Appendix 2 Structural Integrity of Pool Structure

Introduction

As a part of the Generic Issue 82, "Beyond Design Basis Accidents in Spent Fuel Pools," NRC has studied the hypothetical event of an instantaneous loss of spent fuel pool water. The recommendation from a study in support of this generic issue indicates that a key part of a plant specific evaluation for the effect of such an event is the need to obtain a realistic seismic fragility of the spent fuel pool. The failure or the end state of concern in the context of this generic issue is a catastrophic failure of the spent fuel pool which leads to an almost instantaneous loss of all pool water and the pool having no capacity to retain any water even 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 inch thick, except at Dresden Unit 1 and Indian Point Unit 1. These two plants do not have any liner plates. They were decommissioned more than 20 years ago and no ~~unusual~~ 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 located in the reactor building at an elevation several stories above the ground. In pressurized water reactor (PWR) plants, the spent fuel pool structures are located outside the containment structure supported on 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 in terms of being 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 (April, May, June and in July 1999), there was a two-day public workshop to discuss NRC's draft report of the Technical Working Group. At the public workshop, we proposed and the industry group agreed to develop a seismic check list which could be used to examine the seismic vulnerability of any given plant. This draft report examines the effort so far on the development of a seismic check list.

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Available NRC studies

There are two relevant reports on this issue:

1. NUREG/CR 4982, Severe Accidents in Spent Fuel Pools in Support of Generic Safety Issue 82, Published July 1987.

2. NUREG/CR 5176, Seismic Failure and Cask Drop Analyses of the Spent Fuel Pools at Two Representative Plants, Published January 1989.

Subsequent to the completion of work for the above studies, NRC performed a study to review the central and eastern US probabilistic seismic hazard and issued NUREG-1488, Revised Livermore Seismic Hazard Estimates for 69 Nuclear Power Plant Sites East of the Rocky Mountains, Published, October 1993. It is well recognized that the LLNL seismic hazard curves used prior to 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 and at the H. B. Robinson site the HCLPF value is 0.65 g. A comparison of the 1989 and the 1993 LLNL hazard curves shows 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, 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 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 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 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.

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 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,
    - 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,
    - 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.

- 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 check list. Such an effort may be necessary for plants in higher seismic hazard areas also.

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 not likely, based on probability of the initiating events, and should be eliminated from further consideration in the risk informed decommissioning rule making. NEI arguments are risk based and do not take into account uncertainties associated with the seismic risk which range from 2.4×10^{-4} to 3.1×10^{-11} per ry. 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 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 a 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 which 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 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 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 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 3 time the SSE) and there is also a plant specific structural assessment of the HCLPF 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 consists of the frequency of the initiating event, the earthquake, and the conditional catastrophic failure of the pool structure. Based on numerous past studies conducted 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 were 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 US 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 US coast site) and 2 times the SSE at a higher seismicity location (west coast site) can be considered to be 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 is the proposed performance goal and it 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, that have higher seismic hazards or have lower structural capacity, 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 an effort 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 rule making.

July 1999, Public Workshop

In the July 15-16, 1999, workshop it was agreed that a check list 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 check list.

NEI Draft Seismic Check List

The draft check list provided in a letter post marked August 18, 1999, includes 7 elements which identify areas of potential weaknesses. The use of such a check list would ensure that potential vulnerabilities are either rectified or mitigation measures are put in place. The check list 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 feature and actual drawing review to ensure adequate reinforcement at openings, anchoring and lap lengths of reinforcing bars are 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 walk-down of the spent fuel pool, it does not mention recording condition of concrete walls and the liner plates to look for degradation. Similarly, the soil condition around the pool foundation, such as, shrinking of soil away from the foundation or any signs of swelling would be indications of underlying foundation degradation. Instances of such pre-existing degradation should be noted and evaluated to pass the seismic screening. Additional discussion on inspection of spent fuel pool strictures is provided 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 also.

Items 4-7: These recommendations are adequate.

Revised NEI Draft Seismic Check List

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

Kennedy Report

~~As a part of an independent technical review,~~ Dr. Robert P. Kennedy was requested to conduct ~~this~~ a review of the seismic portions of the June draft preliminary report and the NEI-proposed seismic check list. 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 8 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 check list, an evaluation of the POF using plant specific fragility information will be necessary. For all other sites, the use of the seismic check list 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 check list could be finalized.

Conclusion

~~The potential for any significant adverse impact on public health, from a spent fuel pool at a decommissioned plant induced by seismic events, can only arise from a catastrophic failure of the pool structure. Only~~ A very large seismic event~~s~~ (beyond design basis) ~~have~~ has the potential for causing ~~such damage~~ a catastrophic failure of the pool structure. In general, based on our review consideration of seismic event frequency can be decoupled from the decommissioned plant risk and, instead, a seismic check list approach can be used to identify seismic vulnerabilities and adopt corresponding mitigation measures.

~~Recommendations~~ Staff comments on proposed check list and NEI followup actions

The following items describe the NRC comments from the December 3, 1999, memorandum on the proposed check list. NEI requested a subsequent telephone conference call to discuss recommendations on the staff's comments. On December 13, 1999, NEI submitted proposed revisions to the check list.

1. The seismic check list be improved to incorporate consideration of out of plane shear and flexure. The revised check list has incorporated this item.
2. Identification of preexisting concrete and liner plate degradation be added to the check list.
3. The target seismic level for Eastern US sites be set at three times the SSE. The revised check list has incorporated this item.
4. 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.
5. The target seismic level for higher seismicity sites (west coast locations) be set at 2 times the SSE.

Other Stakeholder Interactions:

From April 13, 1999, stakeholder meeting:

1. A member of the public related to the potential effects of Kobe and Northridge earthquake related to risk informed considerations for decommissioning. The individual stated, "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 look 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."

The two NUREGs mentioned by the individual member of the public were written in the middle and late 1980's 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 bases 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 was made follow the criteria and procedures required by NRC regulations and using a multiple hypothesis approach in which 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 challenges previous design and licensing decisions. The Kobe and Northridge earthquakes were tectonic plate boundary events which occurred 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 focal mechanism. It is also a function of the distance of the facility to the fault and the geology along the travel path of the seismic waves and the geology immediately under the facility site. There are two operating nuclear power plant sites in the U. S. which 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 eight 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 four 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 that 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 what the individual is alluding to is 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 and others, 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 in the site areas for the evaluations of these sites, 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, to determine 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 data base used to develop the ground motion attenuation relationships contain 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 raise 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 those obtained from previous studies.

Notwithstanding the above explanation, there is uncertainty in the seismic risk from spent fuel pool structures is significant enough, to conclude that it is not prudent to base the rule making 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.

From July 15 - 16, 1999, workshop:

1. 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.

These 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 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 check list.

2. 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 the presence of 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.

Concrete gains compressive strength with aging of about 20% in an asymptotic manner; consequently, 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 freeze and thaw effect when 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 Part 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 check list. Significant degradation of reinforced concrete structures would take more than five 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 short period of 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.

Recommendations

1. The target seismic level for Eastern US sites be set at three times the SSE.

2. The target seismic level for higher seismicity sites (west coast locations) be set at 2 times the SSE.

3. To demonstrate a spent fuel pool structure is within recommendations 1 and 2, the licensee shall complete the seismic check list.

Seismic Check List

Item 1: 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: 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: 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: 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: 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: 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: 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: 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: 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 Section 7 and Appendix C of "A Method for Assessment of Nuclear Power Plant Seismic Margin, Revision 1 (EPRI-NP-6041-SL), August 1991.

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.)