (BWR) plants, the pool structures are in the reactor building at an elevation several stories above the ground. In pressurized water reactor (PWR) plants, the spent fuel pool structures are outside the containment structure and supported by the ground or partially embedded in the ground. The location and supporting arrangement of the pool structures determine their capacity to withstand loads beyond their design basis. The dimensions of the pool structure are generally derived from radiation shielding considerations rather than structural needs. Spent fuel structures at operating nuclear power plants are inherently rugged, able to withstand loads substantially beyond those for which they were designed. Consequently, they have significant seismic capacity. Because of the ruggedness of the spent fuel pools, licensees have proposed that the continued implementation of the emergency plan at a decommissioned plant is burdensome and unnecessary. Also of concern to the licensees are insurance indemnity and safeguards.

The focus of the current effort is to examine the effect of a large seismic event at a plant immediately following decommissioning and to prepare input for the draft report by the Technical Working group. Several public meetings were held in (April, May, June, and July 1999), and there was a 2-day public workshop to discuss the June 1999 draft report. At the public workshop, we proposed, and the industry group agreed, to develop a seismic check-list to be used to examine the seismic vulnerability of any given plant. This draft report examines the effort so far on the development of the seismic checklist.

Available NRC Studies

Two reports are relevant to this issue:

(1) NUREG/CR-4982, Severe Accidents in Spent Fuel Pools in Support of Generic Safety Issue 82, July 1987.
(2) NUREG/CR-5176, Seismic Failure and Cask Drop Analyses of the Spent Fuel Pools at Two Representative Plants, January 1989.

L/130

After completing work for the above studies, NRC performed a study to review of seismic hazard at central and eastern US plants and issued NUREG-1488, Revised Livermore Seismic Hazard Estimates for 69 Nuclear Power Plant Sites East of the Rocky Mountains (October 1993). It is well recognized that the Lawrence Livermore National Laboratory (LLNL) seismic hazard curves used before the publication of NUREG-1488 were overly conservative. In the NUREG/CR-5176 study of the Vermont Yankee plant, the high confidence of low probability of failure (HCLPF) level for the spent fuel pool is 0.5 g. At the H.B. Robinson site the HCLPF value is 0.65 g. The 1989 and the 1993 LLNL hazard curves show that the probabilities of exceeding these values are factors of 2 and 1.6, respectively, higher if the older and more conservative 1989 study is used.

Structural Behavior

Seismic vulnerability of spent fuel pool structures is expected at levels of earthquake ground motion equal to 2.5 to 3.5 times the plant's safe shutdown earthquake (SSE). These are such large earthquake motions that design basis seismic analyses are not likely to be representative of the behavior of the pool structure under failure level earthquakes. There is considerable difficulty in judging the adequacy of simple analytical models. These large earthquake motions would induce large strain in the foundation medium, the soil structure interaction effect would be modified and if there was not much rocking motion under the SSE, increased rocking motion can be expected for large earthquakes. Impact with adjacent buildings cannot be ruled out for the large seismic event and damage to the pool structure due to the failure of the overhead crane equipment or the failure of the superstructure would have to be taken into account. Uplift of the pool foundation mat and impact on the subgrade would seek out weak links in the pool structure and could lead to local spalling of concrete. Amplification of ground motion up through the reactor building could be substantially higher than the SSE response for BWR pool structures. The design, layout and construction of the pool structures are very important to consider in a seismic vulnerability assessment.

The seismic hazards at the west coast sites are generally governed by known active tectonic sources; consequently, the hazard curves have a much steeper slope near the higher ground motion level. Another way to say this is, as the magnitude of the seismic event increases, the probability of its occurrence goes down rapidly. Thus a seismic event equal to 2.5 to 3.5 times the SSE at a west coast site may be considered incredible for the site. Therefore, for west coast sites a seismic event greater than 2 times the SSE could be considered to be too large to be incredible. Spent fuel pool structures at these sites would then need to have capacity against catastrophic failure at 2 times the SSE.

Structural Failure Modes

Among the various ways a pool structure can fail, the only failure modes that are of concern are those that involve failure of the pool floor slab, the side walls at the bottom of the pool or 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 and this takes 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, so that the resistance in the horizontal and vertical directions are unequal. The resistance differs between one wall and another wall. This requires a case-by-case assessment of the pool wall capacity using plant-specific information.

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.

We emphasize 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 the seismic capacity of pool structures typically consist of the following steps:

- Inspect the pool structure and its vicinity and note:
  - the physical condition (cracking and spalling of concrete, signs of leakage or leaching and separation of pool walls from the grade surface, and potential for piping connections, either buried under ground 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)
  - the 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
  - the adjacent structures that can impact the pool structure both above and below the grade surface, the supporting arrangement for superstructure and crane and potential for failure of the superstructure and the crane, and the potential impact from heavy objects that can drop in the pool structure and the corresponding drop heights

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

- Review existing layout drawings and structural dimensions, 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 that the pool structure could survive, determine whether design dynamic response analysis, including soil-structure interaction effects, are still applicable at the capacity-level seismic event, and 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 the impact of the 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 during the collapse of a superstructure or the overhead crane.

- Determine a list of plausible failure modes: e.g., 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, and local failure by punching shear due to impact between structures and the spent fuel racks or to 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 a simple examination using a seismic check-list. Such an effort may be necessary for plants in higher seismic hazard areas.

Public Meeting of April 13, 1999

Presentations made by NEI relied on the NRC-sponsored studies and concluded that structural failure of the spent fuel pool is unlikely, based on probability of the initiating events, and should be eliminated from further consideration in the risk-informed decommissioning rulemaking. NEI arguments are risk based and do not take into account uncertainties associated with the seismic risk which ranges from 2.4×10^{-4} to 3.1×10^{-11} per reactor-year. For this reason, it is important to conduct a seismic vulnerability assessment on a case-by-case basis and either establish a risk-informed performance goal or develop a simple method of eliminating plants on the basis of their seismic robustness.

Other Considerations

NRC-sponsored studies have assessed the seismic capacity of spent fuel pools by relying on the seismic margins method to determine the high confidence of low probability (less than 5%) of failure (HCLPF). The HCLPF value for a structural failure may well be unrealistic and unnecessarily conservative for an instantaneous loss of water inventory. This point needs to be emphasized because the shear and moment capacity of the walls and slabs is determined by using upper limits of allowable stresses. In the study documented 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. The relatively low margin for allowable out-of-plane shear strength, combined with the uncertainty of the extent to which reinforcement details ensure ductile behaviors, makes 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 of the pool slab or the wall near the bottom of the pool.

For PWR pools that are fully or partially embedded, the probability of an earthquake motion that could cause a catastrophic failure, is very high. 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 most BWR pools are at a higher elevation and the seismic motion is amplified since 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 new LLNL data from NUREG-1488 we examined 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. We also reviewed 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, our evaluation is driven by the 1993 LLNL seismic hazard results, and we determined that, except for a few sites, the POEs for SSE are lower than $1X10^{-4}$ per reactor year, and for 3 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 3 times the SSE should be less than $5X10^{-7}$. This approach makes the simplifying assumption that the conditional probability of failure (POF) or reaching the end state of a structure is $5X10^{-2}$ and gives confidence that the seismic hazard is low (at 3 times the SSE) and that a plant-specific structural assessment of the HCLPF value will yield a value which is more than or equal to 3 times the SSE.

Performance Goal for Spent Fuel Pool Structures

The seismic risk of a decommissioned plant is based on the frequency of the initiating event, the earthquake, and the conditional catastrophic failure of the pool structure. Based on numerous past studies by the industry and the NRC, site-specific seismic hazard estimates are better understood. The design basis earthquake ground motion, or the SSE ground motion, for nuclear power plant sites was based on the largest event geophysically ascribable to a tectonic province or at a capable structure at the closest proximity of the site. In the case of a tectonic province, the event is assumed to occur at the site. For the eastern seaboard, the Charleston event is the largest magnitude earthquake and current research has established that such large events are confined to Charleston region. The New Madrid zone is another zone in the central U.S. where very large events have occurred. However, both these tectonic sources are fully accounted for in the assessment of the SSE for currently licensed plants. The SSE ground motions for nuclear power plants are based on conservative estimates of the ground motion from the largest earthquake estimate to be generated under the current tectonic regime. If we amplify these SSE ground motions by three, we are at or the beyond the limit of credibility. This is not a probabilistic statement, but a statement based on geophysical reality.

Therefore, it appears reasonable to assume that a seismic event greater than 3 times the SSE at a lower seismicity location (eastern U.S. coast site) and 2 times the SSE at a higher seismicity location (west coast site) is incredible. The seismic hazard component of the risk statement can, thus, be set aside if it can be demonstrated that structural capacity, the HCLPF value, is greater than or equal to 2 times the SSE at higher seismicity sites and at 3 times the SSE at lower seismicity sites. Implicit in this proposed performance goal is the assumption that pool structures are free from pre-existing degradation or other seismic vulnerabilities that can be identified by the use of a seismic check list. It is noted that the configuration, layout, and structural details vary considerably from one plant to another. Therefore, the performance requirement should be such that a spent fuel pool structure have a HCLPF value equal to or greater than 2 and 3 times the SSE ground motion for higher or lower seismicity sites, respectively. This proposed performance goal simplifies the task of demonstrating that the seismic risk from the spent fuel pool is negligible. Those plants that can demonstrate that they meet the proposed performance goal could be eliminated from any further seismic evaluation. For sites that fail the seismic check list screening of the pool structure, or have a HCLPF value lower than the performance goal, it would be necessary to conduct a detailed assessment of the seismically induced probability of failure of spent fuel pool structures.

Additional Activity

Past evaluation of seismic fragility was based on conservative, rather than realistic assumptions. The failure mode of concern is catastrophic failure of the pool structure such that an instantaneous loss of water will result and recovery is not possible. Efforts to evaluate the realistic seismic capacity of spent fuel pools should be undertaken by the industry with confirmatory review by the NRC. Through such efforts, it may be feasible to establish that seismic risk from spent fuel pools is negligible, even at the sites where seismic hazard is relatively high, and thus this issue could be eliminated from the risk-informed rulemaking.

July 1999 Public Workshop

In the July 15-16, 1999, workshop it was agreed that a checklist of seismic characteristics that could be verified by the licensee would be a viable way to demonstrate robust seismic capacity of spent fuel pools. NEI volunteered to propose a draft seismic checklist.

NEI Draft Seismic Checklist
4
The draft checklist provided in a letter postmarked August 18, 1999, includes 7 elements which 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 reasonably comprehensive, but it can be improved in several ways as addressed in the comments corresponding to each item in the draft NEI check list:

Item 1: This item should include a caution that the HCLPF value for reinforced concrete shear walls in this context corresponds to in-plane shear. Capacity in out-of-plane shear and flexure is not covered by the HCLPF value cited in this item. The recommended design features and the actual drawings review to ensure adequate reinforcement at openings, anchoring, and lap lengths of reinforcing bars in accordance with the ACI 318-71 or ACI 349-76 or later reinforced concrete codes constitute the most important part of checking for ductility. Although the check list includes a walkdown of the spent fuel pool, it does not mention recording the condition of concrete walls and the liner plates to look for degradation. Similarly, the soil condition around the pool foundation should be examined. Shrinking of soil away from the foundation or any signs of swelling would be indications of underlying foundation degradation. Instances of such preexisting degradation should be noted and evaluated to pass the seismic screening. Inspection of spent fuel pool structures is further discussed under the heading of Structural Failure Modes.

Item 2: This item is similar to the previous one and the review and design features, drawings of reinforcing bars for appropriate details need to be emphasized.

Item 3: The SFP walk-down and design features review should include the overhead crane gantry and support structures.

Items 4-7: These recommendations are adequate.

Revised NEI Draft Seismic Check List

The revised checklist transmitted by a December 13, 1999 letter from Alan Nelson of NEI to William Huffman of NRC, added items on identifying preexisting degradation of concrete and the liner plate, out-of-plane shear and flexure loads, and potential mitigation measures. The revised check list responded to the detailed comments in our letter dated December 3, 1999.

Kennedy Report

Dr. Robert P. Kennedy was requested to review the seismic portions of the June draft preliminary report and the NEI-proposed seismic checklist. 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 (attached) endorses the feasibility of the use of the seismic screening concept and identifies eight sites by site numbers for which the 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. Dr. Kennedy has also suggested that a few more seismic fragility evaluations for spent fuel pools that are not backed up by soil backfill, would be necessary before the seismic checklist can be finalized.

Conclusion

A very large seismic events (beyond design basis) has the potential for causing a catastrophic failure of the pool structure. In general, consideration of seismic event frequency can be decoupled from the decommissioned plant risk; instead, a seismic checklist approach can be used to identify seismic vulnerabilities and adopt corresponding mitigation measures.

Recommendation

The following actions are recommended:

1. The seismic check list be improved to incorporate consideration of out of plane shear and flexure. NEI's revised check list has incorporated this item.
2. Identification of preexisting concrete and liner plate degradation be added to the check list. NEI's revised check list has incorporated this item.
3. Set the target seismic level for Eastern US sites at three times the SSE. The revised check list has incorporated this item.
4. A realistic seismic fragility evaluation should be conducted for a few plants where the pool structure is not backed up by soil backfill. Insights from such analyses are need to finalize the check list. NEI's revised check list has not incorporated this item.
5. Set the target seismic level for higher seismicity sites (west coast locations) be set at 2 times the SSE. This item is not addressed in the NEI check list.

Other 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 onTuesday, 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. Due to the 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., if the probabilistic hazard studies were to be performed again, the hazard estimates for most sites would probably be reduced still further.

The design basls 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 use a multiple hypothesis approach. In this approach, several different methods were used 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, we 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 operating nuclear power plant sites in the U.S. 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 were evaluated against the actual spectra of near field ground motions of a suite of earthquakes gathered on a world wide 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 indicates that this statement is not correct.

We assume 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 would result in less uncertainty and lower hazard estimates today than have previous studies.

Notwithstanding this explanation, there is enough uncertainty in the seismic risk from spent fuel pool structures, to conclude that it is not prudent to base the rulemaking purely on generic risk numbers. This is why a risk-informed performance goal is recommended for a case-by-case assessment of seismic vulnerability of spent fuel pool structures.

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 needs to be examined on a case-by-case basis. This is a good candidate for a 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 a high neutron fluence, 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, a 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 above discussion, 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.