Seismic Evaluation Guidance

Spent Fuel Pool Integrity Evaluation

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EPRI Project Manager
J. Richards

ELECTRIC POWER RESEARCH INSTITUTE

3420 Hillview Avenue
Palo Alto, CA 94304-1338
USA

PO Box 10412
Palo Alto, CA 94303-0813
USA

800.313.3774
650.855.2121

askepri@epri.com

www.epri.com

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THE FOLLOWING ORGANIZATION(S), UNDER CONTRACT TO EPRI, PREPARED THIS REPORT:

Jensen-Hughes

Simpson Gumpertz & Heger, Inc

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Acknowledgments

The following organizations, under contract to the EPRI, prepared this report:

Simpson Gumpertz & Heger Inc.
4695 MacArthur Court, Suite 500
Newport Beach, CA 92660

Principal Investigator
G. Hardy

Jensen Hughes
111 Rockville Pike, Suite 550
Rockville, MD 20850

Principal Investigator
B. Tegeler

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Divakar Bhargava, Dominion Resources Inc.
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Product Description

Following the accident at the Fukushima Daiichi nuclear power plant (NPP) resulting from the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami, reviews were conducted, including examinations of the seismic safety of NPPs. In the United States, the Nuclear Regulatory Commission (NRC) established a Near-Term Task Force (NTTF) to 1) conduct a systematic review of NRC processes and regulations and 2) determine whether the agency should make additional improvements to its regulatory system.

Background

From time to time, new assessments of seismic hazard are performed for NPPs around the world. In some cases, updated information has led to an assessment that the seismic hazard is, in some ways, higher than had been previously understood. When a new catalog of seismic sources was compiled for plants in the central and eastern portion of the United States, this catalog was used to develop updated estimates of seismic hazard. At about the same time that this reassessment of seismic hazard was completed, the NTTF developed a set of recommendations intended to clarify and strengthen the regulatory framework for protection against natural phenomena in light of the Great Tohoku Earthquake and the resulting tsunamis. Subsequently, the NRC issued a letter that requested information to ensure that all U.S. NPP licensees address these recommendations.

EPRI report 1025287 provides guidance for conducting seismic evaluations, including those requested in the NRC's letter, which asks that licensees reevaluate the seismic hazards at their sites against present-day NRC requirements and guidance. Section 7 of EPRI report 1025287 provides guidance for performing an evaluation of the spent fuel pool (SFP) that considers all seismically induced failures that can lead to draining of the SFP.

Objectives

- To provide evaluation guidance for the effects of seismic ground motions on SFPs that supplements the SFP evaluation guidance provided in EPRI 1025287

- To provide alternate guidance for performing the structural evaluation and generic implementation guidance for performing the non-structural evaluations

Approach

This report provides SFP seismic evaluation guidance for the structural and non-structural aspects of SFP integrity. Separate guidance is provided for plants in which the ground motion response spectrum (GMRS) peak spectral acceleration is less than or equal to 0.8g, and for plants in which the GMRS peak spectral acceleration is greater than 0.8g.

The seismic evaluation criteria in EPRI report NP-6041 are applied to address seismic adequacy of the SFP structure up to 0.8g spectral acceleration. Above 0.8g spectral acceleration, structural analysis criteria are provided for evaluating failure of the SFP walls and floor.

Generic seismic assessments are also provided for the non-structural elements of SFP piping penetrations, refueling gates, and potential siphoning conditions. Evaluation criteria are also provided for SFP seismic-induced sloshing losses and boil-off losses using site-specific parameters to evaluate the ability of SFPs to retain adequate water inventory for 72 hours.

Results and Findings

The evaluations provided in this report show that SFPs can retain adequate water inventory for 72 hours provided the plant meets a limited set of parameters or passes a focused structural evaluation of the SFP walls and floor. A key conclusion of the report is that the seismic-induced SFP inventory losses are modest and that the majority of inventory losses over 72 hours are a result of evaporation and boil-off.

Applications, Value and Use

The criteria presented in this report support the SFP seismic evaluations as identified in EPRI 1025287 and licensee responses to the NRC 50.54(f) letter. These criteria can also be

applied at any plant performing an SFP seismic evaluation for beyond design basis accelerations.

Keywords
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EXECUTIVE SUMMARY

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PRIMARY AUDIENCE: Engineers developing responses to the seismic portion of the NRC's Fukushima 50.54(f) letter

SECONDARY AUDIENCE: Engineers at any nuclear plant performing seismic evaluations of Spent Fuel Pools

KEY RESEARCH QUESTION

How can users perform the Spent Fuel Pool seismic evaluations in response to the seismic portion of the NRC's Fukushima 50.54(f) as identified in EPRI 1025287?

RESEARCH OVERVIEW

This report provides guidance for performing seismic evaluations of spent fuel pools. The report supplements guidance provided in EPRI 1025287, *Seismic Evaluation Guidance: Screening, Prioritization and Implementation Details (SPID) for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic* for performing seismic evaluations that may be required as part of the resolution of the U.S. Nuclear Regulatory Commission's Near Term Task Force Recommendation 2.1: Seismic. EPRI Technical Update 3002007148, *Seismic Evaluation Guidance: Spent Fuel Pool Integrity Evaluation* provided guidance for performing SFP evaluations for plants where the ground motion response spectrum (GMRS) peak spectral acceleration was equal to or less than 0.8g. This report replaces 3002007148 and adds guidance for plants with higher GMRS. The previous guidance in 3002007148 for plants with lower GMRS is carried forward unchanged in this report.

KEY FINDINGS

- The report provides relatively straightforward evaluation criteria for demonstrating that an SFP will retain adequate water inventory for 72 hrs following a seismic event, as specified be EPRI 1025287.
- For plants where the GMRS peak spectral accelerations (Sa) ≤ 0.8g, generic evaluations are provided in Section 3 crediting structural criteria from NP-6041 SLR1, *A Methodology for Assessment of Nuclear Power Plant Seismic Margin*. Generic evaluations are also provided for non-structural elements that could lead to SFP rapid drain down (refueling gates, piping penetrations, siphoning, sloshing, and bloi off). Plant specific parameters are identified that when confirmed, show that the SFP will retain adequate water inventory.
- For plants where the GMRS peak Sa > 0.8g, criteria are provided in Section 4 to analyze the SFP walls and floor as single degree of freedom plates using plant theory and site-specific parameters. Generic evaluations are also provided for non-structural elements that could lead to SFP rapid drain down (refueling gates, piping penetrations, siphoning, sloshing, and boil off). Plant specific parameters are identified that when confirmed, show that the SFP will retain adequate water inventory.
- Sample boil off calculations are provided in Appendix B and sample SFP wall and floor structural calculations are provided in Appendix C.
- As noted in this report, the SFP sloshing calculation in EPRI 1025287 is conservative. It was applied in this report because the results were acceptable with that conservatism. If plants have difficulty meeting these criteria, a more detailed sloshing evaluation may be warranted.

This publication is a corporate document that should be cited in the literature in the following manner:

Seismic Evaluation Guidance: Spent Fuel Pool Integrity Evaluation EPRI, Palo Alto, CA: 2016. 300200xxxx

WHY THIS MATTERS

This report provides guidance to assist U.S. nuclear plants in responding to NRC Near-Term Task Force Recommendation 2.1: Seismic.

The report also provides value to any plant performing a beyond-design-basis seismic evaluation of a spent fuel pool.

HOW TO APPLY RESULTS

Engineers required to perform spent fuel pool seismic evaluations should focus on Sections 3 and 4 of the report.

Users of this report also may want to consult EPRI 1025287, which provides overall guidance for performing evaluations for NTTF Recommendation 2.1 Seismic.

LEARNING AND ENGAGEMENT OPPORTUNITIES

Users of this report may be interested in periodic Early SPRA Workshops sponsored by EPRI. Contact John Richards at 704.595.2707 or jrichards@epri.com for additional information.

EPRI CONTACTS: John Richards, Principal Technical Leader, jrichards@epri.com, 704.595.2707

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Electric Power Research Institute
3420 Hillview Avenue, Palo Alto, California 94304-1338 • PO Box 10412, Palo Alto, California 94303-0813 USA
800.313.3774 • 650.855.2121 • askepri@epri.com • www.epri.com

List of Acronyms

ACI	American Concrete Institute
Btu	British thermal unit
BWR	boiling water reactor
CEUS	central and eastern United States
CFR	Code of Federal Regulations
cm	centimeter
D	panel flexural rigidity
E_{FC}	floor panel combined demand (100-40-40)
E_{FH}	floor panel hydrodynamic demand
E_{FI}	floor panel inertial demand
E_{FV}	floor panel demand in vertical direction
EPRI	Electric Power Research Institute
E_W	wall panel demand in horizontal direction
E_{WC}	wall panel combined demand (100-40-40)
E_{WH}	wall panel hydrodynamic demand
E_{WI}	wall panel inertial demand
°F	degrees Fahrenheit
ft	feet
F_S	strength factor
g	acceleration due to gravity
GMRS	ground motion response spectrum
HCLPF	high confidence of low probability of failure
hr	hour(s)
Hz	Hertz
in	inches
K_e	panel elastic stiffness
K_E	equivalent panel stiffness (flexure controlled)
K_{ES}	equivalent panel stiffness (shear controlled)
K_{ep}	panel elastic-plastic stiffness
ksi	kilopound per square inch
lb	pound
m	meter
M_n	nominal concrete strength
MPa	megapascal
M_t	total panel mass

MW	megawatts
MW_t	megawatts thermal
NPP	nuclear power plant
NRC	Nuclear Regulatory Commission
NTTF	Near Term Task Force
PGA	peak ground acceleration
psi	pound per square inch
PWR	pressurized water reactor
R_e	panel elastic strength
RG	Regulatory Guide
RLE	review level earthquake
RPV	reactor pressure vessel
R_{UF}	maximum panel flexural strength
R_{US}	maximum panel shear strength
R_{USL}	panel shear strength in longitudinal direction
R_{UST}	Panel shear strength in transverse direction
Sa	spectral acceleration
SFP	spent fuel pool
SPID	Screening, Prioritization, and Implementation Details
SPRA	seismic probabilistic risk assessment
SRSS	square root sum of the squares
SSC	structures, systems, and components
SSE	Safe Shutdown Earthquake
T	fundamental panel period
UHS	uniform hazard spectra
U_{TOT}	combined seismic and dead loads
V_n	nominal shear strength
W_e	yield line analysis external work
W_i	yield line analysis internal work
Y_e	panel elastic displacement
Y_E	equivalent panel displacement
Y_{ep}	panel elastic-plastic displacement

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Section 1: Introduction and Purpose

Many plants in the United States are performing seismic evaluations of spent fuel pools (SFP) in response to the Nuclear Regulatory Commission's (NRC) 50.54(f) letter [1] requesting that plants perform a number of seismic evaluations using updated site-specific seismic hazards. The primary guidance for performing those evaluations is provided in EPRI 1025287 [2].

The 50.54(f) letter requested that, in conjunction with the response to Near Term Task Force (NTTF) Recommendation 2.1, a seismic evaluation be made of the SFP. More specifically, plants were asked to consider "…all seismically induced failures that can lead to draining of the SFP." This report provides evaluation guidance for the effects of seismic ground motions on SFPs that supplements the SFP evaluation criteria provided in EPRI 1025287.

Although the guidance in this document is specifically directed at supporting responses to the 50.54(f) letter, the evaluation guidance is applicable to any SFP seismic evaluation.

1.1 NRC Near Term Task Force Recommendations

Following the accident at the Fukushima Daiichi Nuclear Power Plant (NPP) resulting from the March 11, 2011 Great Tohoku Earthquake and subsequent tsunami, the U.S. NRC established the (NTTF) in response to Commission direction. The NTTF issued a report with a series of recommendations, some of which were to be acted upon "without unnecessary delay," concerned with the capability of NPPs to deal with extreme events. NTTF Recommendation 2.1 instructed NRC staff to issue requests for licensees to re-evaluate the seismic hazards at their sites, using present-day NRC requirements and guidance, and to identify and address any site-specific vulnerabilities associated with the updated seismic hazards. Subsequently in 2012, the NRC issued a 50.54(f) letter [1] that requested information to ensure that these recommendations were addressed by all operating U.S. NPPs.

The NRC requested that each plant provide information about the updated hazard on an accelerated schedule and proposed a progressive screening/evaluation approach to evaluate the potential risk posed by future seismic events. While the full seismic hazard studies were requested, the measure of the re-evaluated seismic hazard for a given site

was provided by a new horizontal ground motion response spectrum (GMRS) developed using updated uniform hazard spectra (UHS) [1]. Depending on the comparison between the GMRS and the current design basis spectrum, the plants either were screened-out from further evaluation or were screened-in to perform a seismic risk assessment using the updated seismic hazard.

1.2 Industry Response

EPRI 1025287, *Seismic Evaluation Guidance, Screening, Prioritization and Implementation Details (SPID) for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic* [2] provided screening, prioritization, and implementation details to the U.S. nuclear utility industry for the resolution of NTTF Recommendation 2.1: Seismic. This report was developed with NRC participation and was subsequently endorsed by the NRC. The SPID [2] provided screening guidance for the comparison of site-specific horizontal GMRS, developed from the new seismic hazard evaluations, with the site safe-shutdown earthquake (SSE).

The NRC 50.54(f) letter [1] requested that seismic evaluations be performed for SFPs to consider all seismically induced failures that could lead to draining of the SFP. Section 7 of the SPID [2] describes an approach for performing the requested SFP drain-down evaluations.

The approach outlined in the SPID focuses on those plants that have GMRS exceedances of the SSE in the frequency range of structural significance (i.e., 1 to 10 Hz). For those plants where the GMRS exceeds the SSE in the 1 to 10 Hz frequency range, spent fuel pool evaluations are required. The SPID [2] provides guidance on how to consider possible failures that could lead to a rapid drain-down uncovering more than 1/3 the height of fuel stored in the SFP within 72 hours. Such failures that could conceivably lead to SFP rapid drain-down events would include the following:

- A significant failure of the steel-lined, reinforced concrete structure of the SFP, causing inventory of the pool to drain out.
- Failure of a connection penetrating the SFP structure (drain line, cooling water line, etc.) below the top of the stored fuel.
- Failure of a connection penetrating the SFP structure above the fuel sufficient to drain significant inventory from the pool such that (in absence of adequate makeup) evaporation and boil-off would cause fuel to be uncovered within 72 hours.
- Extensive sloshing such that sufficient water could be lost from the pool and lead to uncovering of fuel within 72 hours.
- Failure of a cooling-water line or other connection that could siphon water out of the pool sufficient to lead to uncovering of the fuel within 72 hours.

With regard to the above scenarios, the SPID [2] notes that the SFP structure can be evaluated using a checklist described in NUREG-1738 [4], or another approach can be used if sufficiently justified. The SPID also provides additional criteria for selecting and evaluating SFP penetrations, estimating sloshing losses, and estimating boil-off losses using a method in EPRI 1025295 [17].

As part of the rapid drain-down evaluations, licensees may consider the ability to make up inventory losses to ensure that the spent fuel remains adequately covered. SFP inventory makeup strategies can be credited provided; makeup resources, including any necessary instrumentation, are seismically rugged and available; procedures exist to guide the response by the operator; and there is sufficient time for operators to recognize the need for makeup and take action. Credited operator actions should be reviewed to account for habitability and accessibility limitations.

In addition to the SFP evaluation guidance in the SPID [2], seismic walkdowns were performed of plant equipment, including SFP equipment, in accordance with EPRI 1025286 [5] to address NTTF Recommendation 2.3. These walkdowns were performed to verify that the current plant SFP configuration is consistent with the design basis, identify degraded, nonconforming, or unanalyzed conditions, and verify the adequacy of licensee monitoring and maintenance programs. The walkdown criteria in EPRI 1025286 [5] address equipment anchorage, seismic spatial interactions, and adverse seismic conditions. Any potentially degraded, non-conforming, or unanalyzed conditions identified during the seismic walkdown program were to be assessed in accordance with the plant corrective action program.

The SFP seismic walkdowns also focused on SFP connections whose failure could result in a rapid drain-down. The rapid drain-down event was defined as a failure that could lead to uncovering of spent fuel assemblies stored in the SFP within 72 hours of the earthquake.

1.3 Purpose of Report

The purpose of this report is to provide supplemental guidance for performing the SFP evaluations identified in the SPID [2]. EPRI Technical Update 3002007148 [26] described the technical basis supporting the evaluation of plants with low-to-moderate seismic ground motions, or peak spectral accelerations up to 0.8g. This final report adds guidance for plants with higher GMRS. The previous guidance for lower GMRS plants in EPRI 3002007148 is carried forward unchanged in this report.

Section 2 of this report provides an overall description of SFP general arrangements and systems functions. Distinctions are made between Pressurized Water Reactor (PWR) and Boiling Water Reactor (BWR) (Mark I, II, and III) SFP designs. This section also provides a brief

description of the seismic classification of SFP structures, systems, and components (SSC).

Section 3 of this report provides evaluation criteria for SFPs at sites with GMRS peak spectral accelerations up to 0.8g. An approach is described for evaluating the SFP structure that can be used as an alternate to the NUREG-1738 checklist identified in the SPID [2]. This section also provides criteria for evaluating the non-structural aspects of the SFP, such as piping connections, fuel gates, and anti-siphoning devices, as well as an evaluation of SFP sloshing and an approach for assessing the heat up and boil-off of SFP water inventory. The results are based on industry survey results, site-specific GMRS demands, conservative estimates of sloshing losses, and realistic SFP heat loads. Finally, Section 3 of this report provides screening criteria, which will enable licensees to confirm that their site-specific parameters are within the bounds of the criteria considered in this report.

Section 4 of this report provides evaluation criteria for SFPs at sites with GMRS peak spectral accelerations greater than 0.8g. An approach is described for evaluating the SFP structure using single-degree-of-freedom plate analysis techniques that can be used as an alternate to the NUREG-1738 checklist identified in the SPID [2]. This section also provides criteria for evaluating the non-structural aspects of the SFP, such as piping connections, fuel gates, and anti-siphoning devices, and adopts the same methods as Section 3 for evaluating SFP sloshing and assessing the heat up and boil-off of SFP water inventory. The results are based on industry survey results, site-specific GMRS demands, conservative estimates of sloshing losses, and realistic SFP heat loads. Finally, Section 4 of this report provides screening criteria, which will enable licensees to confirm that their site-specific parameters are within the bounds of the criteria considered in this report.

Appendices provide a summary of a survey of SFP characteristics, an example SFP heat up and boil-off calculation of a representative pool, and example SFP wall and floor plate analysis methods.

Section 2: Characteristics of Spent Fuel Pools

2.1 Spent Fuel Pool Characteristics

Spent fuel pools (SFPs) are generally rectangular in cross section and approximately 40 feet (12 m) deep. Both BWR and PWR reactor SFPs typically range from 30 to 60 feet (9 to 18 m) in length and 20 to 40 feet (6 to 12 m) in width. For multi-unit sites there may be a shared SFP, two SFPs that may or may not be connected, or a separate SFP which stores used nuclear fuel for multiple units. Fuel assemblies are placed vertically in storage racks which maintain an adequate spacing to prevent criticality and to promote natural convective cooling in water medium. The pools themselves are constructed of reinforced concrete with sufficient thickness to meet radiation shielding and structural requirements, and are lined with stainless steel plates of approximately ¼ inch (0.64cm) thickness to ensure a leak-tight system.

Because of design features of the reactor pressure vessel (RPV) and containments, there are characteristic differences between BWR and PWR SFP locations. For example, BWR reactor vessels are taller than PWR vessels. A typical BWR RPV may be over 66 ft (20 m) in height, while a PWR RPV may be 44 ft (13.4 m) in height (see Figure 3-1). PWRs operate the reactor using borated water as one reactivity control measure; BWRs do not. PWRs do not have a Reactor Building surrounding the containment [6, 7].

PWRs typically have a spent fuel storage pool which is located in the Fuel Building or Auxiliary Building (not the Reactor Building) and is either embedded partially below ground or is significantly closer to ground level than the BWR Mark I and II plants. Some PWR SFPs are positioned such that their used nuclear fuel is below grade.

Mark I and II boiling water reactors are designed with the SFP within the secondary containment (reactor building), with the fuel pool adjacent to the RPV cavity. The bottom of the pool is usually elevated approximately 50 ft (15m) above ground level, which places the top of the pool at the level of the refueling floor. Mark III BWRs have their SFP outside of secondary containment in the fuel handling building, with the bottom of the pool at ground level. During BWR Mark I and II refueling operations,

the spent fuel is moved directly from the RPV to the SFP through a narrow refueling cavity. Under normal operations, the cavity is sealed with removable steel gates. Removable concrete blocks are also installed for radiation shielding purposes.

Both BWR and PWR SFPs are designed such that failure of any SFP cooling line penetrating the pool will not permit the water level to drop below approximately 10 ft (3 m) above the top of the spent fuel racks. All lines that penetrate the SFP are provided with isolation valves located as close as possible to the penetration.

Piping design is such that it is not possible to siphon the SFP water level down as the result of a failed pipe or component to a water level below approximately 10 ft (3 m) above the top of the spent fuel rack. This level provides adequate shielding and cooling of the spent fuel while system repair is completed.

2.2 Brief Systems Description

A typical SFP cooling system consists of circulating pumps, heat exchangers, filter-demineralizers, a makeup tank, piping, valves, instrumentation and their structural supports (Figures 2-1 and 2-2). The pumps circulate the pool water in a closed loop, taking suction from the pool, circulating the water through heat exchangers and filters, and discharging through diffusers at or near the bottom of the pool. The SFP system takes suction from the SFP through a skimmer (or strainer) at an elevation that is typically higher in the SFP (e.g., more than ten feet above the top of the fuel assemblies. The SFP cooling return lines either discharge near the top of the SFP or extend deeper into the pool to distribute coolant to the bottom of the fuel. For systems where the SFP coolant lines extend deep into the pool, anti-siphon devices (e.g., drilled holes or valves) are used to prevent loss of inventory should there be a break in the piping system [8].

Each plant has a source of high purity water to provide makeup to the SFP. The typical sources are the refueling water storage tank (borated water) for PWRs and the condensate storage tank (demineralized) for BWRs. Plants will also typically have alternate sources of makeup if normal makeup is unavailable, and may include the service water system and the fire protection system [8].

The spent fuel assemblies are stored in stainless steel racks and submerged with approximately 23 feet (7.01 m) of water above the top of the stored fuel [8]. In addition to cooling, the SFP water inventory provides radiological shielding for personnel in the fuel pool area and adjacent areas. Many plants assume that a minimum of 5-10 feet (1.52-3.05 m) of water above the fuel assemblies provides adequate shielding [8].

During refueling operations, the refueling cavity above the reactor is filled with water equal to the water level in the SFP. Fuel is moved from the reactor to the SFP via transfer canals in BWRs or transfer tubes in PWRs. Removable gates, or refueling gates, are used in both PWR and BWR applications to isolate the SFP during normal operations. These gates are further discussed in Section 3.2.3 of this report.

2.3 Seismic Classification

Buildings that house the SFPs, as well as the pool structure itself, are required to be Seismic Category I and designed to the SSE [9]. However, due to the distribution of U.S. NPP vintage, there is variability in the classification of the SFP cooling and makeup systems. For the design of SFPs, Regulatory Guide (RG) 1.13, "Spent Fuel Storage Facility Design Basis," (Rev 1, 1975) [9] requires that drains, permanently connected mechanical or hydraulic systems, and other features that by maloperation or failure could cause loss of coolant resulting in uncovering the fuel should not be installed or included in design. Systems for maintaining water quality and quantity should be designed so that any maloperation or failure of such systems (including failures resulting from the SSE) will not cause fuel to be uncovered. RG 1.13 states that these systems are not otherwise required to meet Seismic Category I requirements.

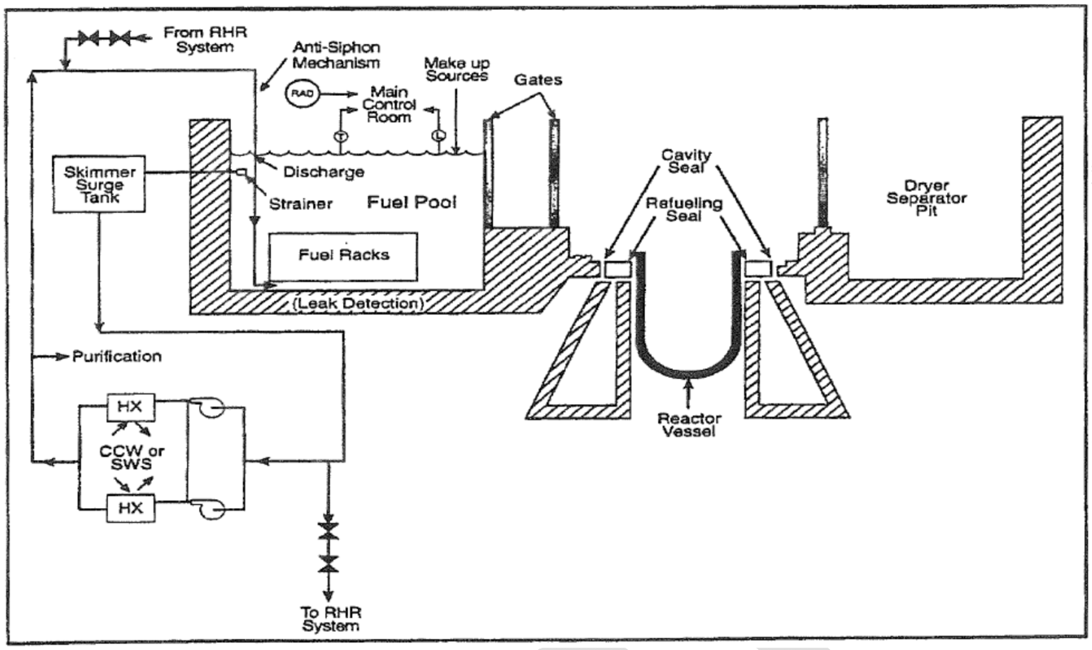

Figure 2-1
Typical BWR SFP Cooling System (Source: NUREG-1275)

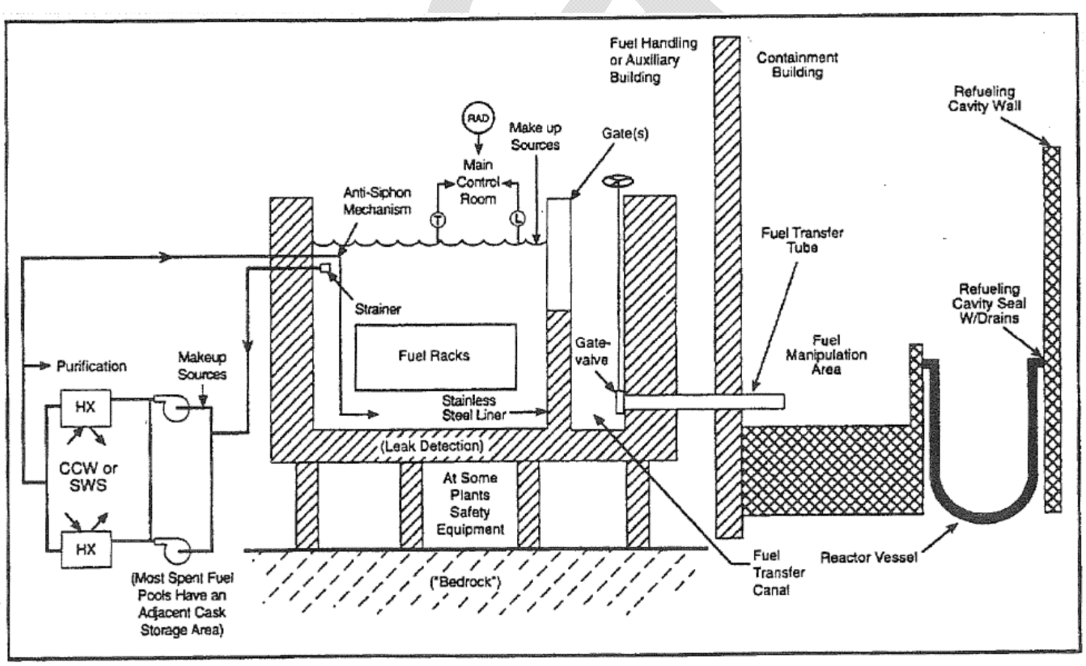

Figure 2-2
Typical PWR SFP Cooling System (Source: NUREG-1275)

Section 3: Seismic Review of Spent Fuel Pools at Low GMRS Sites

Spent fuel pool evaluation criteria are provided in this section for plants with GMRS peak spectral accelerations (Sa) less than 0.8g. The criteria address SFP structural elements (e.g., floors, walls, and supports), non-structural elements (e.g., penetrations), seismic-induced SFP sloshing, and loses due to heat-up and boil-off.

3.1 Spent Fuel Pool Structural Evaluation

Section 7 of the SPID [2] identifies a checklist in NUREG-1738 [4] as an acceptable way of demonstrating that an SFP structure is sufficiently robust. The SPID [2] also allows for an alternate approach if it is sufficiently justified. This section describes an alternate SFP structural evaluation approach for plants with GMRS peak spectral accelerations (Sa) less than 0.8g, using an accepted method of assessing seismic capacity of nuclear power plant SSCs in EPRI NP-6041 [10].

3.1.1 Background on Spent Fuel Pool Structures

Spent fuel pool structures are typically constructed as part of the reactor building or as part of a separate structure to support the fuel handling operations at the reactor. The SFP structures at NPPs currently operating in the U.S. are configured differently depending on the reactor design vintages, site-specific design requirements and also to the design preferences of the engineering and construction companies involved in the design of the facility. To support the NTTF 2.1 seismic assessment of the SFPs, industry surveys were conducted to identify the structural characteristics of the SFPs and their supporting structures for the fleet of U.S. operating nuclear plants. The key elements from that survey are summarized in Table 3-1 below. Additional summaries are presented in Appendix A.

The fundamental structural configurations of the SFPs themselves have similar characteristics due to functional design requirements (including radioactive shielding considerations) and due to structural design loading requirements (seismic, dead weight, etc.). The SFPs are constructed of reinforced concrete shear walls with stainless steel liners attached to the floors and walls. The SFPs are rectilinear with adjoining compartments

next to the fuel storage pool for various operations, such as a station for loading and unloading fuel, and a canal for transferring the fuel assemblies into and out of the reactor. The industry SFP survey results included structural characteristics such as wall spans, thicknesses, concrete strength, and percentage of steel reinforcing.

Table 3-1
Summary of Key Spent Fuel Pool Dimensional and Strength Parameters

Parameter	Minimum	Maximum	Average
Wall span	30 ft (9.1 m)	120 ft (36.6 m)	52 ft (15.9 m)
Wall thickness	42 in (to 106.7 cm)	96 in (243.8 cm)	64 in (162.6 cm)
Concrete strength	3 ksi (20.7 MPa)	5 ksi (34.5 MPa)	3.6 ksi (24.8 MPa)
Reinforcing ratio	0.1%	0.9%	0.3%
Reinforcing strength	24 ksi (165 MPa)	60 ksi (414 MPa)	52 ksi (359 MPa)
Liner thickness	1/8 in (0.32 cm)	3/8 in (0.95 cm)	1/4 in (0.64 cm)

The characteristics of the structures supporting/housing the SFPs were also part of the industry SFP surveys. The SFPs are part of three different nuclear structures depending on the site design:

- Auxiliary Building – 33% of the plants
- Fuel Building – 38% of the plants
- Reactor/Containment Buildings – 29% of the plants

The Boiling Water Reactors (BWR) with Mark I and Mark II containment designs typically have different designs of the structures housing the SFPs than both the Pressurized Water Reactors (PWR) and the BWR Mark III designs. The spent fuel storage pools at BWR Mark I & II sites are typically located within the BWR reactor building at an elevation above grade, which allows alignment of the top of the pool with the operating deck used for re-fueling the reactor. Figure 3-1 depicts one of the early BWR plant configurations with the SFP elevated in the reactor building. The BWR structures housing the SFPs are typically designed with reinforced concrete shear walls providing the primary structural load path. In a few cases, the primary load path also contains reinforced concrete moment frame elements or structural steel frame members. In one case, the structural load path included post-tensioned concrete walls associated with the containment structure.

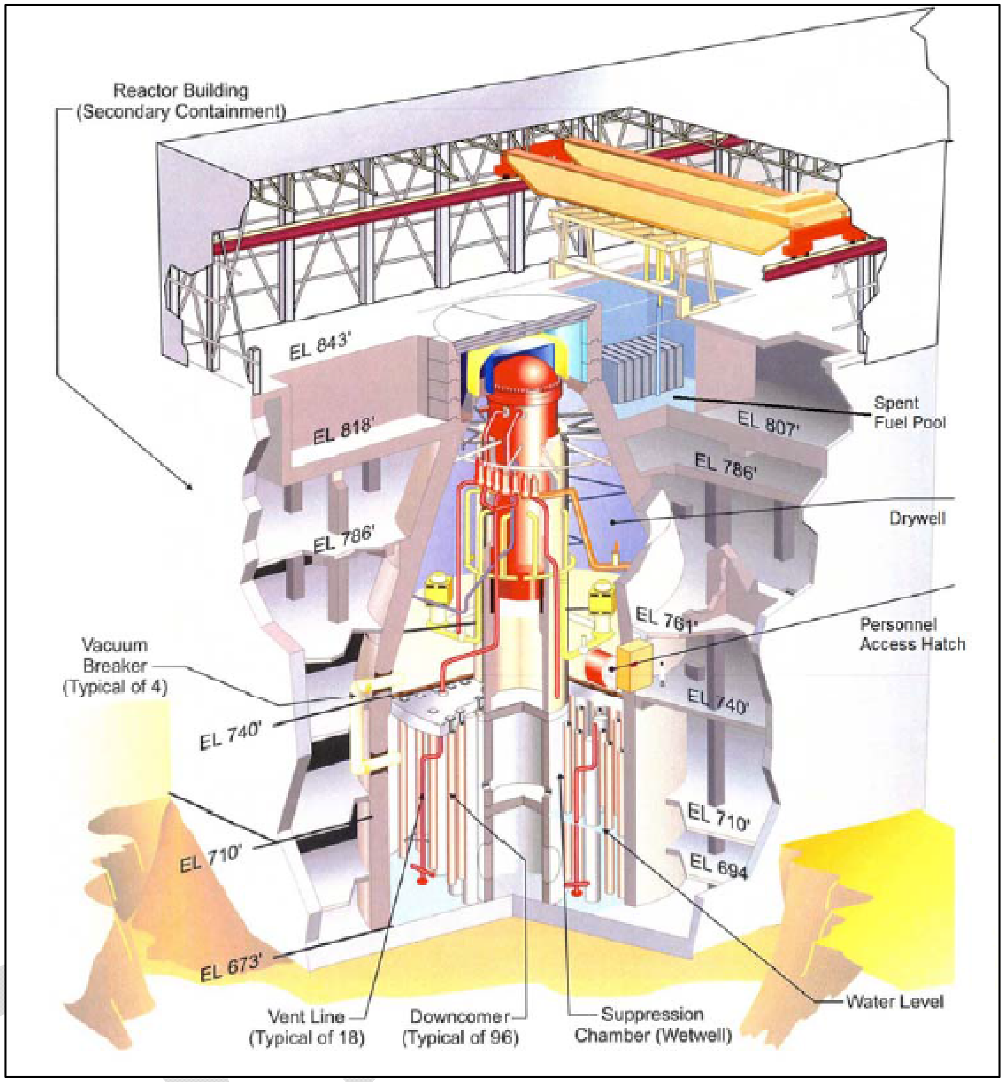

Figure 3-1
Schematic for Typical Boiling Water Reactor Configuration with Elevated Spent Fuel Storage Pool

At PWR and BWR Mark III sites, the floor (bottom) of the pool is generally on or even partially below grade with the pool floor constructed as part of a thick foundation. Figure 3-2 depicts a typical PWR plant configuration including the location of the SFP. The structures housing the SFP typically have load paths with reinforced concrete shear walls with the pool bottom typically supported directly on the building foundation. As with the BWR structures, there are a few PWRs where the SFP structural load paths include reinforced concrete frame members and/or structural steel frame members.

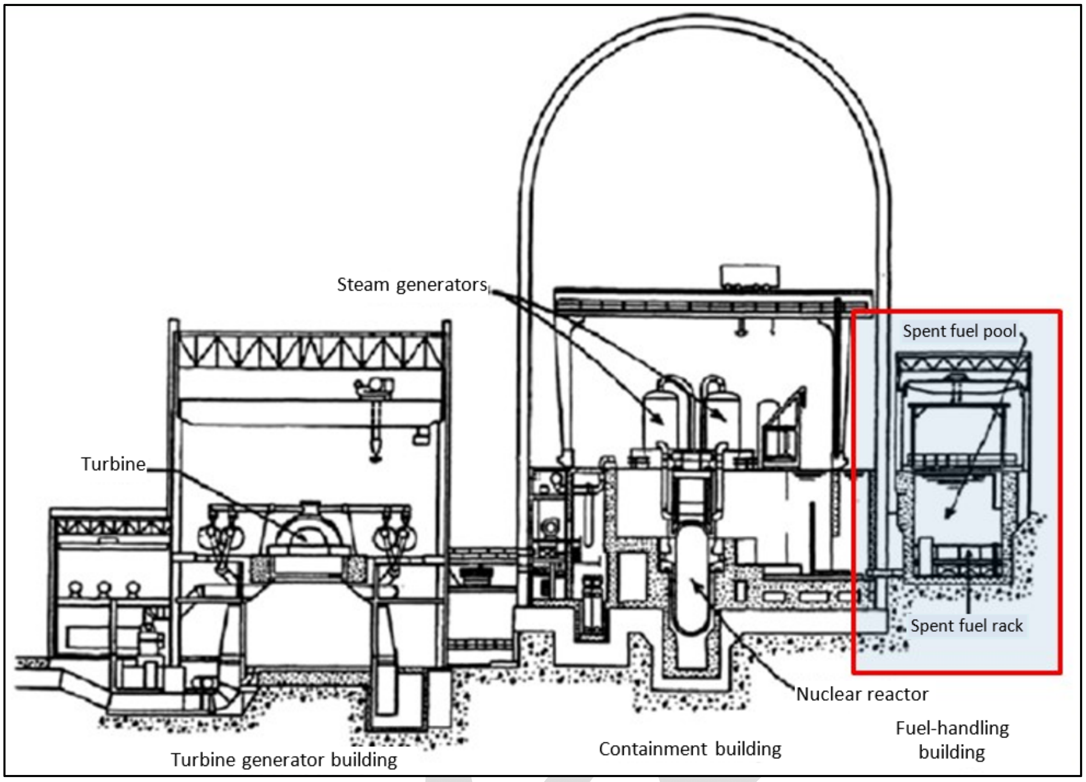

Figure 3-2
Schematic for Typical Pressurized Water Reactor Configuration with Spent Fuel Storage Pool

3.1.2 Treatment of Spent Fuel Pool Structures within NTTF 2.1 Seismic

The 50.54(f) letter [1] requested that, in conjunction with the response to NTTF Recommendation 2.1, a seismic evaluation be made of the SFP. More specifically, plants were asked to consider "…all seismically induced failures that can lead to draining of the SFP." Such an evaluation would be needed for any plants that are not screened from further assessment based on the screening process documented in the SPID [2].

Previous evaluations in NUREG-1353 [11], NUREG-1738 [4] and NUREG/CR-5176 [12] characterized the generally robust nature of the design of SFPs currently in use. NUREG-1738 further identified a checklist that could be used to demonstrate that a SFP would achieve a high confidence of a low probability of failure (HCLPF) of at least 1.2g spectral acceleration. Evaluations reported in NUREG/CR-5176 [12] for two older plants concluded that "…seismic risk contribution from SFP structural failures is negligibly small." Tearing of the stainless-steel liner due to overall structural failure of the fuel pool structure would be precluded by the successful completion of the EPRI NP-6041 structural evaluations. Tearing of the stainless-steel liner due to sliding or other movement of the fuel assemblies in the pool is considered to be very

unlikely [11]. The SPID [2] states that either the checklist in NUREG-1738 [4] can be used to demonstrate that the structure is sufficiently robust or another approach can be used if sufficiently justified. The purpose of this section is to present the seismic adequacy justification for SFP structures at nuclear power plant sites with a relatively low GMRS.

As noted in the SPID [2], the screening criteria for civil structures in EPRI NP-6041 [10] provide principles that are helpful in evaluating the seismic capacity of SFP structures. The approach used for the screening of the lower GMRS sites is the EPRI NP-6041 Table 2-3 assessment criteria. As noted earlier in Section 3.1.1, SFP structures have structural load paths consisting of one or more of the following structural configurations:

- Reinforced concrete shear walls
- Reinforced concrete moment frames
- Structural steel frames
- Post-tensioned containments

As such, the SFPs and their supporting structures all fall within four rows of the NP-6041 Table 2.3 [10] addressing these four structural configurations. Table 3-2 shows an excerpt of the NP-6041 Table 2-3 structural screening criteria.

The first capacity column in Table 3-2 presents the requirements for the assessment of different types of structures to a 5% damped ground motion peak spectral acceleration of 0.8g. The SFP structures for all NPPs fall within the first, fourth, sixth and seventh rows of Table 3-2. Row #1 addresses concrete containments designed using post-tensioning and reinforcement. These post-tensioned containment structures have been shown to be rugged up to the 0.8g peak spectral acceleration level and can be screened from further consideration based solely on demonstration of meeting the "<0.8g" spectral acceleration criteria in the first capacity column. For the "<0.8g" of Table 3-2, the footnote requirements for the other three structural configurations (shear wall structures, concrete frame structures and steel frame structures) considered in this SFP study are limited to the single footnote "e", which states:

> e. *Evaluation not required for Category I structures if design was for a SSE of 0.1g or greater.*

All spent fuel pool structures are, by necessity, Category 1 structures since they contain spent fuel and are designed to the site SSE. All U.S. nuclear plants have design basis SSEs (or the equivalent Design Basis Earthquakes) at or exceeding the 0.1g threshold. Thus, all operating U.S. nuclear plant SFP structures meet the EPRI NP-6041 [10] criteria that demonstrates that they have a high confidence of exceeding the 0.8g spectral acceleration capacity in the free field. The criteria associated with EPRI NP-6041 stipulates that the 0.8g screening level would apply to sites

where the peak spectral acceleration of their review level earthquake (RLE) is less than or equal to this 0.8g spectral acceleration. For purposes of the SFP structure review, the GMRS is used as the RLE.

Table 3-2
Excerpted Table 2-3 from EPRI NP-6041

EPRI NP-6041 Table 2-3
Summary Of Civil Structures Criteria For Seismic Margin Evaluation
(Page 1 of 2)

Row	Type of Structure	< 0.8g	0.8 - 1.2g	> 1.2g
1	Concrete containment (post-tensioned and reinforced)	no	(a)*	(b)
2	Freestanding steel containment	(c)(d)	(c)(d)	yes
3	Containment internal structures	(e)	(f)	yes
4	Shear walls, footings and containment shield walls	(e)	(f)	yes
5	Diaphragms	(e)	(g)	yes
6	Category I concrete frame structures	(e)	(f)	yes
7	Category I steel frame structures	(e)	(h)	yes
8	Masonry walls	yes	yes	yes
9	Control room ceilings	(i)	(i)	yes
10	Impact between structures	no	(j)	yes
11	Category II structures with safety-related equipment or with potential to fail Category I structures	(k)	yes	yes

3.1.3 Structural Evaluation Criteria for Plants with GMRS Sa Less Than 0.8g

All U.S. operating nuclear plants have submitted GMRS values to the NRC. Based on the review of the industry GMRS submittals and the NRC responses [13], 21 plants screen out of having to conduct a review of the SFPs. Of the remaining 40 plants, 31 plants have GMRS peak spectral acceleration (5% damping) values are below the 0.8g peak spectral acceleration threshold value (Figure 3-3).

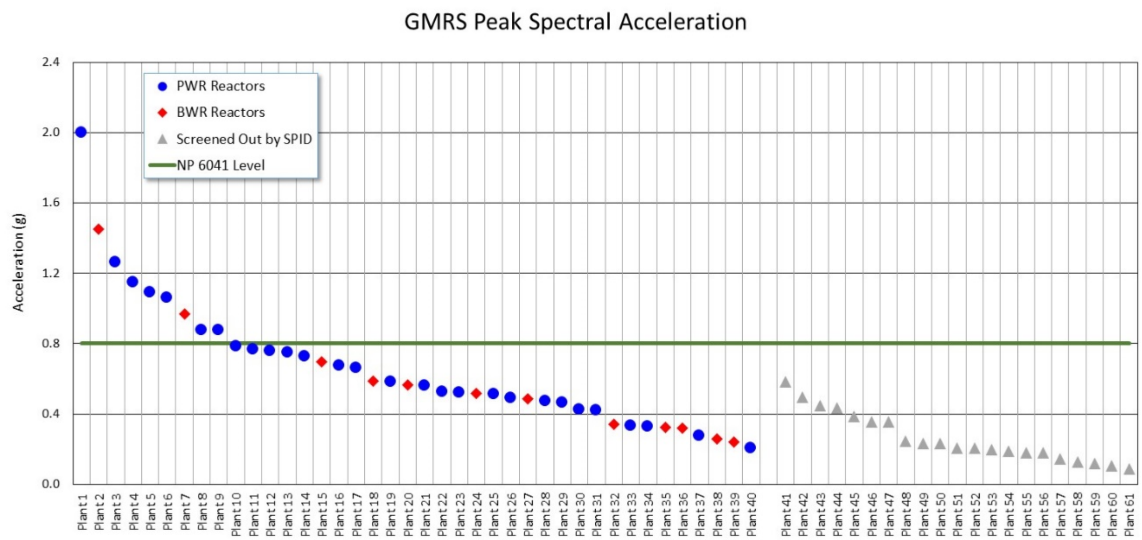

Figure 3-3
GMRS Peak Spectral Acceleration Comparisons to 0.8g Ground Spectral Acceleration Threshold for U.S. Plants

Thus, based on the criteria in Table 3-2, these 31 plants can demonstrate seismic adequacy of the SFP structure to the GMRS level. As such they can demonstrate that they have adequate seismic capacity to withstand the new seismic hazard at their sites by verifying that:

a) the GMRS is less than or equal to the 0.8g spectral acceleration capacity level from column 1 of Table 3-2,

b) the structure housing the SFP was designed to an SSE of at least 0.1g,

c) the structure load path to the SFP consists of some combination of reinforced concrete shear wall elements, reinforced concrete frame elements, post-tensioned concrete elements and/or structural steel frame elements, and

d) the SFP structure is included in the Civil inspection program in accordance with the NRC Maintenance Rule (10 CFR 50.65) [14].

While not required by the checks within EPRI NP-6041 [10] for the low ground motion site reviews (column 1 of Table 3-2), a review of the potential for out-of-plane response was conducted. Three previous studies on the seismic capacities of SFPs were reviewed to determine the lowest HCLPF values associated with the out-of-plane response of the SFP walls and floors. The results of those studies indicate relatively high HCLPF values compared to the 0.8 peak spectral acceleration (PSA) value (or approximately 0.3g PGA) associated with the first column of Table 3-2:

- NRC SFP Scoping Study [15] results indicate HCLPF (for out-of-plane response) of 0.5g PGA
- NUREG 5176 [12] documents HCLPFs (for out-of-plane response) of 0.65g PGA and 0.5g PGA for Robinson and Vermont Yankee, respectively.

3.2 Spent Fuel Pool Non-Structural Evaluation

The focus of SFP evaluations in Section 7.0 of the SPID [2] is on the elements of the SFP that might fail due to a seismic event such that a rapid draining could result. This rapid draining (or “drain-down”) is defined as failure of a pool’s structures, systems, and components (SSCs) such that there is an uncovering more than 1/3 of the spent fuel height within 72 hours. The non-structural considerations that could affect the ability of SFPs to maintain SFP inventory for 72 hours are (1) penetrations that could lead to uncovering the fuel, (2) SFP cooling functional failures that could lead to siphoning inventory from the pool, (3) sloshing losses, and (4) boil-off losses. This section also provides evaluation criteria for demonstrating that there will not be an uncovering of SFP inventory within 72 hours.

3.2.1 Background on Non-Structural Considerations

Earlier seismic risk studies have included SFP cooling and makeup systems in the analysis and have demonstrated that rapid drain-down events for SFPs are not likely. One such study, NUREG/CR-5176 [12], focused on the seismic response of a BWR and PWR SFP. For the systems analyzed in this study, SFP failure was defined as loss of water inventory leading to spent fuel rupture or degradation and possible radioactive material release. Loss of pool inventory was assumed to occur due to water boil-off following the failures of the pool cooling system and the systems that provide water makeup. The scope of the systems analysis included only those front-line systems that perform the primary functions of pool cooling and inventory makeup and the immediate systems or components that supported these systems.

These analyses showed that failure of cooling and make-up systems would not result in immediate uncovering of fuel rods. It was concluded, for the plants evaluated, that there is a response time of at least 3 days, and perhaps as much as 7 days before fuel uncovering would occur [12]. This study also demonstrated that SFP failure attributed to these systems are not directly comparable to SFP failures caused by structural degradation leading to sudden loss of water in the pool [12, Section 6.1].

Further, a 1989 NRC study (NUREG-1353) [11] found that the risk from the storage of spent fuel in the SFP at light water reactors is dominated by the beyond design basis earthquake scenario. The report concluded that the seismic capacities, or fragility, of two older SFPs indicate that the high confidence of the low probability of failure (HCLPF) is about three times

the SSE design level. The HCLPF values for SFPs were estimated to be in the 0.5g to 0.65g range.

A later study, NUREG-1738 (2001) [4], states that for 60 days after reactor shutdown for boil-off type events, there is considerable time (>100 hours) to take action to preclude a fission product release or zirconium fire before uncovering the top of fuel. Reference 4, Table 2.1, indicates the estimated time to heat up and boil-off SFP inventory down to 3 feet above top of fuel is 100 hours for PWR and 145 hours for a BWR.

More recently, a SFP Scoping study [15] was performed by NRC to continue its examination of the risks and consequences of postulated SFP accidents initiated by a low likelihood seismic event. The seismic event considered in the study was based on a central and eastern United States (CEUS) location (Peach Bottom) and an extremely rare recurrence interval (frequency of 1/60,000 years). The resulting free-field ground motion had a peak spectral acceleration of 1.8g and peak ground acceleration of 0.7g. The ground motion assumed in the NRC SFP Scoping Study envelopes all the U.S. plants with peak spectral accelerations (Sa) less than 0.8g (Figure 3-4).

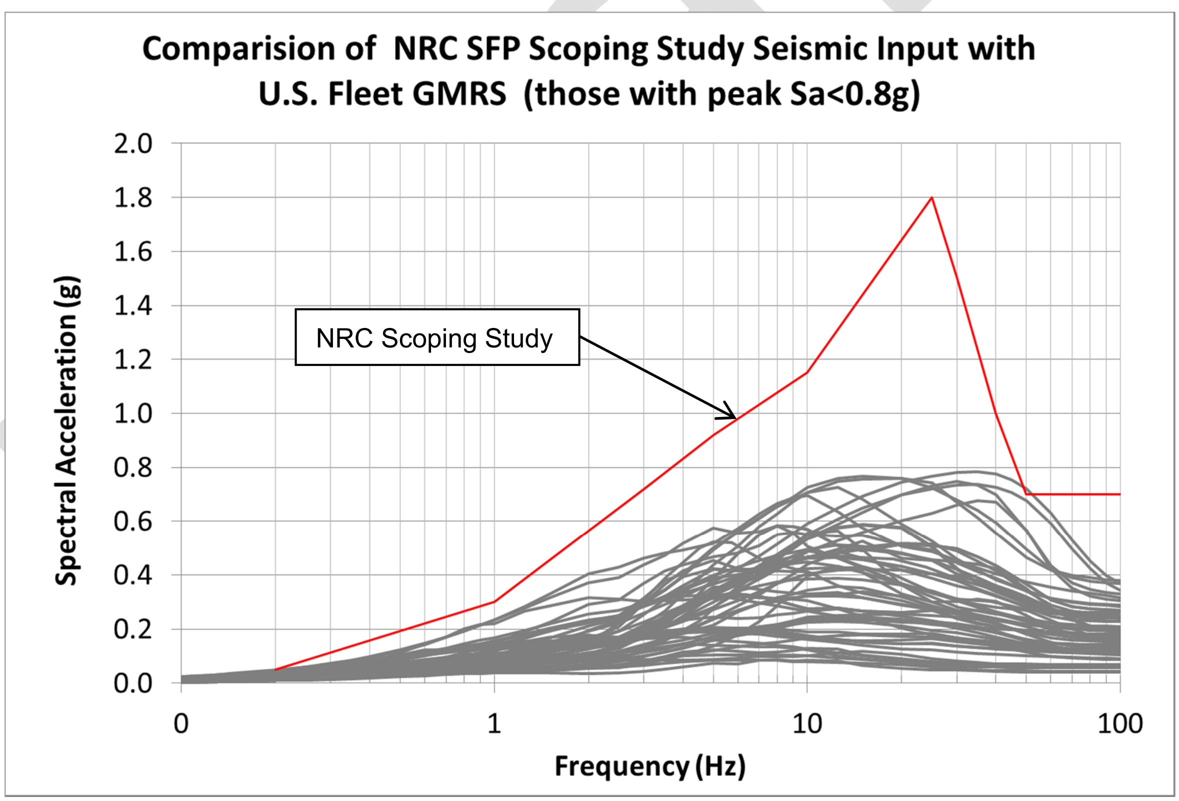

Figure 3-4
Comparison of NRC Scoping Study Seismic Input (red) with U.S. Fleet GMRS (Those with Peak Sa < 0.8g)

At the assumed seismic ground motion level, 0.7g PGA, radiological release from the reference BWR SFP was predicted to not occur within at least the first 3 days of the event [15]. Based on detailed analyses, the maximum amplitude of SFP sloshing was found to be 20 inches (51 cm). The reference SFP has no connections that would allow water to drain below the bottom elevation of the refueling gate or below 10 feet above the top of active fuel [15]. The refueling cavity gate and piping attached to the SFP were evaluated and found to be sufficiently strong and flexible enough to resist ground motion without leakage [15].

In addition to the aforementioned risk and consequence analyses, earthquake experience contributes to an understanding of how SFPs behave under extreme seismic events. The Japanese earthquake experience is relevant, as the SFP designs (in the case of BWRs) are similar to those designed in the U.S. The previously mentioned NRC Scoping Study reference plant is a BWR 4 with a Mark I containment, as was Fukushima Daiichi Units 2-5. Five Japanese nuclear power plant sites with a combined total of 20 reactors (all BWR designs) and 20 SFPs were subjected to severe ground motions from two major earthquakes in recent years [15]. In the case of the six units at Fukushima Daiichi, the measured horizontal peak ground accelerations from the 2011 Tohoku Earthquake ranged from 0.29g to 0.56g [15], which is generally higher than the U.S. fleet GMRS PGAs in Figure 3-4. For the 20 BWR SFPs, there was no reported leakage of water, other than from seismic-induced sloshing. Sloshing effects are discussed in Section 3.2.3, below.

3.2.2 Treatment Within Near Term Task Force 2.1 Seismic

Section 7 of the SPID [2] provides guidance for evaluating potential drain-down of the SFP due to a seismic event, with emphasis on those failure modes that could lead to uncovering the spent fuel within 72 hours. Guidance is provided to evaluate penetrations (both above and below top of fuel), potential for siphoning inventory, potential for sloshing, and drain-down and evaporative losses.

Consideration of SFP connections (penetrations) whose failure could result in rapid drain-down was included in the scope of the NTTF Recommendation 2.3 seismic walkdowns [5]. In response to the NTTF Recommendation 2.3, licensees performed seismic walkdowns of the SFP to verify that the current plant configuration was consistent with the design basis, verify the adequacy of current strategies, monitoring, and maintenance programs, and identify degraded, nonconforming, or unanalyzed conditions. The SFP walkdowns also addressed adverse anchorage conditions, seismic spatial interactions, and adverse seismic conditions. Any potentially degraded, non-conforming, or unanalyzed

conditions identified during the seismic walkdown program were to be assessed in accordance with the plant corrective action program.

3.2.3 Non-Structural Evaluation Criteria for Plants with GMRS Sa Less Than 0.8g

Penetrations and Piping Connections

Section 7.2.1 of the SPID [2] requires an evaluation of whether fuel could be uncovered in the event of a failure of an interconnection at a level above the fuel. This section allows for the demonstration of seismic adequacy of any connections or penetrations. Typical SFP penetrations are those associated with refueling gates and piping connections.

Refueling Gates

Spent fuel pools are typically configured with refueling gates, which are removed for refueling operations, and allow for the transfer of fuel assemblies in and out of the SFP. These removable gates have seals that are pneumatic or mechanical by design. Some plants (mostly PWRs) use inflatable seals and others (mostly BWRs) make use of permanent spring bellows (or elastomeric seals) that are not susceptible to large leak rates [8]. The refueling gate openings have narrow widths (short spans) and the gate structures themselves are comprised of stiffened steel plates.

Refueling gates have been shown to have high seismic capacities in the past due to their inherent ruggedness. Their designs have high ductility and seismic loads do not dominate the design. Rather, the design is typically dominated by hydrostatic pressure and thermal loads. As such, the designs of these gates have an inherently high seismic margin to failure and have a negligible seismic risk contribution. This is particularly true for the plants within the scope of this section where the peak Sa is less than 0.8g.

As a specific example of this ruggedness, the NRC Spent Fuel Pool Scoping Study [15] evaluated the fuel transfer gate for the referenced BWR Mark I design at Peach Bottom. The NRC analysis concluded that the refueling gate would not fail for the seismic event and will continue to maintain its intended function during the assumed high seismic event. In particular, the evaluation found that there was redundancy in the design (e.g., use of two back-to-back gates), use of polymeric seal around the perimeter that is compressed against the concrete by mechanical means which is not expected to be lost during the seismic event, and tolerances around the seals that are sufficient to accommodate the already small distortions of the SFP concrete wall. These results are representative of the high seismic capacity of SFP gates.

As previously mentioned, refueling gates have seals for ensuring water-tight integrity. As a defense-in-depth measure, many plants have design

features that help to limit the loss of pool inventory in the event of a gate failure. For most SFP designs, it is expected that a catastrophic seal failure will result in only limited water level loss due to either (1) limited volume of the adjacent cavity (e.g., refueling cavity), (2) redundant steel refueling gates, or (3) having a weir opening that has a bottom elevation that is above the top of the fuel assemblies.

During seismic events, the water in the SFP will impart increased pressures on the fuel transfer gates and seals. The water in the upper part of the pool (typically the upper 20%) will undergo convective motion (or sloshing) and the remaining water will impart impulsive pressure demands on the SFP. Guidance for estimating seismic-induced wall pressures on rectangular-shaped liquid storage tanks, which are representative of SFPs, is provided in ACI-350.3-01, "Seismic Design of Liquid-Containing Concrete Structures and Commentary" [16]. Using site-specific SFP geometry (based on survey results), reevaluated GMRS, and ACI-350.3 provisions (for pressure distribution), a comparison of the relative magnitudes of hydrostatic, impulsive, convective, and vertical impulsive pressures was made for current U.S. plants with peak Sa less than 0.8g. The results indicate that at the mid-height of the pool (approximately the bottom elevation of the refueling gate), the median convective pressures are small compared to median hydrostatic, impulsive, and vertical pressures (Table 3-3). When combined using the square-root-sum-of-the-squares methodology, (SRSS), the median seismic pressures for both BWR and PWR SFPs are generally less than the hydrostatic pressure (at pool mid-height). This finding supports the earlier statement that SFP refueling gate designs are dominated by hydrostatic pressure rather than seismic-induced pressures.

For SFP gate designs that make use of pneumatic seals (mostly PWR designs [8]), seal pressures are typically greater than the seismic-induced pressures, and will therefore not be damaging. For example, in a few observed PWR applications, the refueling gate seal pressures were found to be 30 psi, which is significantly higher than the maximum SRSS pressure of 8.1 psi in Table 3-3.

Table 3-3
Comparison of Spent Fuel Pool Wall Pressures at Mid-Height (Median Values for all U.S. Spent Fuel Pools with Peak Sa < 0.8g)

	Hydrostatic Pressure (psi)	Horizontal Impulsive Pressure (psi)	Horizontal Convective Pressure (psi)	Vertical Pressure (psi)	Combined Pressure SRSS(psi)
PWR SFPs	8.9	4.0	0.3	4.5	5.9
BWR SFPs	8.5	5.3	0.3	6.2	8.1

On the basis that (1) refueling gates (including seals) are inherently rugged components typically comprised of stiffened steel plates and (2) the observation that refueling gate design is dominated by hydrostatic demands rather than seismic, it is judged that typical SFP refueling gates will remain functional in cases where the GMRS peak Sa is less than 0.8g.

Piping

Piping connections to the SFP are required for the SFP cooling system discharge and suction lines. Most piping penetrations are well above the elevation of the fuel assemblies in the SFP. However, there are limited cases where SFPs have piping connections below about 10 feet above the top of the fuel assemblies. An industry survey of the U.S. plants screened-in to perform SFP confirmations, substantiated this assumption. The SFP cooling systems were included in the scope of the NTTF 2.3 seismic walkdowns, where seismic interactions, corrosion, and degraded conditions were assessed [5].

SFP cooling and makeup piping systems are considered to be seismically rugged. Seismically designed (or seismically evaluated) piping is inherently rugged and has been shown in past seismic margin and seismic risk studies not to contribute appreciably to the seismic risk. Past SFP risk assessments concluded that HCLPF capacities of piping systems are estimated to be in excess of 0.5g PGA [12], which exceeds the GMRS PGAs for each of the plants within the scope of this section (Figure 3-4). As a check at higher seismic acceleration levels, the NRC Scoping Study [15] evaluated piping connections for the SFP at Peach Bottom (peak Sa of 1.8g) and found that due to the very small resulting displacements / distortions, the piping would remain functional and leak tight.

On the basis that (1) SFP piping systems are typically designed for seismic loading and considered to be rugged [12], (2) the 2.3 seismic walkdowns included SFP piping systems, and (3) the detailed NRC SFP Scoping Study analysis, which found small relative displacements for an elevated SFP, there is high confidence that the SFP piping and penetrations will remain functional for plants with GMRS peak Sa less than 0.8g.

Siphoning of Spent Fuel Pool Inventory

Section 7.2.3 of the SPID [2] provides guidance for assessing SFP cooling functional failures that could lead to siphoning inventory from the pool. As SFP suction lines are typically connected near the top of the pool, these lines are not susceptible to siphoning significant amounts of inventory in an event leading to siphoning. However, SFP discharge lines can extend to near the bottom elevation of the SFP. These lines typically have anti-siphoning devices (holes or valves) that prevent drain-down of the SFP. Anti-siphoning valves are typically passive mechanical devices that permit flow in one direction.

EPRI NP-6041 [10], Table 2-4, identifies that passive valves are assumed to be rugged for peak spectral accelerations less than 0.8g. The NP-6041 criteria also require the evaluation of extremely large extended operators on valves attached to 2 inch (5.1 cm) or smaller piping. The NP-6041 screening criteria are applicable to the SFP cooling system.

Therefore, siphoning of SFP inventory is not a significant risk provided anti-siphoning devices exist in applicable piping systems and any extremely large extended operators on valves attached to 2 inch (5.1 cm) or smaller piping are evaluated.

Sloshing

Horizontal seismic demands on the SFP can induce vertical fluid motion, or sloshing. SFP sloshing is addressed in SPID Section 7.3.2 [2]. This section provides guidance for estimating the fundamental sloshing frequencies (one in each direction of the pool) and for estimating the slosh height. Industry SFP survey results of all of the U.S. NPP fleet screened in to perform SFP evaluations confirm that sloshing frequencies are in the low frequency range (< 0.5 Hz). For plants with peak Sa less than 0.8g, it is observed that most GMRS are enveloped by the SSE in the low frequency range. In the few cases where site-specific GMRS exceeds the SSE in the low frequency range, these exceedances have low spectral amplitude (< 0.1g) and are generally no more than 20 percent above the SSE. The sloshing heights, resulting from these exceedances, are estimated to be several feet, not accounting for pool free-board. Using the conservative SPID [2] equations, the median slosh height is calculated to be 3.7 feet (1.1 m). Assuming a minimum free-board of 1 foot (0.3 m), this corresponds to a conservative estimate of 2.7 feet (0.8 m) of SFP water inventory lost due to sloshing. This loss of inventory is judged to not be significant given the typical SFP depth of 40 feet (12.2 m).

SPID Section 7.3.2 [2] acknowledges the conservatism in the slosh height analysis. As the GMRS of most U.S. plants are enveloped by the SSE in the low frequency range, the level of conservatism in the SPID equations is not a significant consideration. However, to estimate the approximate degree of conservatism, sloshing results were calculated (using the SPID equations) for the plant in the NRC Spent Fuel Pool Scoping Study [15]. While the SPID methodology yields sloshing frequencies that are comparable to those calculated in the detailed Scoping Study, the SPID-based sloshing heights are well above the Scoping Study sloshing heights. For the reference SFP, the SPID methodology predicts a sloshing height of 10 feet (3.0 m). However, the NRC Scoping Study, which used a more refined sloshing analysis based on finite element modeling, predicted a maximum sloshing height of approximately 2 feet (0.6 m). In this case, the conservative SPID equation yields results that are approximately five times higher than the detailed analysis.

Another comparison case is the Fukushima-Daiichi Unit 2 SFP, which was subjected to severe seismic ground motion during the March 11, 2011 Tohoku earthquake. The Unit 2 reactor is a BWR 4 Mark I with similar characteristics as the NRC Scoping Study reference plant (BWR 4 Mark I). The Unit 2 SFP has dimensions of 40 feet by 32.4 feet (12.2 m by 9.9 m) and a depth of 38.7 feet (11.8 m) and has fundamental sloshing frequencies of 0.25 Hz and 0.28 Hz (each horizontal direction). The ground motion in this frequency (0.2-0.3 Hz) is comparable to the NRC Scoping Study (peak Sa of 0.1g) (Figure 3-5). Using the Fukushima-Daiichi Unit 2 parameters, the SPID [2] equation predicts a sloshing height of 10 feet (3.0 m); however, the estimates of actual sloshing amplitudes for Fukushima-Daiichi Unit 2 were approximately 2.6 feet (0.8 m) [15]. In the case of Fukushima-Daiichi Unit 2, the SPID equation estimates sloshing heights that are approximately 3.8 times higher than those observed. Despite the apparent conservatism in the SPID methodology, this method is used in this report for estimating SFP sloshing losses.

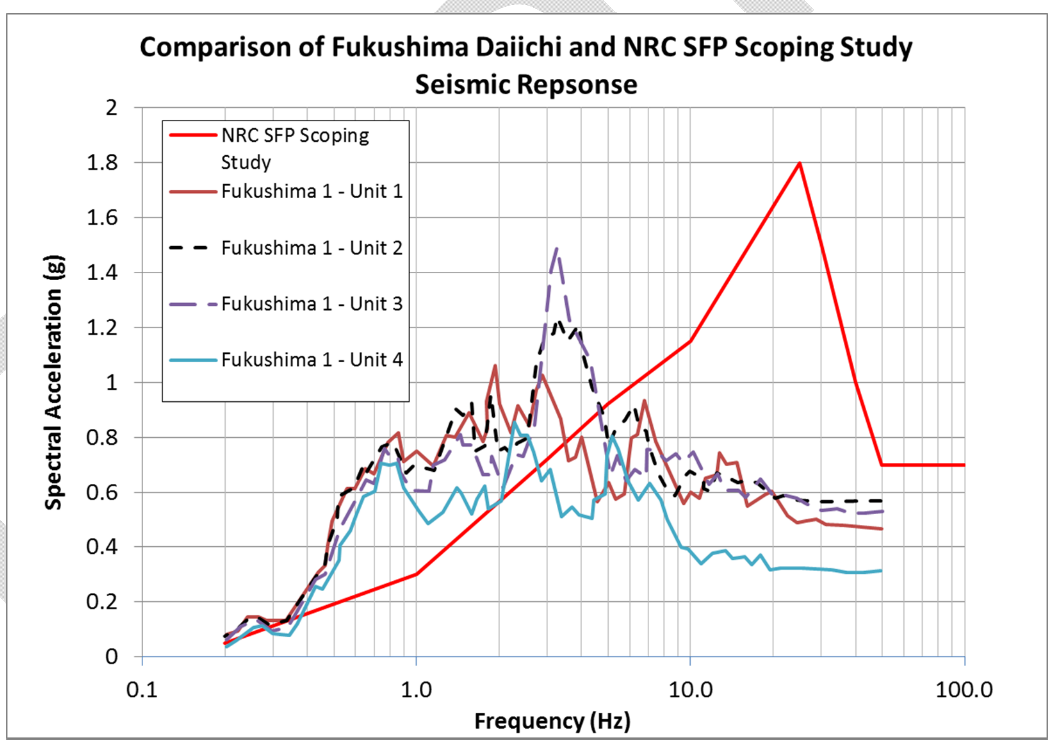

Figure 3-5
Horizontal Response Spectra (5% Damping): Fukushima Daiichi Units 1-4 (Foundation) and NRC Spent Fuel Pool Study (Free-Field) (Source: NRC Spent Fuel Pool Scoping Study)

Estimates of sloshing for plants with peak spectral accelerations less than 0.8g were computed using the industry SFP survey results, site-specific GMRS demands, and SPID [2] sloshing equations. These results, shown

in Figure 3-6, indicate the distribution of sloshing losses and remaining water inventory. Despite losses due to sloshing, the remaining water inventory above the top elevation of fuel assemblies ranged from 16.8 ft to 29.2 ft (mean = 22.4 ft) for the plants analyzed. Accounting for allowed inventory losses down to 1/3rd the height of the fuel assemblies in accordance with SPID [2], the increased inventory height ranges from 21.8 ft to 34.2 ft (mean = 27.4 ft). The remaining water inventory after accounting for sloshing losses is used as the initial condition for boil-off losses which are discussed in the next section of this report.

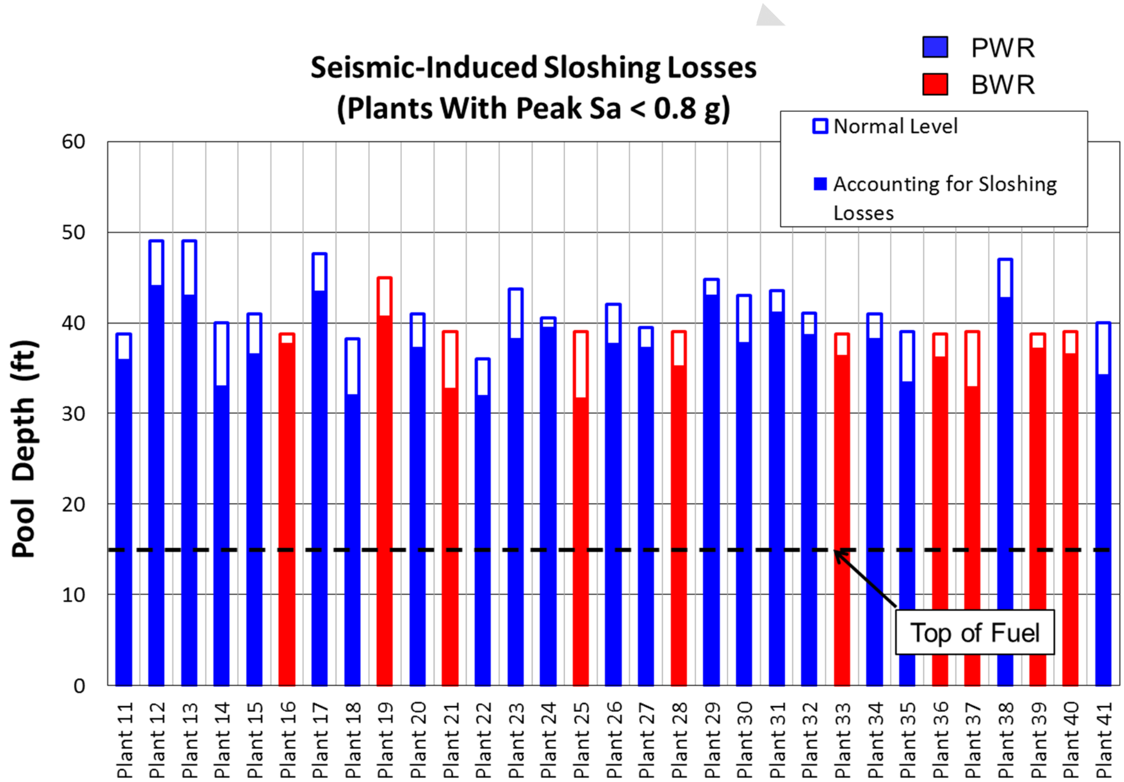

Figure 3-6
Seismic-Induced Sloshing Losses and Remaining Spent Fuel Pool Inventory for Plants with Peak Sa < 0.8g

Boil-off Losses

The SPID [2] criteria includes an assessment of SFP boil-off inventory losses using equations in Appendix EE of EPRI 1025295, "Update of the Technical Bases for Severe Accident Management Guidance" [17]. These equations can be used to determine the length of time necessary to uncover more than 1/3rd of the height of the spent fuel.

Site-specific estimates of heat up and drain down times for plants with peak spectral accelerations less than 0.8g were computed using the industry SFP survey results, site-specific sloshing results, and Appendix EE of [17].

The SFP heat loads assumed in the analysis were based on realistic values obtained from several plants (Figure 3-7). A distribution of BWR and PWR units, ranging in core thermal power from 1,800 MW_t to 4,000 MW_t, were surveyed for representative SFP heat loads and outage durations. Results indicated (1) a strong correlation of SFP heat load with core thermal power, (2) plants only retain 30-40% of the core in the SFP after the outage, and (3) the typical outage periods last from 20-30 days. This data was used to estimate site-specific SFP heat loads corresponding to 20 days following shutdown, which corresponds to the shortest estimated outage duration. Additional information about the process used to estimate the heat loads is provided in Appendix B.

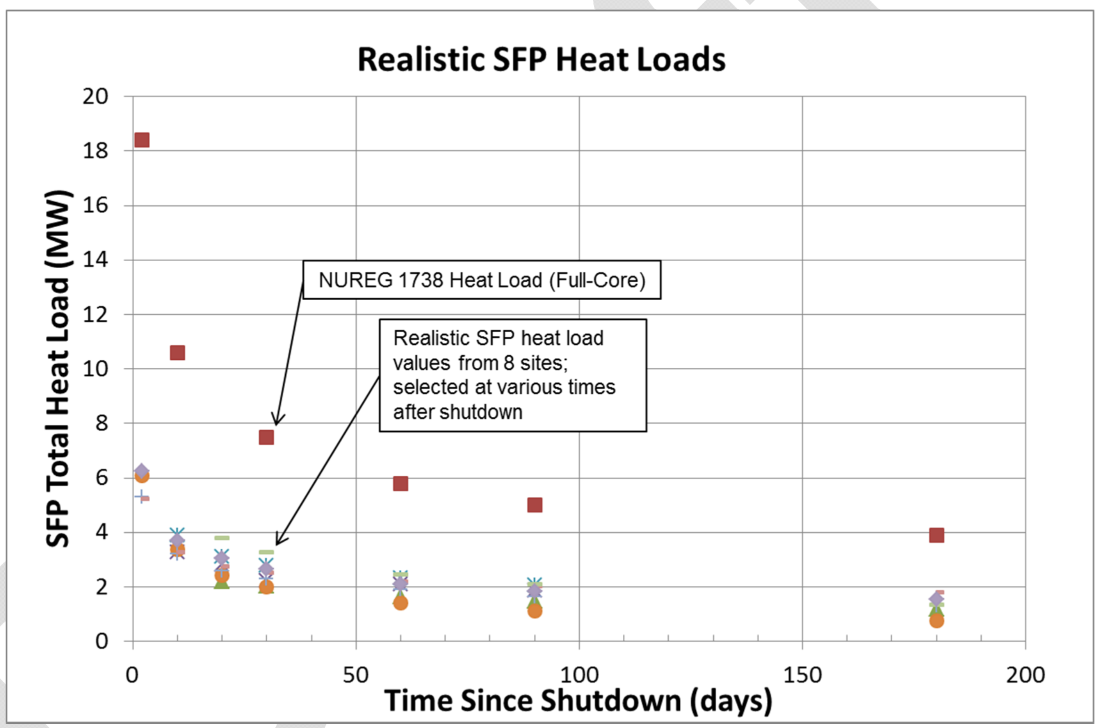

Figure 3-7
Comparison of Realistic and Full-Core Spent Fuel Pool Heat Loads

The boil-off analysis results indicate that all plants have significantly more than 72 hours before uncovering fuel. These results, shown in Figure 3-8, indicate the distribution of drain-down times to uncover the upper $1/3^{rd}$ height of the fuel assemblies.

Site-specific results indicate that plants have a minimum of 149 hours and a maximum of 711 hours (mean=279 hours) before drain-down to the upper $1/3^{rd}$ fuel assembly height (Figure 3-8).

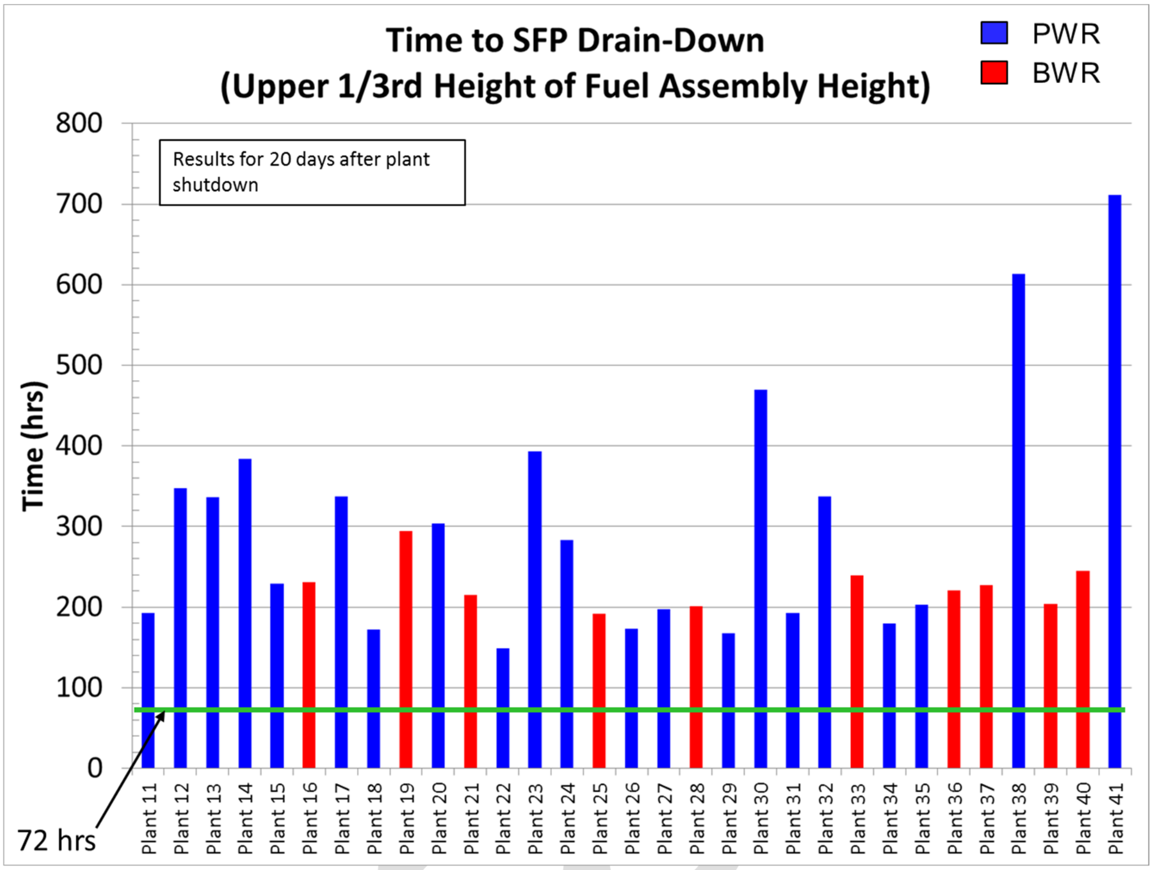

Figure 3-8
Spent Fuel Pool Drain-Down Results for Plants with Peak Sa < 0.8g

3.2.4 Summary of Non-Structural Evaluation Criteria

Table 3-4 provides a summary of the SFP non-structural evaluation criteria derived in Section 3.2

Table 3-4
Spent Fuel Pool Non-Structural Drain-Down Criteria

Potential Rapid Drain-Down Mechanism	Evaluation	Applicability Criteria
Scope of Evaluation	Evaluations in Section 3.2 are applicable to plants with GMRS peak Sa < 0.8g.	• Site-specific GMRS has a peak Sa < 0.8g.
Piping Connections	Past risk evaluations have found SFP piping, evaluated to SSE demands, to be rugged.	• Attached piping up to the first valve designed (or evaluated) to the SSE.
Fuel Transfer Gate	Gates and seals have rugged designs with adequate capacity for GMRS with peak Sa < 0.8g.	• No additional criteria.
Siphoning	Anti-siphoning devices are rugged and not a significant contributor to rapid drain-down. In accordance with NP-6041 Table 2-4 [10], in cases where active anti-siphoning valves are used, confirmation that extremely large extended operators (attached to 2-inch or smaller piping) be walked down to confirm lateral support.	• Anti-siphoning devices exist in applicable piping systems • In cases where active anti-siphoning devices are attached to 2-inch or smaller piping and have extremely large extended operators, the valves should be walked down to confirm adequate lateral support.
Sloshing	Site-specific sloshing analyses show that inventory losses are minor. For plants with peak Sa less than 0.8g, a conservative estimate of SFP inventory lost to sloshing is 3 feet. This lost inventory is accounted for in estimating evaporative losses (below).	• Maximum pool dimension (length or width) is less than 125 ft. • SFP pool depth greater than 36 ft. • GMRS peak Sa < 0.1g at 0.3 Hz.
Evaporative Losses	Estimated time to heat up and boil-off and uncover more than 1/3 of the SFP fuel assemblies is more than 72 hours.	• SFP surface area greater than 500 ft^2. • Licensed core thermal power less than 4,000 MWt / unit.

3.3 Site-Specific Spent Fuel Pool Criteria for Low GMRS Sites

Sections 3.1 and 3.2 provide evaluation criteria that can be applied at sites where the GMRS peak spectral acceleration (Sa) is less than 0.8g. The following parameters should be verified on a site-specific basis to confirm that the evaluation criteria applies to the site.

Site Parameters:

1. The site-specific GMRS peak spectral acceleration at any frequency should be less than or equal to 0.8g.

Structural Parameters:

2. The structure housing the SFP should be designed using an SSE with a peak ground acceleration (PGA) of at least 0.1g.
3. The structural load path to the SFP should consist of some combination of reinforced concrete shear wall elements, reinforced concrete frame elements, post-tensioned concrete elements and/or structural steel frame elements.
4. The SFP structure should be included in the Civil Inspection Program performed in accordance with Maintenance Rule [14].

<u>Non-Structural Parameters:</u> (the criteria below assumes the site and structural criteria above (items 1-4) are satisfied)

5. To confirm applicability of the piping evaluation in Section 3.2, piping attached to the SFP up to the first valve should have been evaluated for the SSE.
6. Anti-siphoning devices should be installed on any piping that could lead to siphoning water from the SFP. In addition, for any cases where active anti-siphoning devices are attached to 2-inch or smaller piping and have extremely large extended operators, the valves should be walked down to confirm adequate lateral support.
7. To confirm applicability of the sloshing evaluation in Section 3.2, the maximum SFP horizontal dimension (length or width) should be less than 125 ft, the SFP depth should be greater than 36 ft, and the GMRS peak Sa should be <0.1g at frequencies equal to or less than 0.3 Hz.
8. To confirm applicability of the evaporation loss evaluation in Section 3.2, the SFP surface area should be greater than 500 ft^2 and the licensed reactor core thermal power should be less than 4,000 MWt per unit.

Section 4: Seismic Review of Spent Fuel Pools at High GMRS Sites

This report section addresses spent fuel pools at sites where the GMRS peak spectral acceleration is greater than 0.8g. Spent fuel pool evaluation criteria are provided in this section to address structural elements (e.g., floors, walls), non-structural elements (e.g., penetrations), seismic-induced sloshing, and losses due to heat-up and boil-off. These criteria help to provide assurance that the SFP will maintain adequate water inventory for 72 hours following a beyond design basis seismic event.

4.1 Structural Evaluation

Section 3.1.2 of this report describes spent fuel pool structures as robust and that they have relatively high HCLPF values. Previous SFP studies, such as the NUREG 5176 [12] and the NRC SFP Scoping Study [15] identified that out-of-plane shear and flexure response of floors and walls were the governing structural failure modes. These evaluations have also found that SFP walls and floors have relatively high stiffness with natural frequencies in the 10 to 20 Hz range. As a result, these structures have relatively small seismic-induced displacements (i.e., within design code deflection limits [24]). The evaluation of SFP structures, for the purpose of ensuring integrity under GMRS demands makes use of these insights.

The approach for evaluating structural responses of SFP wall and floor panels makes use of a detailed single degree-of-freedom model that is generic and can be applied to a wide-range of SFP designs. The model makes use of commonly referenced materials, including codes and standards typical of those used in seismic structural design and analysis. Appendix C to this report provides a detailed example of the use of this model and also provides a comparison of results with the more detailed SFP analysis described in the NRC SFP Scoping Study [15]. The results of the approach described in Appendix C compared well with the NRC Study and demonstrated that the approach is reasonable for estimating the HCLPF of typical SFP wall panels and floor slabs.

The method of analysis of the SFP structure is based on the Conservative Deterministic Failure Margin Approach (CDFM). The CDFM approach for developing fragilities is a simpler method that can be performed

consistently by more analysts and is an acceptable approach for evaluating SFP structural integrity.

The procedure for the SFP structural evaluation is described below:

4.1.1 Estimate Ultimate Strength of SFP Floors and Walls

The ultimate strength of SFP floor and wall panels can be developed using the SFP floor slab and wall panel geometry, design material properties, and reinforcement details. The method for estimating ultimate strength in Appendix C is based on yield line theory; typical of what is used to estimate design of reinforced concrete slabs [21, 22] and what was used in the two previous SFP fragility analyses documented in NUREG 1738 [4]. The material properties should be consistent with those assumed for design. Nominal design values for concrete compressive strength and rebar yield strength are assumed in the analysis. In addition, the effective stiffness of the SFP floor and wall systems are reduced 50 percent to account for concrete cracking in accordance with ASCE 43-05 [20].

Appendix Sections C3.0 and C4.0 provide equations for the ultimate strength evaluation of representative SFP floor panels (i.e., slabs) and wall panels. While the equations in Appendix C relate to rectangular panels, the method can be applied to panels with non-uniform rectangular shapes (e.g., bump-outs, etc.).

Industry SFP survey results indicate that SFP wall and floor panels are not highly reinforced, and typically have reinforcement ratios in the range of 0.001 to 0.003 (Figures A-10 and A-12). While reinforcement ratio can vary spatially within a panel, the evaluation for panel strength should be based on the average positive reinforcement ratio at mid-span and average negative reinforcement ratio at supports. Industry survey results indicate that shear reinforcement is not typical in floor slabs and wall panels, but can be credited if used in the design. The ultimate strength of wall and floor panels should be based on the factored ACI code strength and should be based on the nominal flexure and shear section capacities.

The SFP wall and floor panels are evaluated separately. Survey results, which included wall spans, wall thickness, material strengths, and amount of steel reinforcement, indicate that the wall panels with the longest spans are typically controlling. In some cases, SFP walls are supported by intermediate floors that decrease the unsupported span lengths and therefore increase out-of-plane wall stiffness. If these intermediate floors are sufficiently rigid (e.g., 18"-24" thick), the clear span of the SFP wall can be assumed to be the height difference between these floors. A similar approach can be used if a thick SFP wall intersects with another SFP wall. This is a common arrangement in cask pit areas and refueling canals. Another instance is the case where an SFP wall will have variable thickness (lower portion of wall thicker than the top portion). In this case, an average wall thickness can be assumed.

Industry SFP survey results indicate that there are cases where a refueling gate weir opening is located in one or more of the SFP walls. These weir openings typically have narrow widths (3 to 4 ft) and extend down approximately to the elevation of the top of fuel assemblies. The walls with weirs typically have increased thickness and reinforcement to maintain adequate strength and stiffness. In BWR Mark 4 cases, this weir opening is located in the thick reactor shield wall and does not significantly affect out-of-pane stiffness. The detailed BWR Mark 4 SFP model developed for the NRC SFP Scoping Study [15] included a weir opening in the shield wall. The analysis results indicate that the out-of-plane displacements of this particular wall under high-seismic demands were small compared to adjacent walls without weir openings.

In PWR applications, these weir openings are typically in walls with shorter spans (cask pits, and refueling canals), whose stiffness is not significantly affected. In addition, these walls are not usually critical because they are not exterior walls of the SFP and inventory losses to cask pits and refueling canals can be mitigated. Based on these observations, SFP walls with weir openings are not risk-significant and can be excluded from the structural evaluation.

Boundary conditions for wall and floor panels are assumed on a case-by-case basis. For wall panels, it is reasonable to assume rigid fixity at wall-to-wall and wall-to-floor interfaces. For the top wall support, it is reasonable to assume free or pinned depending on design. The assumption of a pinned boundary condition is reasonable if a floor slab supports the top of the wall.

For the evaluation of floor slabs, it can be assumed that the slab is sufficiently rigid if the slab bears directly on grade (rock or soil). In this case the slab does not have to be evaluated and the effort can focus on the limiting wall panel.

For estimating the floor slab boundary conditions, it is reasonable to assume the floors are rigidly fixed (against rotation) at all supports. In the case of a column-supported floor slab, the yield line approach is still applicable, however, it should be confirmed that the punching shear (two-way) is not a limiting failure mode. If so, floor slab strength should be based on this failure mode using the provisions in ACI 349 [24] for punching shear strength of reinforced concrete slabs.

Estimate Panel Stiffness

Appendix Sections C3.6 and C4.6 provide equations for estimating panel stiffness. The elastic and elastic-plastic flexural stiffness are calculated using the panel properties and assumed boundary conditions. These stiffness values are combined to develop an equivalent stiffness as shown in Figure C-9. Estimating panel stiffness is common in structural engineering practice and references exist for estimating stiffness of panels

with various span aspect ratios and boundary conditions. References such as Biggs [21] can be used for this purpose, but other approaches are acceptable.

Estimate Flexural Panel Frequency

Appendix Sections C3.7 and C4.7 provide equations for estimating panel frequencies. The fundamental flexural frequency for the wall and floor panels is calculated using the panel stiffness, panel mass, and effective water mass. The mass acting on the SFP floor slabs will also need to account for the masses of fuel assemblies and fuel racks. Although mass distribution of assemblies and racks will vary within the pool, average values (total mass/total area) are recommended. For floor slabs, all of the SFP water mass should be combined with the mass of floor slab, assemblies, and racks. In the case of wall panels, 50 percent of the SFP water inventory mass should be combined with the mass of the wall panel.

Estimate Seismic Demands

The following seismic demands are needed for the SFP evaluation in the form of in-structure response spectra (ISRS) or floor response spectra (FRS):

- Horizontal ISRS X-direction[1] – mid-height of SFP wall. Top and Bottom ISRS can be averaged. Used to estimate horizontal inertial and hydrodynamic loads in the X-dir.
- Horizontal ISRS Y-direction[1] – mid-height of SFP wall. Top and Bottom responses can be averaged. Used to estimate horizontal inertial and hydrodynamic loads in the Y-dir.
- Vertical ISRS[1] – elevation of SFP floor support. Used to estimate vertical inertial and hydrodynamic loads in the vertical.
- Horizontal GMRS (0.5% damping) – Used to calculate sloshing amplitudes.

These seismic demands are typically derived from seismic models that have been evaluated for site-specific GMRS demands (e.g., SPRA building models). The GMRS-based ISRS can be derived from simple lumped mass or more detailed FE models as long as the structural modes up to 20 Hz are captured and the input is based on site-specific GMRS demands. Many plants already have SFPs incorporated into their detailed building models for SPRA applications. The ISRS (or FRS) derived from these models are used to estimate inertial and hydrodynamic forces acting on the SFP structure.

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[1] The default damping is 5%. Alternative values (e.g. 7%) may be justified on a case-by case-basis.

For existing SFP seismic models based on detailed finite element methods, FRS at the six locations (opposite-adjacent corners and wall centers) can be used (Figure 4-1). For lumped-mass models, FRS at the top and bottom of the SFP can be used.

The seismic input in the horizontal direction is assumed to be the average response from horizontal FRS at the top and bottom of the SFP. If FRS from several SFP locations on the top or bottom elevations are available, the average of the FRS should be used.

The seismic input in the vertical direction is assumed to be the response from the vertical FRS at the bottom of the SFP. If several FRS at the SFP bottom are available, the vertical demand should be based on the average of these FRS. The two horizontal and vertical FRS are then used to estimate seismic inertial and hydrodynamic loads on the SFP floors and walls.

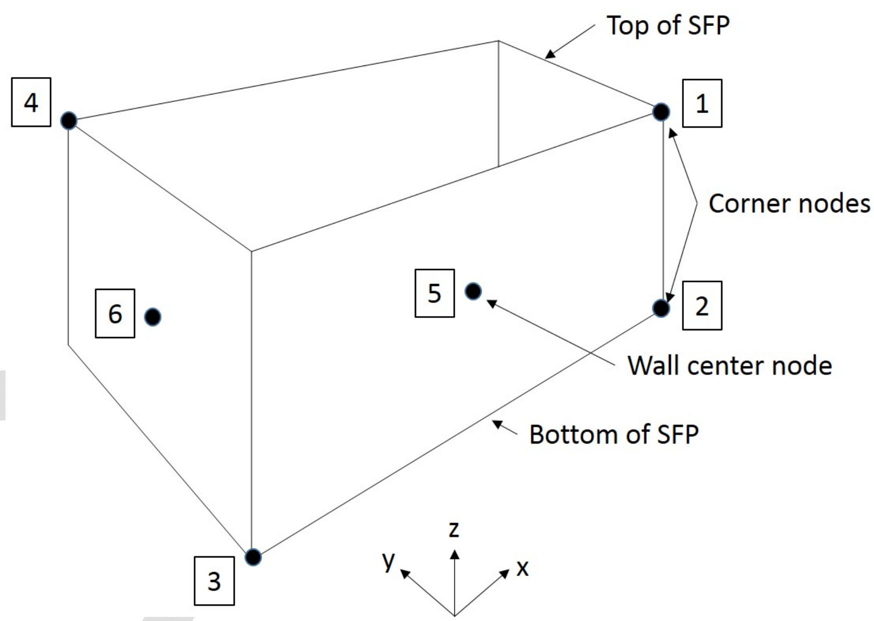

Figure 4-1
Example SFP finite element model indicating the six desired locations of nodal output

Estimate Inertial and Hydrodynamic Forces

Inertial and hydrodynamic forces are developed for each orthogonal pool direction using the equations in Appendix Section C3.8 and 4.8. SFP floor slab demands are calculated using the vertical FRS at the elevation of the floor. Wall panel demands are calculated using the average of FRS at the top and bottom pool elevations. Horizontal hydrodynamic pressures are computed for each direction using the method in ACI 350.3/350.3R-15 [16], which provides guidance for the seismic analysis and design of liquid-containing concrete structures.

The steps in the analysis of hydrodynamic pressures are to calculate inertial and convective masses, estimate heights to center of mass, and calculate the resultant forces and corresponding pressure distributions. Sloshing frequencies, used in the estimation of convective wall pressures are estimated using on Section 7.3.2 of the SPID [2].

Load Combination

The evaluation for inertial and hydrostatic demands is performed separately for each direction (X, Y, Z) using the equations in Appendix Sections C3.8 and C4.8. The combined seismic demand on a SFP floor slab is based on 100-40-40 method:

$$E_{floor} = 1.0 \text{ x } E_{vert} + 0.4 \text{ x } E_{horizontal}$$

Similarly, the combined seismic demand on a SFP wall with the highest hydrodynamic pressure is based on the following load combination:

$$E_{wall} = 1.0 \text{ x } E_{horizontal} + 0.4 \text{ x } E_{vert}$$

Compute HCLPF

The HCLPF's for floors and walls are calculated in consideration of seismic and dead loads using the equations in Appendix Sections C3.10 and C4.10. If shear strength controls the design of the floor or wall panel, and no shear reinforcement is present, ductility effects should not be credited in the HCLPF calculation. However, if flexure controls and shear capacity is at least a factor of 1.20 times the flexure strength, an appropriate flexural failure mode inelastic factor can be estimated and credited in the HCLPF calculation.

4.1.2 GMRS and HCLPF Comparison

If the site-specific GMRS is less than the limiting SFP HCLPF in the frequency range-of-interest (e.g., 10-20 Hz), then the SFP structure will have an acceptably low rate of failure under seismic conditions (<1% mean probability of failure). However, if the GMRS is greater than the HCLPF in the frequency range-of-interest, then a few options can be considered:

- Perform a more refined fragility analysis of the SFP using the separation-of-variables methodology. EPRI TR-103959 [19] provides guidance on this approach.
- Perform a more detailed dynamic time-history analysis of the SFP structure and coupled fluid inventory. Seismic analysis should be performed in accordance with ASCE 4-98 [18] to demonstrate adequacy of the wall and floor panels.

4.2 Non-Structural Evaluation

The focus of SFP evaluations in Section 7.0 of the SPID [2] is on the elements of the SFP that might fail due to a seismic event such that a rapid draining could result. This rapid draining (or “drain-down”) is defined as failure of a pool’s structures, systems, and components (SSCs) such that there is an uncovering more than 1/3 of the spent fuel height within 72 hours. The non-structural considerations that could affect the ability of SFPs to maintain SFP inventory for 72 hours are (1) penetrations that could lead to uncovering the fuel, (2) SFP cooling functional failures that could lead to siphoning inventory from the pool, (3) sloshing losses, and (4) boil-off losses.

These considerations, as they relate to plants with peak GMRS Sa greater than 0.8g, are discussed in the following sections.

Penetrations and Piping Connections

Section 7.2.1 of the SPID [2] requires an evaluation of whether fuel could be uncovered in the event of a failure of an interconnection at a level above the fuel. This section allows for the demonstration of seismic adequacy of any connections or penetrations. Typical SFP penetrations are those associated with refueling gates and piping connections.

Gates

Section 3.2.3 of this report notes that SFPs are typically configured with refueling gates, which are removed for refueling operations, and allow for the transfer of fuel assemblies in and out of the SFP. These removable gates have seals that are pneumatic or mechanical by design. Some plants (mostly PWRs) use inflatable seals and others (mostly BWRs) make use of permanent spring bellows (or elastomeric seals) that are not susceptible to large leak rates [8]. The refueling gate openings have narrow widths (short spans) and the gate structures themselves are comprised of stiffened steel or aluminum plates.

Section 3.2 also describes that refueling gates have been shown to have high seismic capacities in the past due to their inherent ruggedness. Their designs have high ductility and seismic loads do not dominate the design. Rather, the design is typically dominated by hydrostatic pressure and thermal loads.

Refueling gate designs were reviewed for plants with higher seismic ground motions (peak GMRS Sa > 0.8g) to confirm seismic ruggedness. The review found that most of the gates had similar design details, such as stiffened plates, aluminum or steel material, and welded joints. Design parameters such as gate span, plate material, and thickness were considered in the review.

Based on the use of stiffened plates, welded connections, and ductile materials, the gate designs were judged to be sufficiently rugged to resist higher seismic demands.

From the above observations, it is judged that that refueling gate designs have an inherently high seismic margin to failure and have a negligible seismic risk contribution.

Piping

From reviews of the SFP designs at sites with GMRS peak Sa > 0.8g, the attached piping is either seismically designed (or evaluated) to the SSE or penetrates the SFP relatively high in the SFP wall (within about 6 ft of the top of the pool). Seismically designed (or seismically evaluated) piping is inherently rugged and has been shown in past SPRAs and margin studies not to contribute appreciably to the seismic risk.

Since the SFP piping has all been reported to be attached to the pool at relatively high elevation within the pool, rather than demonstrate that all of the SFP piping is seismically designed, it is simpler to assume that SFP inventory is lost down to the level of the piping penetration. These losses will be accounted for in the total inventory loss calculations.

Siphoning of SFP inventory

The approach for evaluating siphoning of SFP inventory is the same as that described in Section 3.2.3 of this report. Siphoning of SFP inventory is not a significant risk provided anti-siphoning devices exist in applicable piping systems and any extremely large extended operators on valves attached to 2 inch or smaller piping are evaluated.

Sloshing

The approach for evaluating SFP sloshing under seismic events is the same as that described in Section 3.2.3 of this report. The industry SFP survey results, estimated site-specific GMRS demands, and conservative SPID sloshing equations, were used to develop plant-specific estimates of sloshing for plants with peak spectral accelerations greater than 0.8g. These results, shown in Figure 4-2, indicate the distribution of sloshing losses and remaining water inventory. Despite losses due to sloshing, the remaining water inventory above the top elevation of fuel assemblies ranged from 6.4 ft to 21.5 ft (mean = 15.9 ft) for the plants analyzed. Accounting for allowed inventory losses down 1/3rd the height (5 ft) of the fuel assemblies, in accordance with SPID, the increased inventory height ranges from 11.4 ft to 26.5 ft (mean = 20.9 ft). The remaining water inventory after accounting for sloshing losses is used as the initial condition for evaporative losses which are discussed in the next section of this report.

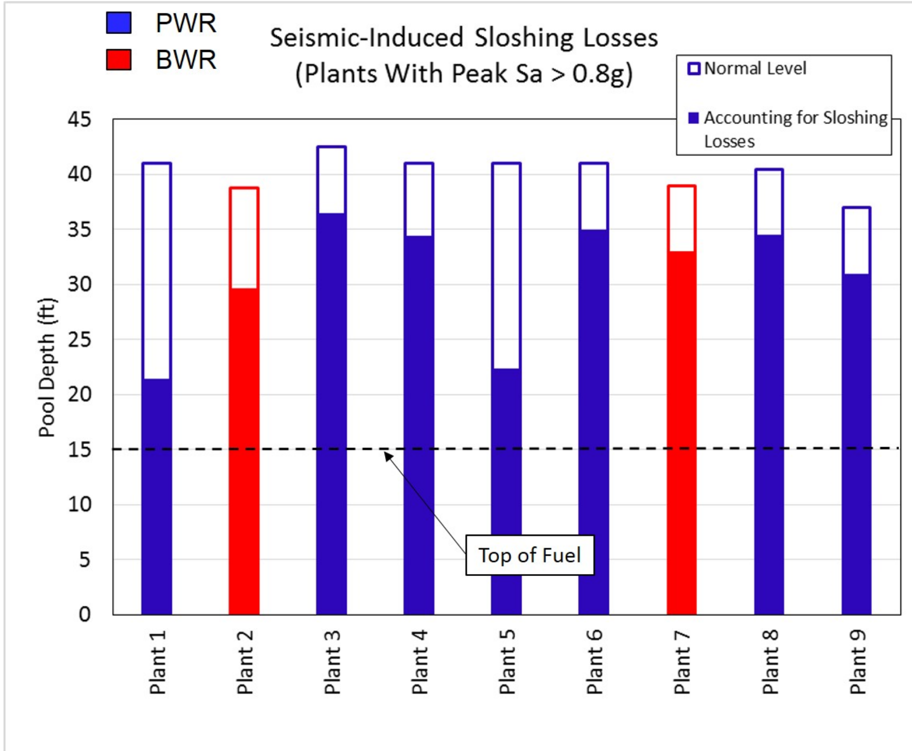

Figure 4-2
Seismic-induced sloshing losses and remaining SFP inventory for plants with peak spectral accelerations greater than 0.8g

Evaporative Losses

The approach for evaluating SFP evaporative and boil-off losses under seismic events is the same as that described in Section 3.2.3 of this report. The available SFP inventory at the start of the evaporative loss calculations accounts for the worst case seismic induced losses down to the piping penetration (take as 6 ft) of estimated sloshing losses if they are more than 6 ft. The drain-down analysis results for plants with peak Sa greater than 0.8g indicate that these plants have significantly more than 72 hours before uncovering fuel to the upper 1/3rd height of the fuel assemblies. These results, shown in Figure 4-3, indicate the distribution of drain down times to the upper 1/3rd height of the fuel assemblies.

Plant-specific results, based on conservative sloshing equations (Section 3.2.3), indicate that plants have a minimum of 94 hours and a maximum of 392 hours (mean= 241 hours) before drain-down to the upper 1/3rd fuel assembly height (Figure 4-3).

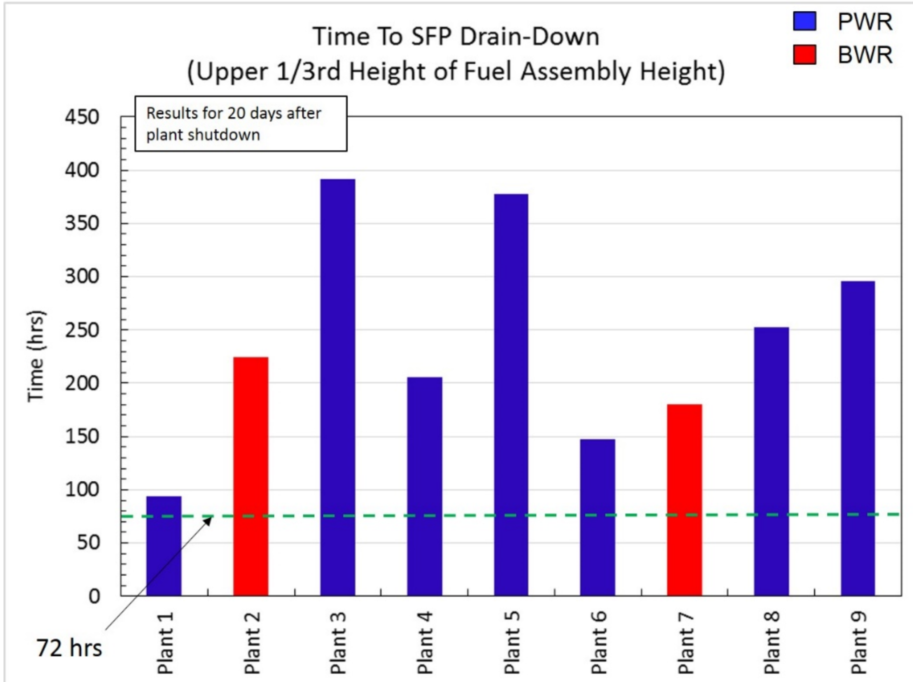

Figure 4-3
SFP drain-down results for plants with peak Sa > 0.8g

4.3 Site-Specific SFP Criteria for Sites with GMRS Peak Sa Greater Than 0.8g

Sections 4.1 and 4.2 provide evaluation criteria that can be applied at sites where the GMRS peak spectral acceleration (Sa) is greater than 0.8g.

Site-specific calculations using the criteria in Section 4.1 are required to demonstrate integrity of the SFP structure. In addition, the following parameters should be verified on a site-specific basis to confirm that the evaluation criteria applies to the site.

Structural Parameters:

1. The SFP structure should be included in the Civil Inspection Program performed in accordance with Maintenance Rule

Non-Structural Parameters: (the criteria below assumes the site and structural criteria above (items 1-3) are satisfied)

2. To confirm applicability of the piping evaluation in Section 4.2, piping attached to the SFP should have penetrations no more than 6 ft below water surface.
3. To ensure ductile behavior, SFP gates should be constructed from either aluminum or stainless steel alloy materials.
4. Anti-siphoning devices should be installed on any piping that could lead to siphoning water from the SFP. In addition, any cases where

active anti-siphoning devices attached to 2-inch or smaller piping and have extremely large extended operators, the valves should be walked down to confirm adequate lateral support

5. To confirm applicability of the sloshing evaluation in Section 4.2.5, the maximum SFP pool horizontal dimension (length or width) should be less than 125 ft and the SFP depth should be greater than 36 ft. The site-specific GMRS should be the same as that submitted to the NRC between March 2014 and July 2015, which the NRC has found acceptable for responding to the NRC 50.54(f) letter [1].
6. To confirm applicability of the evaporation loss evaluation in Section 4.2.6, the SFP surface area should be greater than 500 ft^2 and the licensed reactor core thermal power should be less than 4,000 MWt per unit.

Section 5: Conclusions

The NRC 50.54(f) letter [1] requested that a seismic evaluation be performed on the SFP to consider all seismically induced failures that could lead to rapid draining. The evaluation described in Section 3 addresses this requirement for plants with low-to-moderate seismic ground motions (peak spectral accelerations less than 0.8g). The evaluation described in Section 4 addresses this requirement for plants with high seismic ground motions (peak spectral accelerations greater than 0.8g).

The evaluation addresses both structural and non-structural aspects in accordance with the SPID [2]. Structural failure modes are evaluated using criteria in EPRI NP-6041 [10] as an alternate to the checklist provided in NUREG-1738 [4]. For high-seismic plants, structural failure modes are evaluated in accordance with Section 4 of this report.

The non-structural failure modes considered were those that could affect the ability of SFPs to maintain SFP inventory for 72 hours. These included: (1) penetrations that could lead to uncovering the fuel, (2) SFP cooling functional failures that could lead to siphoning inventory from the pool, (3) sloshing losses, and (4) evaporative losses.

Surveys of industry plants were performed to gain an understanding of the ranges of important SFP parameters, such as wall and floor spans, thicknesses, reinforcement, support configuration, elevation above grade, depth, representative heat loads, gate location, and piping penetration depth, etc.

Spent fuel pool structures are constructed with thick reinforced concrete walls which contribute to the seismic robustness. Past risk studies, including the NRC's Spent Fuel Pool Scoping Study, have demonstrated low seismically-induced failure frequencies and HCLPF values greater than 0.5g PGA. Earthquake experience from Japan has also provided examples of robust pool structures. In the case of the six units at Fukushima Daiichi, the measured horizontal peak ground accelerations from the 2011 Tohoku Earthquake ranged from 0.29g to 0.56g, which is generally higher than the U.S. fleet GMRS PGAs. For these plants, there was no reported leakage of SFP water, other than from seismic-induced sloshing.

Structures supporting or housing SFPs are typically constructed with reinforced concrete shear walls, moment frames, steel frames, and post-tensioned girders. For plants with peak spectral accelerations less than 0.8g, these structures can be screened using criteria described in EPRI NP-6041 [10]. Plants with higher peak spectral accelerations will perform plant-specific structural evaluation of the SFP. Piping systems connected to the SFP are seismically rugged as are fuel transfer gates, which are constructed with stiffened steel plates.

Site-specific calculations, based on GMRS demands, conservative estimates of sloshing losses, and realistic SFP heat loads, were performed. For low-to-moderate seismicity plants, the estimated time to drain-down to the upper 1/3rd fuel assembly height ranged from 149 hours to over 700 hours. These estimates provide reasonable assurance that there will not be a rapid drain-down of the SFP leading to uncovering more than 1/3rd of the fuel in 72 hours and that additional make-up capabilities are not required to be credited.

For higher seismicity plants, the estimated time to drain-down to the upper 1/3rd fuel assembly height ranged from 94 hours to over 390 hours. These estimates provide reasonable assurance that there will not be a rapid drain-down of the SFP leading to uncovering more than 1/3rd of the fuel in 72 hours and that additional make-up capabilities are not required to be credited.

This report provides screening criteria, which will enable plants to confirm that their plant parameters are enveloped by those considered in this evaluation.

Section 6: References

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24. American Concrete Institute, *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*. ACI 349-01, 2001.

25. Timoshenko S., *Theory of Plates and Shells*. McGraw-Hill, 1959.
26. *Seismic Evaluation Guidance: Spent Fuel Pool Integrity Guidance*, EPRI, Palo Alto, CA: 20162. 3002007148

Appendix A: Spent Fuel Pool Data

In April 2015, the industry conducted a survey of key SFP parameters. The parameters considered were those relating to physical configuration of the pool (size, elevation above grade, and supporting structure) and structural characteristics (reinforcement percentage, steel and concrete strength, etc.). Understanding the range of these parameters is helpful in comparing to generic seismic capacity criteria (e.g., EPRI NP-6041 [10]) and in drawing conclusions from earlier studies regarding the robustness of SFP structures. The range of core thermal power was also assessed, as it is an important parameter in evaluating SFP heat loads.

The SFP survey requested design information from those plants that screened in for an SFP evaluation under NTTF 2.1. However, on the basis that the SFP survey collected detailed design information from over 60-percent of the SFPs across the U.S. NPP fleet (e.g., PWR and BWR Mark I, II, III), it is believed that the survey results are also representative of the those plants that did not screen in for an SFP evaluation.

The data presented in this appendix are presented in histogram format in order to easily visualize the distribution in results.

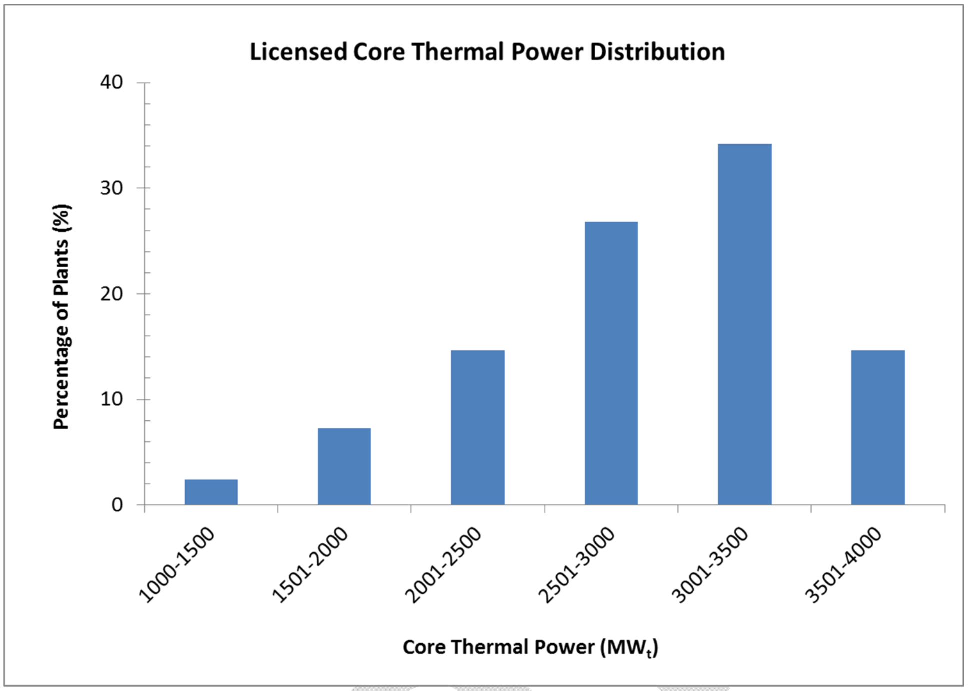


Figure A-1
Plant Licensed Core Thermal Power Distribution

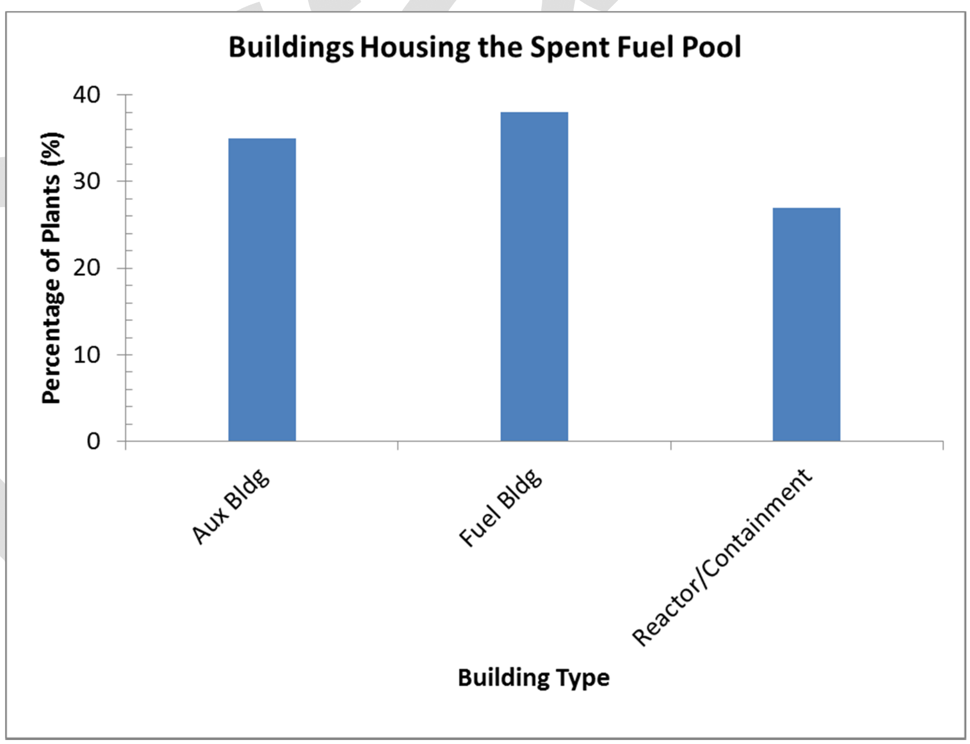


Figure A-2
Buildings Housing the Spent Fuel Pool

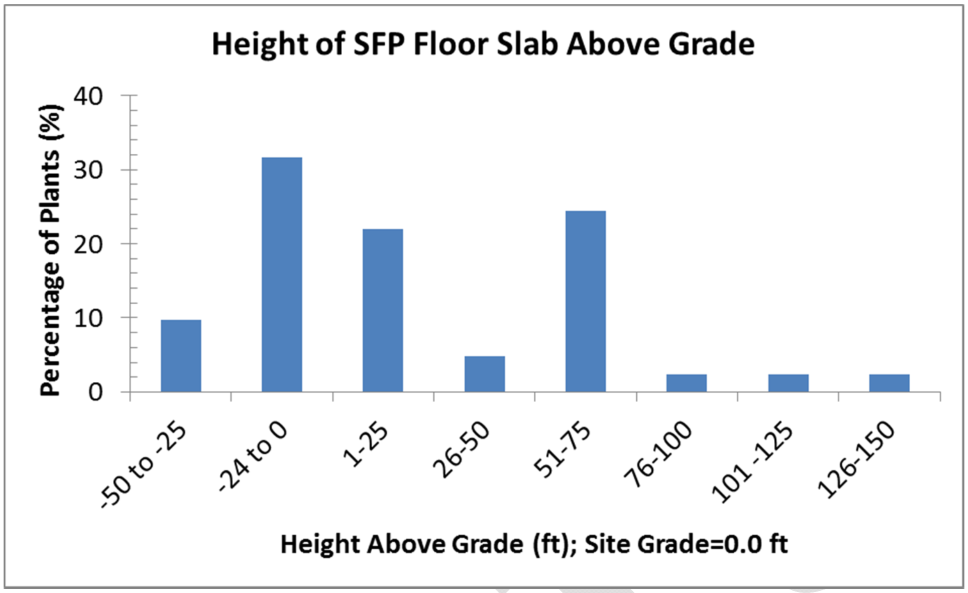


Figure A-3
Distribution of Height of SFP Floor Slab Above Grade Elevation

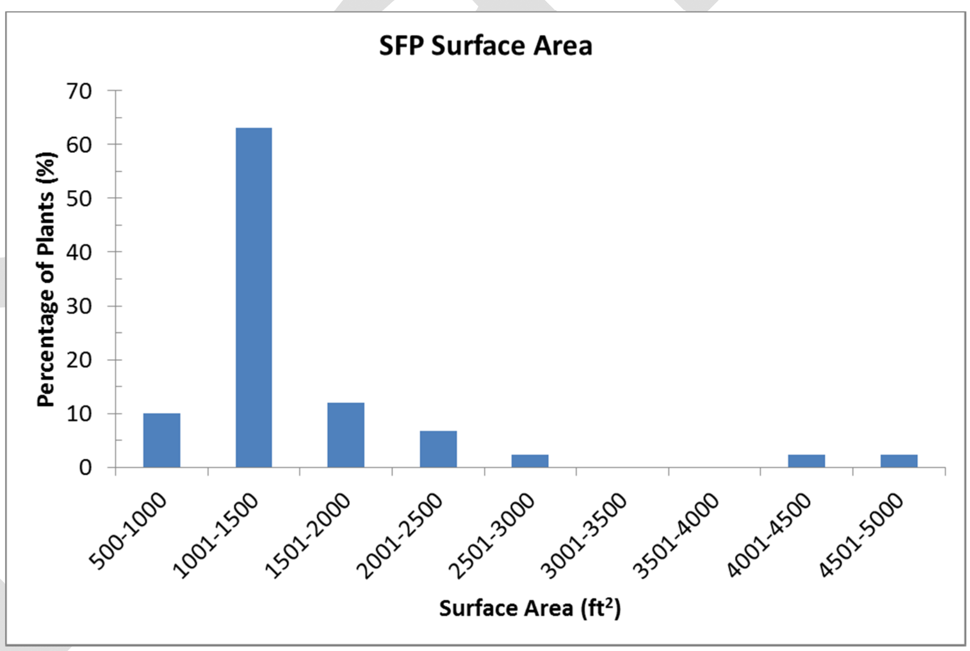


Figure A-4
Spent Fuel Pool Surface Area Distribution

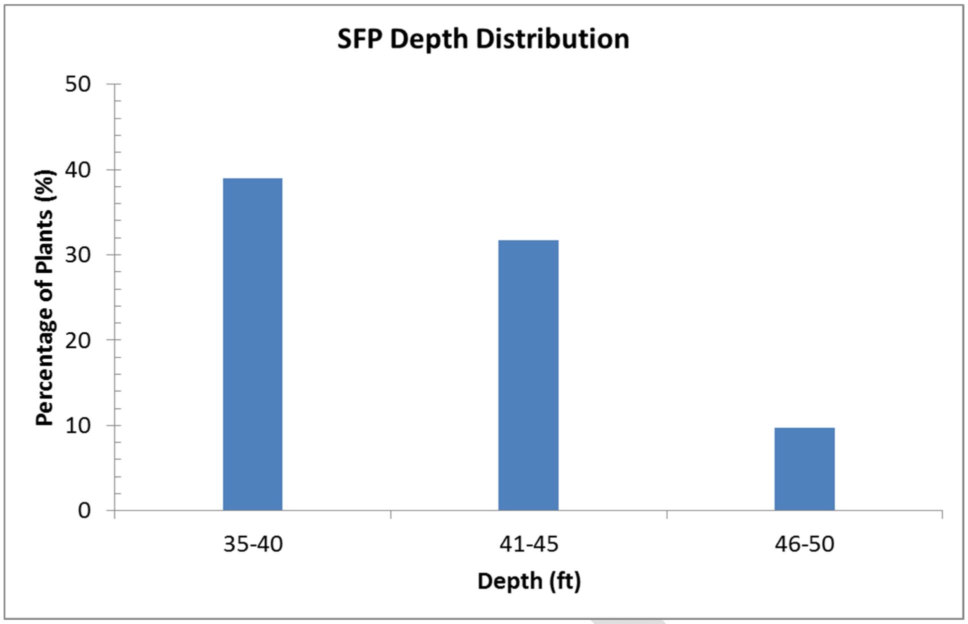


Figure A-5
Spent Fuel Pool Depth Distribution

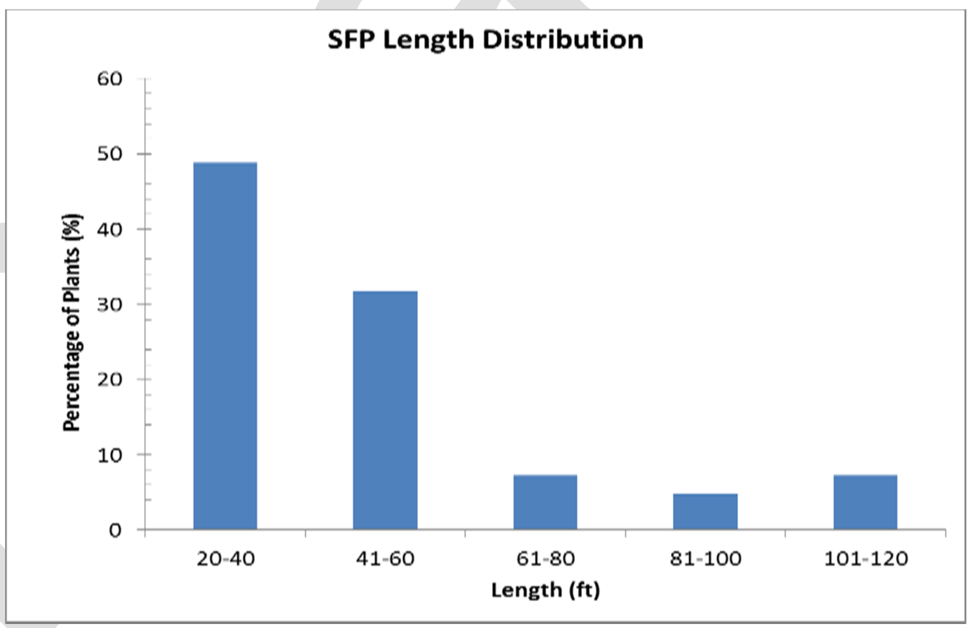


Figure A-6
Spent Fuel Pool Length Distribution

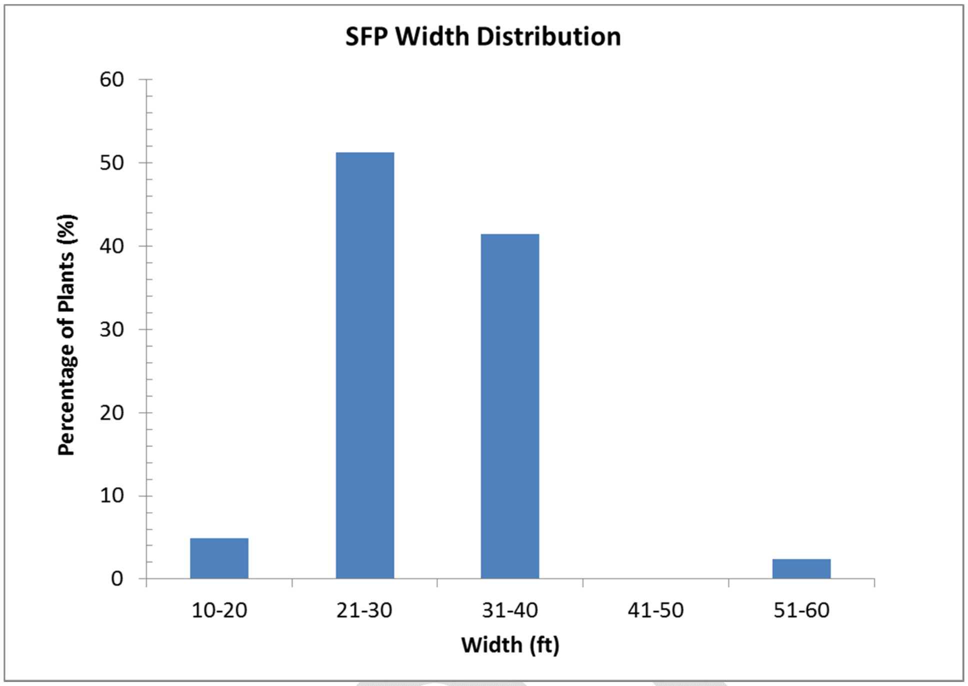


Figure A-7
Spent Fuel Pool Width Distribution

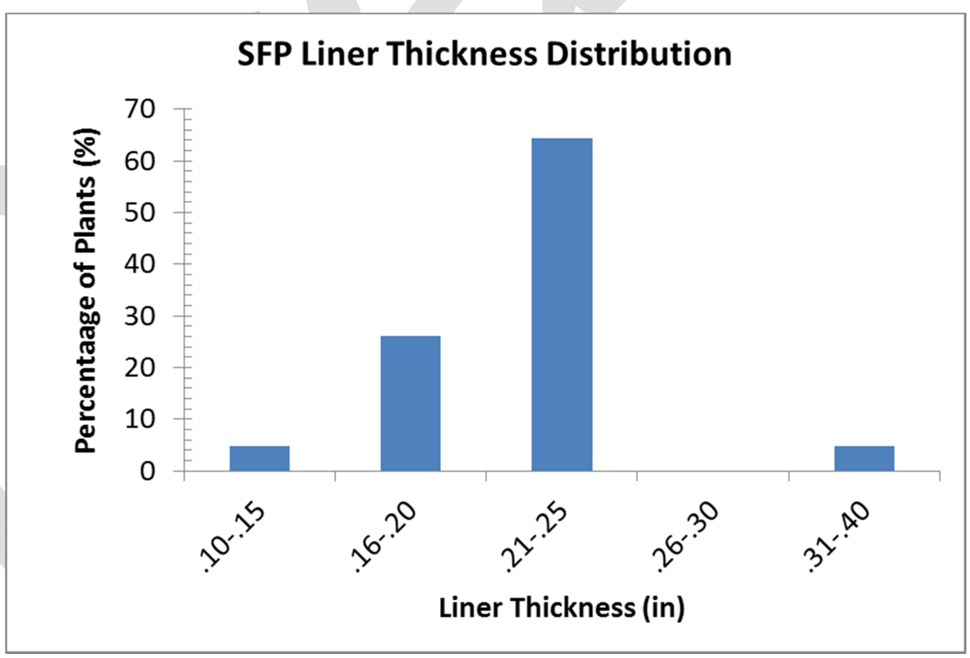


Figure A-8
Spent Fuel Pool Liner Thickness Distribution

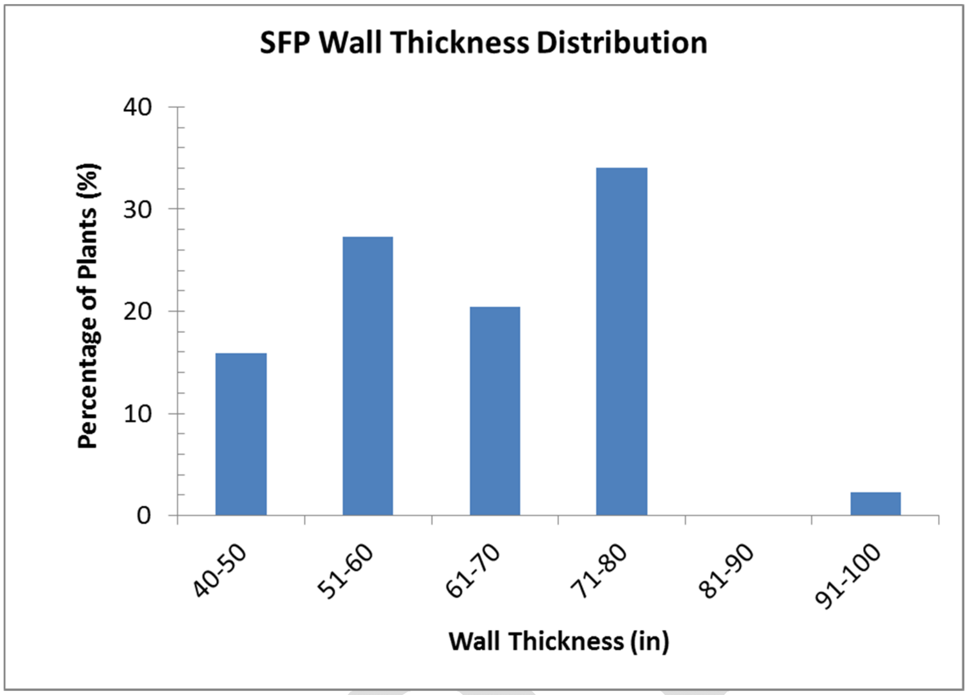


Figure A-9
Spent Fuel Pool Wall Thickness Distribution

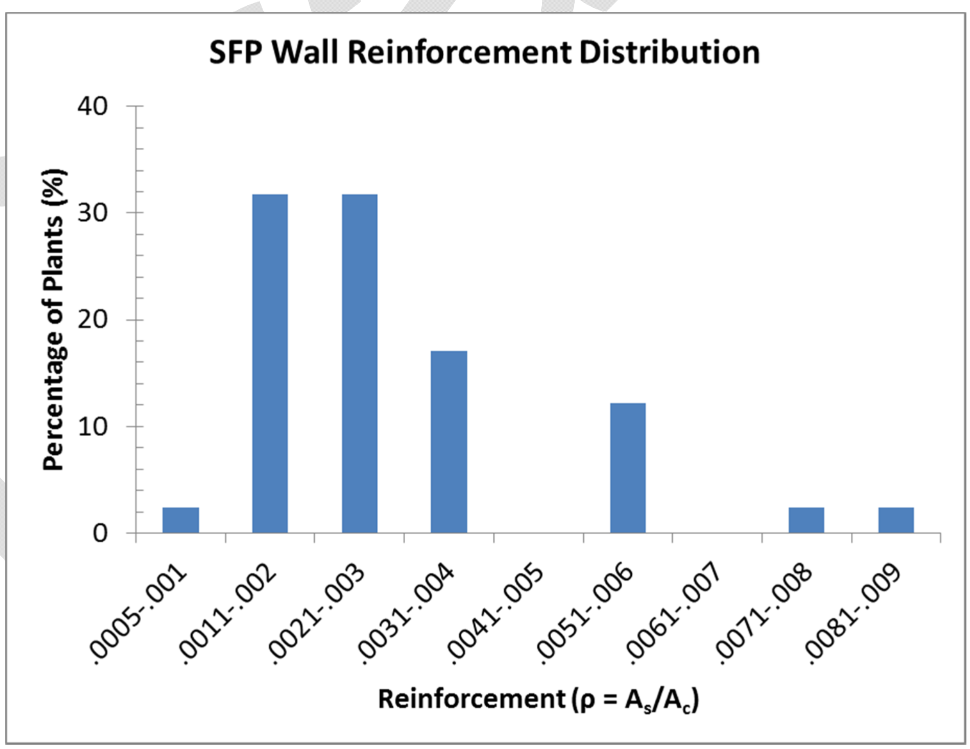


Figure A-10
Spent Fuel Pool Wall Reinforcement Ratio Distribution

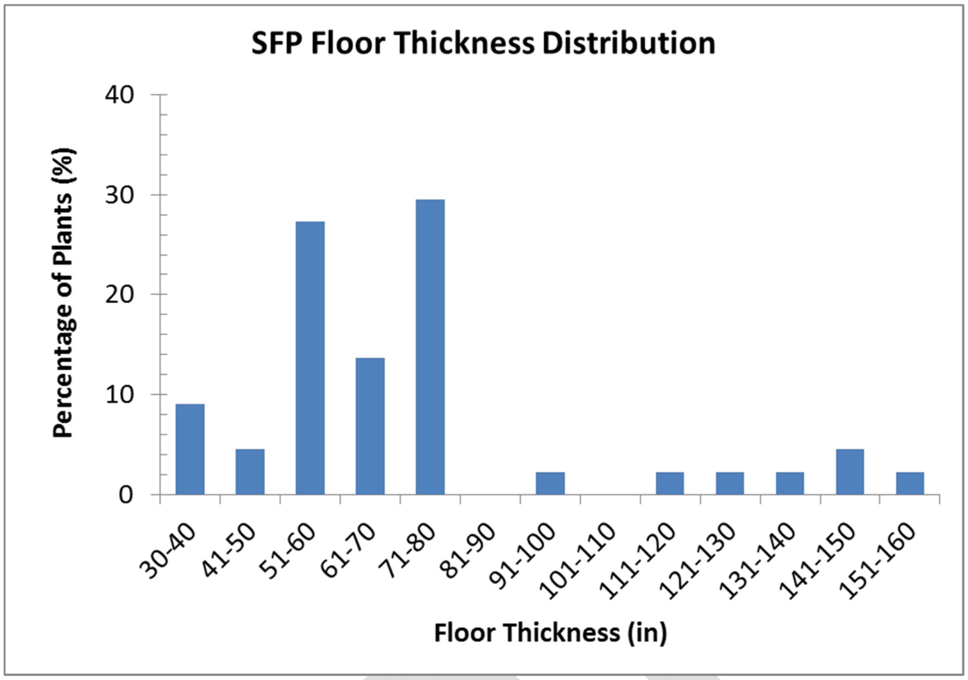


Figure A-11
Spent Fuel Pool Floor Thickness Distribution

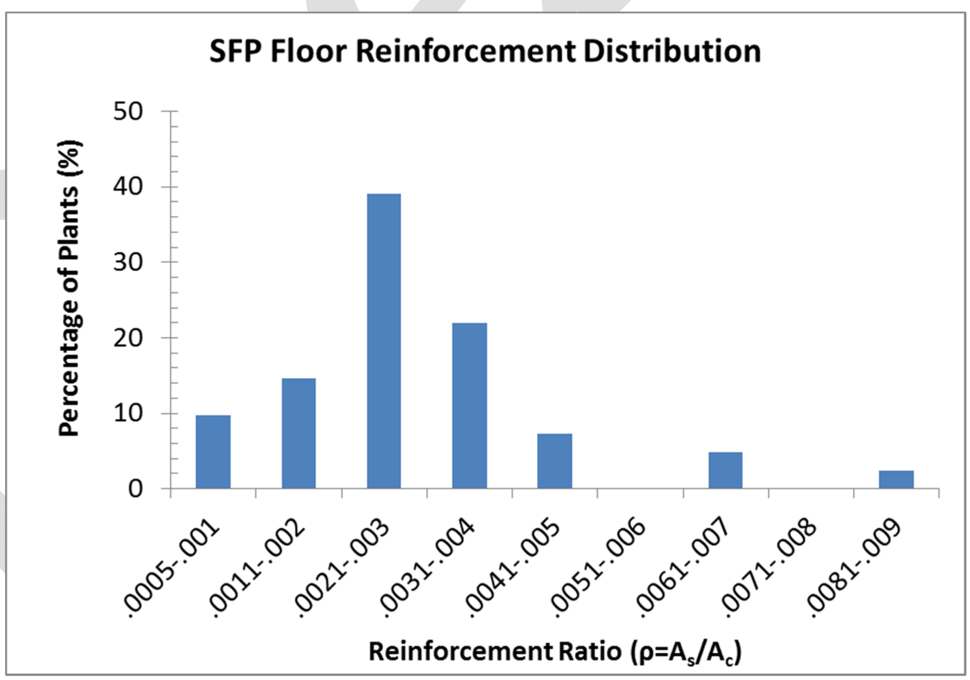


Figure A-12
Spent Fuel Pool Floor Reinforcement Distribution

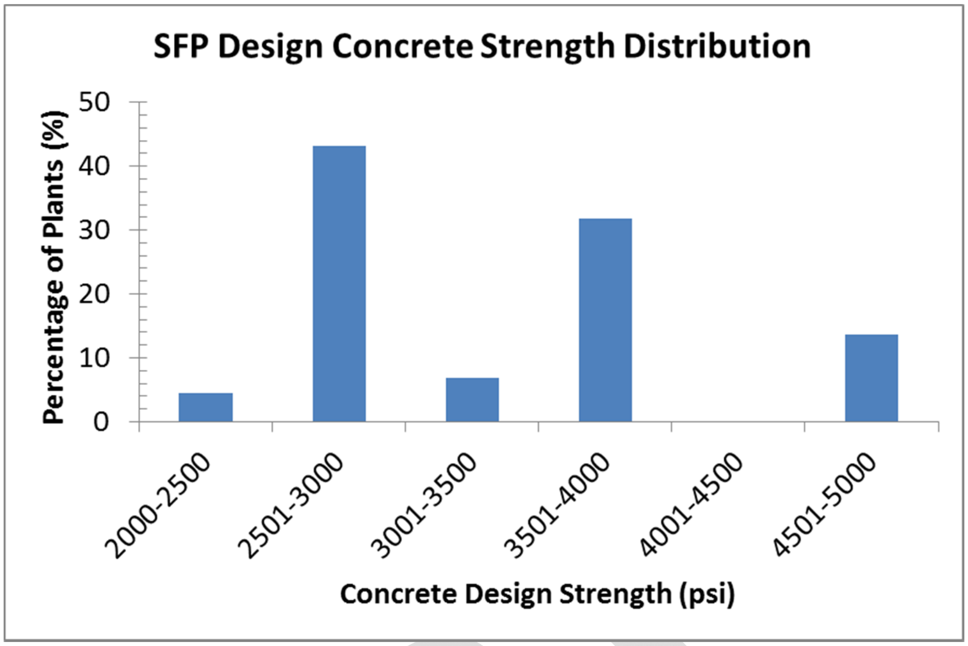


Figure A-13
Spent Fuel Pool Design Concrete Strength Distribution

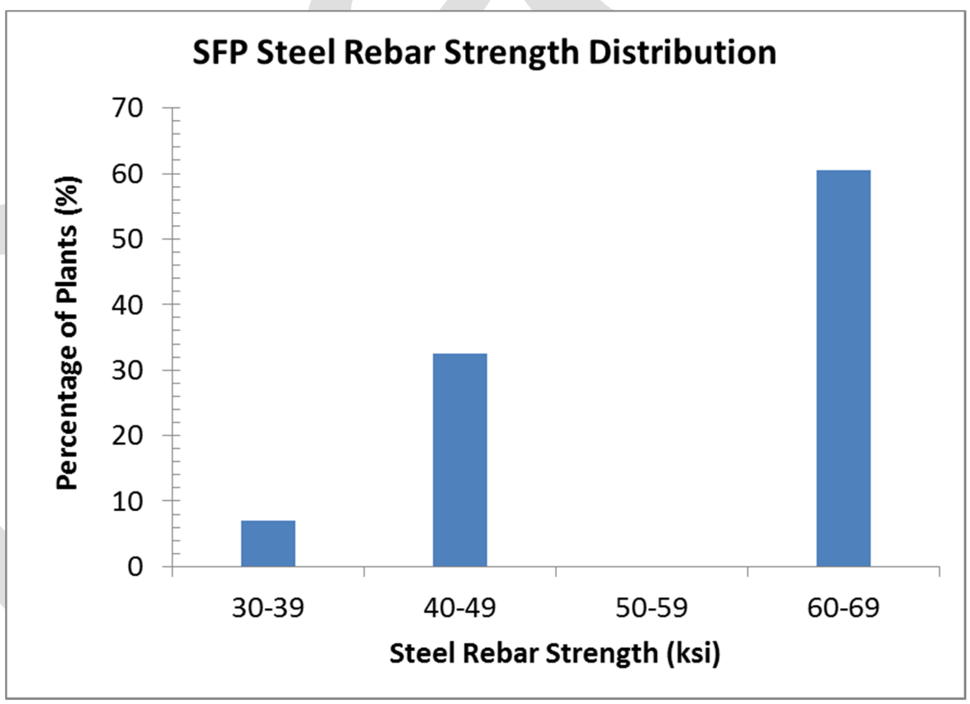


Figure A-14
Spent Fuel Pool Steel Rebar Strength Distribution

Appendix B: Sample Spent Fuel Pool Heat Up and Boil-Off Calculation

B1.0 Purpose

The purpose of this example calculation is to illustrate the approach used to estimate site-specific SFP boil-off times in the event that (1) cooling and makeup systems are rendered in-operable and (2) water inventory is lost due to seismic-induced sloshing. The approach for estimating heat-up and boil-off times is consistent with Appendix EE to EPRI TR 1025295, “Severe Accident Management Guidance Technical Basis Report,” [17] which is referenced as an acceptable method in the SPID [2] (Section 7.3.1). While representative, the selected SFP geometry (35 ft x 45 ft x by 40 ft) does not relate to a particular plant (Figure B-1). Site-specific calculations (Figures 3-6 and 3-8) make use of realistic SFP heat loads (based on survey results from several plants) and site-specific seismic demands based on respective GMRS motions.

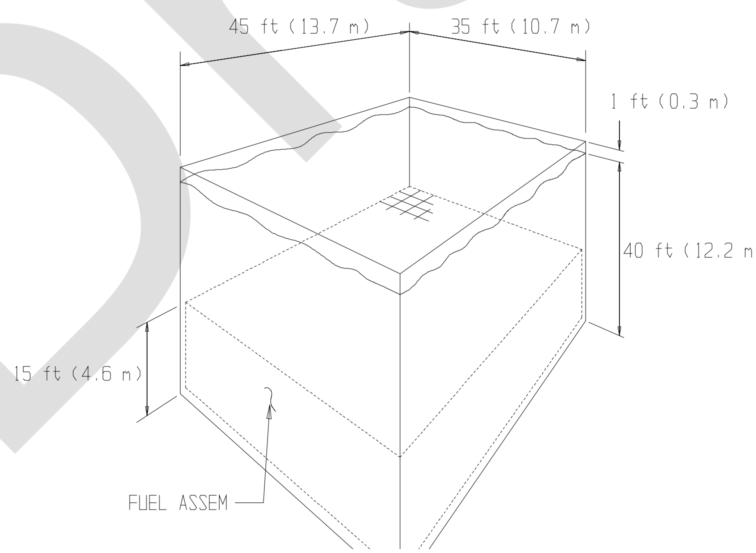


Figure B-1
Example Spent Fuel Pool Geometry

B2.0 Assumptions

The following assumptions are used in this example calculation.

- Licensed reactor core power = 3,000 MWt
- Pool depth = 40.0 ft
- Pool freeboard = 1 ft
- Pool width = 35 ft
- Pool length = 45 ft
- Seismic-induced sloshing height = 4.0 ft (SRSS sloshing height in accordance with SPID [2])
- Fuel assembly and rack height = 15 ft
- Spent fuel volume = 1,779 ft^3 (EPRI 1025295 [17])
- Water specific heat (std value) = 1 BTU/lb/°F
- Initial pool temp = 100°F
- Boil temp = 212°F
- Water heat of vaporization, h_{fg} , = 970.3 Btu/lbm (at 212°F, P=1atm)
- Specific volume of water at 100°F (1 atm) = 62.04 lb/ft^3
- Specific volume of water at 212°F (1 atm) = 60.29 lb/ft^3

B3.0 Sample Calculation

1. Estimate SFP heat load at 20 days after shutdown, Q, based linear interpolation of realistic heat load data.

   A distribution of BWR and PWR units, ranging in core thermal power from 1,800 MWt to 4,000 MWt, were surveyed for representative SFP heat loads and outage durations. Results indicated (1) a strong correlation of SFP heat load with core thermal power, (2) plants only retain 30-40% of the core in the SFP after the outage, and (3) the typical outage period lasts from 20-30 days. The SFP heat load is compared with the rated thermal power in Figure B-2. A linear curve fit was applied to that data, and an 84th percentile was used conservatively estimate the SFP heat load for any plant that was not specifically surveyed using the following equations.

   Q = 5.77E-4 (x) + 1.29 [Blue line in Figure B-2 below]

   x = 3,000 MWt (assumed rated core power for this example)

   Q = 5.77E-4 (3,000) +1.29 (MW)

   Q = 3.02 MW (10.31E6 Btu/hr)

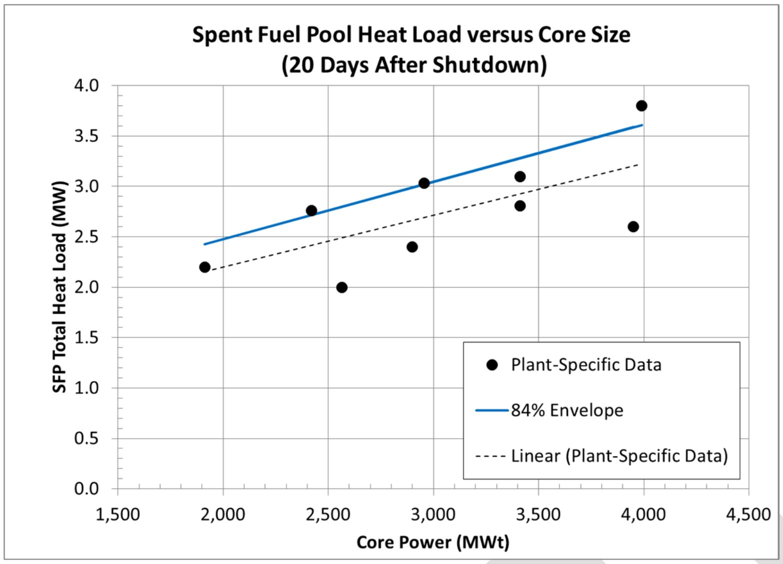


Figure B-2
Survey Results for Licensed Core Power Versus Spent Fuel Pool Heat Load

2. Estimate SFP water depth accounting for seismic-induced losses. Sloshing amplitudes are estimated in in accordance with SPID criteria. For this example, the sloshing amplitude is assumed to be 4.0 ft.

   SFP water depth (post-sloshing) = 40.0 ft - 4.0 ft (sloshing) + 1 ft (freeboard) = 37.0 ft

3. Estimate time to heat pool water to boiling, t

   Neglecting losses due to heat conduction and heat convection (conservative):

   $Q = \frac{mC_p\Delta T}{t}\ (Btu/hr)$; Ref EPRI TR1025295, Equation EE-6 [17]

   $t = \frac{mC_p\Delta T}{Q}\ (hr)$

   Where:

   t = time to boil (hr)

   Q = heat input (Btu/hr)

   m = mass of pool water (lbm)

   C_p = specific heat of water = 1 BTU/lb/°F

   ΔT = temperature rise (°F)

SFP water volume = (pool length • pool width • water depth) - spent fuel volume

SFP water volume = (45.0 ft • 35.0 ft • 37.0 ft) - 1,779 ft^3 = 56,496 ft^3

m = 56,496 ft^3 • 62.04 lb/ft^3 = 3.505E6 lb

ΔT = boil temp - initial temp = 212°F - 100°F = 112°F

t = (3.505E6 lb)(1 BTU/lb/°F)(112°F)/(10.31E6 Btu/hr)

t = 38.08 hr (time to boil)

4. Estimate boil-off rate, $\dot{H}$

   H = Q /(ρ_w • A_{sfp} • h_{fg}) ; Ref EPRI TR1025295, Equation EE-7

   Where:

   - A_{sfp} = 35 ft • 45 ft = 1,575 ft^2
   - ρ_w= 60.29 lb/ft^3 (@212°F)
   - h_{fg} = heat of vaporization; 970.3 Btu/lbm (at 212°F, P=1atm)
   - Q = 10.31E6 Btu/hr

   H = (10.31E6 Btu/hr)/(60.29 lb/ft^3 • 1,575 ft^3 • 970.40 Btu/lb)

   H = 0.112 ft/hr

5. Estimate time to boil-off to top of fuel assemblies:

   Depth above spent fuel = 37.0 ft - 15.0 ft = 22.0 ft

   T_{top} = 38.08 hr (heat up) + (22.0 ft)/(0.112 ft/hr) = 234.5 hrs

6. Estimate time to boil-off to upper 1/3rd of fuel assembly height in accordance with SPID Section 7:

   Depth above upper 1/3rd height = 37.0 ft - 10.0 ft = 27.0 ft

   $T_{upper\text{-}third}$ = 38.08 hr (heat up) + (27.0 ft)/(0.112 ft/hr) = 279.2 hrs

7. Conclusion

   This sample calculation, based on realistic SFP geometry, conservative seismic-induced sloshing losses, and realistic SFP heat loads indicated that there is significantly more than 72 hours before uncovering of spent fuel. For this example, the estimated time to drain-down to the top of the fuel assemblies is 234 hours. Similarly, the estimated time to drain-down to the upper 1/3rd fuel assembly height is 279 hours. The significant margin beyond 72 hours helps to provide confidence that there is adequate time to employ operator actions and mitigation strategies to maintain SFP cooling under extreme seismic events.

Appendix C: Sample Spent Fuel Pool HCLPF Calculation

C1.0 Purpose

The purpose of this report is to provide a calculation that describes an approach for estimating a seismic High Confidence of Low Probability of Failure (HCLPF) of a typical spent fuel pool (SFP). The approach makes use of a rigorous single degree-of-freedom model that is generic and can be applied to a range of SFPs. The referenced materials, including codes and standards, are typical of those used in structural design and analysis. A comparison of results to an NRC SFP analysis [15] was performed to increase confidence in model behavior under seismic demands.

C2.0 Introduction and Background

In 2013, the NRC issued a Spent Fuel Pool (SFP) Scoping study [15] to continue its examination of the risks and consequences of postulated SFP accidents initiated by a low likelihood seismic event. The seismic event considered in the study was based on a central and eastern United States (CEUS) location (Peach Bottom) and an extremely rare recurrence interval (frequency of 1/60,000 years). The resulting free-field ground motion had a 5% damped peak spectral acceleration of 1.8g and peak ground acceleration of 0.7g.

That study used a detailed finite element analysis of the Peach Bottom SFP, including the surrounding structure, and concluded that the SFP had adequate capacity to withstand the evaluated ground motions.

The purpose of this calculation is to use insights and results from the NRC Scoping study to develop a less complex SFP model that can produce a comparable assessment of the ability of an SFP to withstand high ground motions. To achieve this, a seismic analytical model was developed which makes use of several key insights from the NRC Scoping study:

- SFP walls and floors are the key structural elements governing seismic adequacy
- SFP walls and floors have relatively high natural frequencies ($\geq$ 10 Hz).

- SFP walls and floors have relatively small seismic induced displacements.

The central approach in this calculation is to estimate the fundamental frequency of the SFP wall and floor panel systems and then perform an equivalent-static dynamic analysis to estimate peak displacements and reaction forces. For the dynamic analysis, an analytical model, composed of a single-degree-of-freedom lumped mass and bilinear stiffness resistance function, was developed. This elastic-plastic model, common in engineering design and analysis, is based on the approach discussed in Biggs [21]. The dynamic model requires an estimate of panel ultimate strength and stiffness for both the elastic and elastic-plastic response (Figure C-1). The panel ultimate flexural strength is derived using slab yield-line theory practiced in standard structural design [22]. The ultimate panel strength is based on the panel span aspect ratio (long side/short side), and nominal flexural section strength accounting for both positive and negative steel reinforcement. Out-of-plane shear strength, based on ACI-349 provisions, is also evaluated.

The NRC study [15] analyzed the Peach Bottom spent fuel pool structure using a foundation input response spectra (FIRS) with a peak spectral acceleration of 1.8g and a peak ground acceleration of 0.7g in the horizontal direction (ref Figure C-18). For the purpose of comparing results, this study assumes the same seismic input motions (ref Section C5.0).

The method of analysis in this report makes use of the Conservative Deterministic Failure Margin approach (CDFM). EPRI NP-6041 [10] describes the CDFM approach in detail.

The method of seismic modeling is consistent with the approach in ASCE 4-98 [18]. A median-centered model is developed, consistent with the CDFM approach. Consistent with EPRI TR-103959 [19], the median damping for reinforced concrete is assumed to be 5% since the steel reinforcement will be beyond or just below yield for the severe load case. Nominal design values for concrete compressive strength (4,000 psi) and rebar yield strength (60,000 psi) are assumed in the model. The effective stiffness of the SFP floor and wall systems were reduced 50 percent in accordance with ASCE 43-05 [20].

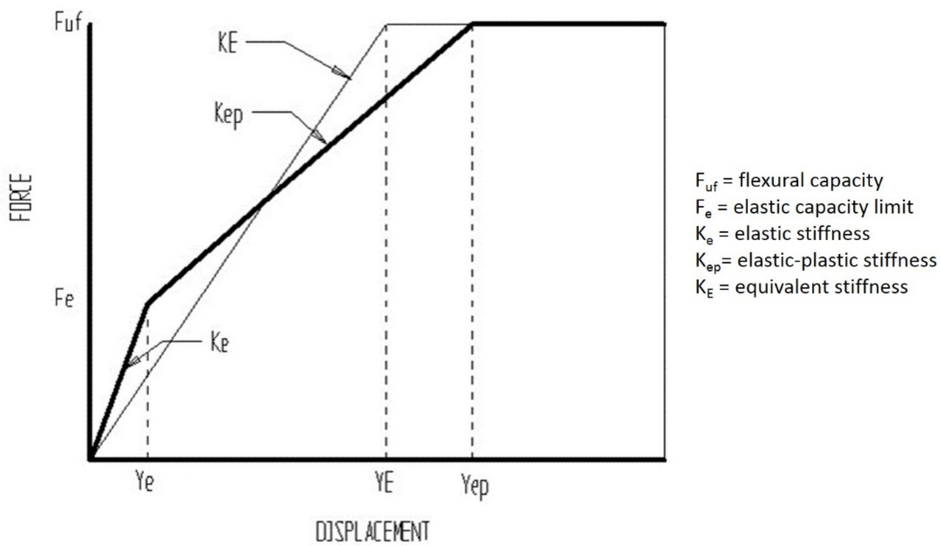


Figure C-1
Example Elastic Plastic Response Model

The natural frequencies of the SFP wall and floor panels are functions of material properties (density, nominal strengths), geometry (thickness, span), support boundary conditions, and rack/fuel assembly masses (in the case of floor panels). Panel frequencies also account for the effects of added water mass.

Response spectra used for estimating equivalent accelerations are referenced from the NRC analysis [15] and are shown in Section C5.0 of this report. Convective and inertial hydrodynamic loads are considered in the analysis, consistent with ACI design guidance pertaining to liquid-containing concrete structures [16]. Responses from the horizontal and vertical directions are combined using the 100-40-40 rule [18]. Displacements are checked to confirm they are small and shear demands are compared to ACI-349 code limits [24]. HCLPF's for both floors and walls are calculated without consideration of a ductility factor due to lack of shear reinforcement.

The following parameters will be compared between the NRC Scoping Study and the SDOF model:

- Total weight for the SFP floor slab
- Natural frequencies of the floor slab
- Dead load, hydrodynamic, and seismic pressures on the floor slab
- Peak floor displacements

The approach described herein is generic and can be applied to a wide range of SFP designs. The steps in the HCLPF calculation, and corresponding appendix sections are described in Figure C-2.

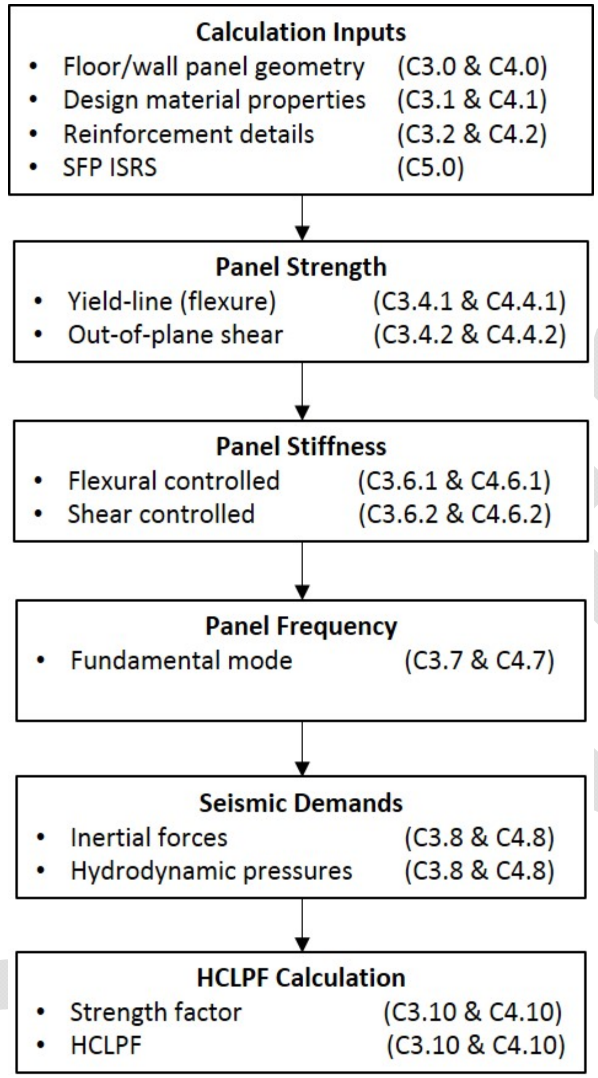


Figure C-2
SFP HCLPF process and corresponding appendix sections

C2.1 Design Description

The Peach Bottom Nuclear Station is a BWR Mark I containment design with an elevated SFP. The BWR Mark I containment design typically has an SFP that is approximately 60 feet above grade elevation (Figure C-3). The layout of the Peach Bottom SFP floor and wall system is shown in Figure C-4. The SFP measures 35.3 ft by 40 ft and is 39.0 ft deep (Figure C-5). The pool wall thickness ranges from 72 inches (lower half) to 60 inches (upper half). The pool floor is approximately 75 inches thick and is supported by deep steel beams (used for supporting construction loads). For this analysis, these beams are not credited in the estimation of floor strength.

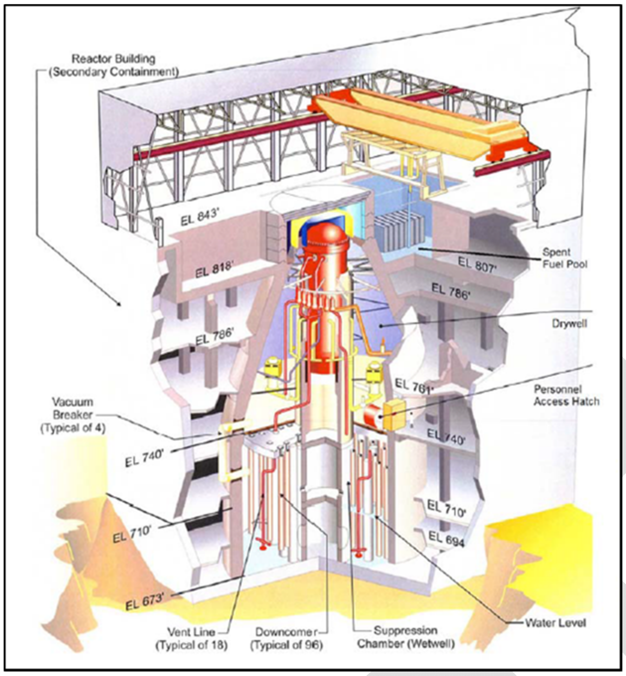


Figure C-3
Schematic for Typical BWR Configuration with Elevated Spent Fuel Storage Pool

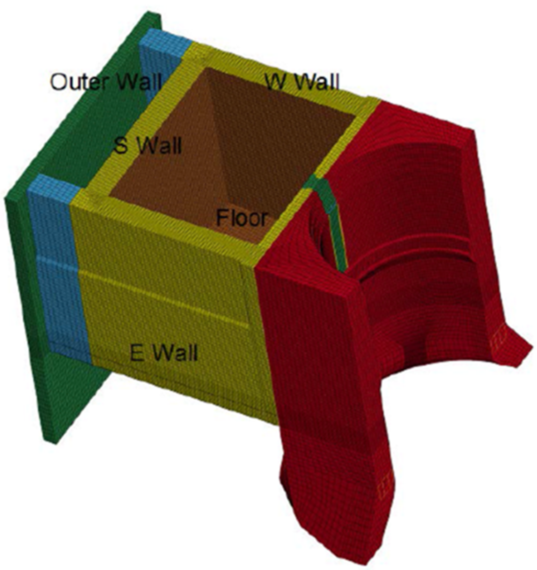


Figure C-4
Peach Bottom Spent Fuel Pool (Source: NRC Spent Fuel Pool Scoping Study)

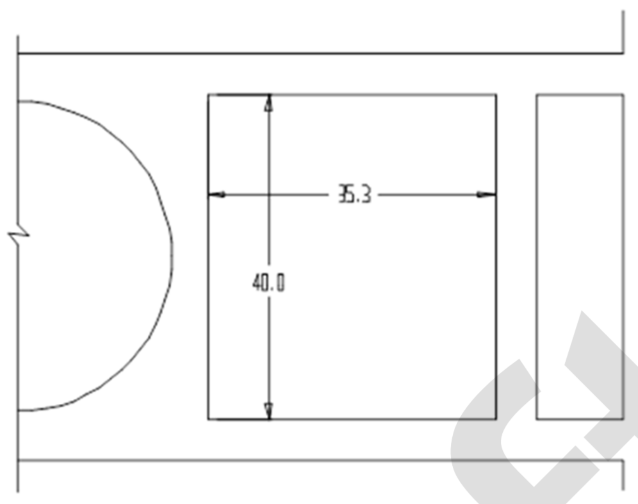


Figure C-5
Plan View of Spent Fuel Pool Indicating Approximate Dimensions

C2.2 NRC Scoping Study Description

The NRC SFP Scoping Study used a detailed seismic model to estimate damage caused by an extreme seismic event. The reactor building and SFP were discretized using a 3D finite element building model. The model was extremely refined and explicitly considered the ¼" thick stainless steel liner. As the model was a full building model, forces transmitted to the supporting structure were captured in the analysis. The horizontal building frequency was calculated to be approximately 7 Hz and the vertical SFP floor frequency was approximately 14 Hz. The wall frequency was not reported. Under the 0.7 PGA demands (Figure C-18), the SFP floor had a relatively small displacement, less than 0.5 inches. Given a span of over 35 feet, the deflection was less than L/800; which is considered a small displacement.

C3.0 SFP Floor Slab Evaluation

The Peach Bottom SFP floor measures 40 ft by 35.3 ft. Steel beams, used for construction purposes, span the 40 ft direction, but they are neglected for the purpose of this calculation. The thickness of the floor slab is 75 inches. For the purpose of strength and stiffness calculations, the floor depth is assumed to be 73 inches due to ACI concrete cover provisions. A review of the SFP design drawings was performed and it was found that the positive and negative reinforcement varied between the panel center and boundary (Figure C-6), which is considered to be a typical condition. For this analysis, the floor panel was assumed to be fixed at the boundary.

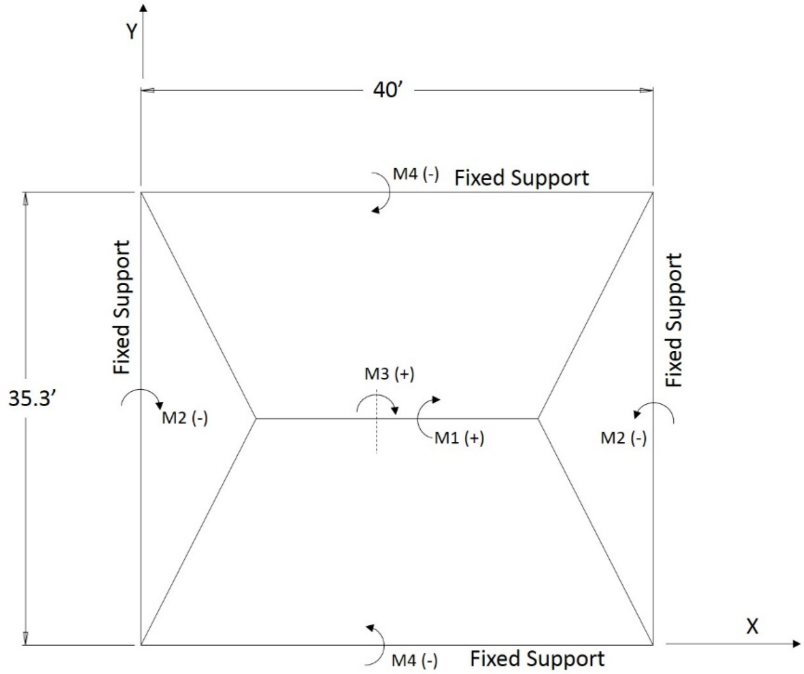


Figure C-6
SFP floor slab schematic indicating example yield line pattern and locations of positive (+) and negative (-) flexural reinforcement. Reinforcement per Table C-2.

C3.1 Material Properties

As the CDFM approach requires the use of nominal design values, the Peach Bottom design values were used as shown in Table C-1.

Table C-1
Assumed Material Properties

Design Parameter	Value Used For This Calculation
Concrete Density	150 lb/ft^3
Nominal Concrete Strength	4,000 psi
Rebar Strength (Grade 60)	60,000 psi
Young's Modulus	3.6E6 psi

C3.2 Concrete Reinforcement

Steel reinforcement ratios were estimated based on the SFP design drawings. Table C-2 provides a summary of the reinforcement design:

Table C-2
Panel Reinforcement Assumptions (Based on Design Drawings)

Panel Location	Reinforcement	Reinforcement Ratio
M1; Total Positive Moment at Center	#11@9”	0.0023
M2; Total Negative Moment at Center of Side Edge	(2) #11@9”	0.0045
M3; Total Positive Moment at Center	Bundle (3) #11@12”; two layers	0.010
M4; Total Negative Moment at Center of Long Edge	#11@12”	0.0017

C3.3 Floor Panel Section Strength

Both flexure and shear section strength is calculated for each panel.

C3.3.1 Section Flexural Strength

The nominal panel flexural strength can be calculated as:

$$M_n = \rho f_y b d^2 \left(1 - 0.59 \frac{\rho f_y}{f'_c}\right) \text{ [24]}$$

Factored strength, $M_u = \Phi M_n$; $\Phi = 0.9$ [24]

Table C-3
Panel Ultimate Flexural Capacities (Factored)

Panel Location	Total Factored Strength (Mu); Φ=0.9	Factored Strength (Mu) per unit length
M1; Total Positive Moment at Center	3.06E8 lb-in	6.37E5 lb-ft/ft
M2; Total Negative Moment at Center of Side Edge	5.40E8 lb-in	1.27E6 lb-ft/ft
M3; Total Positive Moment at Center	1.19E9 lb-in	2.81E6 lb-ft/ft
M4; Total Negative Moment at Center of Bottom Edge	2.30E8 lb-in	4.79E5 lb-ft/ft

C3.3.2 Section Shear Strength

The nominal shear strength of the panel can be calculated from ACI 349 [24]

$$V_n = 2\sqrt{f'_c}\, bd$$

$V_n = 2 \text{ x } (4{,}000 \text{ psi})^{0.5} \text{ x } 12 \text{ in x } 73 \text{ in } = 110{,}806 \text{ lb/ft}$

$V_u = \Phi V_n$; $\Phi = 0.85$ for shear

V_u = 94,185 lb/ft

C3.4 Panel Strength

Both flexure and shear strength is calculated for the floor panel to confirm the limiting failure mode.

C3.4.1 Flexural Strength

The approach to estimate the pressure capacity of the floor panel is based on a yield line analysis. Figure C-6 shows an example yield line pattern. The assumption is that internal work done by the moments at the yield line pattern is equivalent to the external work caused by the pressure loading. The approach described by Nilson and Winter was used [22]. It is noted that the capacity developed using this method is based on flexure and assumes that the supports can carry the shear demands. The shear strength is calculated in Section C3.4.2 of this report.

For the floor slab, uniform hydrodynamic loading was assumed. The angle of the diagonal yield line was varied until the calculated critical pressure was mimimized. From this analysis, the angle of 41.4 degrees minimized the external work required to form hinges at all yield lines. The yield line pattern is characterized as having diagonal yield lines meet at the center of the panel. Based on this, the total internal work provided by the panel yield lines is estimated to be 1.95E7 lb-ft. The work required to develop plastic hinges at the panel boundary is 2.17E6 lb-ft.

The total internal work, W_i, is: 1.95E7 lb-ft

<u>Calculate External Work</u>

The external work, W_e, from the application of 1 lb/ft² is: 470.7 lb-ft

<u>Calculate Maximum Flexural Resistance, R_{UF}</u>

The maximum pressure resistance, R_{UF}, is:

$R_{UF} = (W_i/W_e) \times 1\ \text{lb/ft}^2$

$R_{UF} = (1.95\text{E}7\ \text{lb-ft}) / (470.7\ \text{lb-ft}) \times 1\ \text{lb/ft}^2 = 41{,}428\text{lb/ft}^2$

$F_{UF} = 41{,}428\ \text{lb/ft}^2 \times (35.3\ \text{ft}) \times (40.0\ \text{ft}) = 58.50\text{E}6\ \text{lb}$

<u>Calculate Elastic Resistance, R_e</u>

Several methods exist for estimating the elastic resistance of panels with various boundary conditions. For this example panel, the elastic resistance R_e is assumed to be equal to the pressure at which hinges form at the panel boundary. Alternative methods for estimating elastic resistance are provided in Biggs [21].

The internal work corresponding to the formation of hinges at the panel boundary is defined as W_{BOUND}. The elastic resistance of the panel is related to the ultimate panel resistance, R_{UF}, by the ratio of W_{BOUND} and the total internal work of the panel, W_I:

$R_e = (W_{BOUND} / W_i) \times R_{UF}$

$R_e = (2.17E6 \text{ lb-ft} / 1.95E7 \text{ lb-ft}) \times 41{,}428 \text{ lb/ft}^2 = 4{,}610 \text{ lb/ft}^2$

$F_e = (4{,}610 \text{ lb/ft}^2) \times (35.3 \text{ ft}) \times (40 \text{ ft}) = 6.51E6 \text{ lb}$

C3.4.2 Panel Shear Strength

The floor panel out-of-plane shear strength is based on an average shear force over the length of each edge. As the panel is designed as a two-way slab, the critical section for shear is assumed to be at a distance equal to the slab depth. The uniform pressure shear strength, R_{USL}, in the longitudinal direction is defined as:

$$R_{USL} = \frac{V_u L_x}{A_{TL}}$$

Where V_u = ultimate shear strength (Section C3.3.2)

L_x = panel length in longitudinal direction

A_{TL} = tributary pressure area on longitudinal edge (Figure C-7)

$$A_{TL} = \frac{(L_y - 2d)}{2} \, x \, (L_x - \mathrm{A})$$

Where A = distance to yield line intersection (ref figure C-7)

The uniform pressure shear strength, R_{UST}, in the transverse direction is defined as:

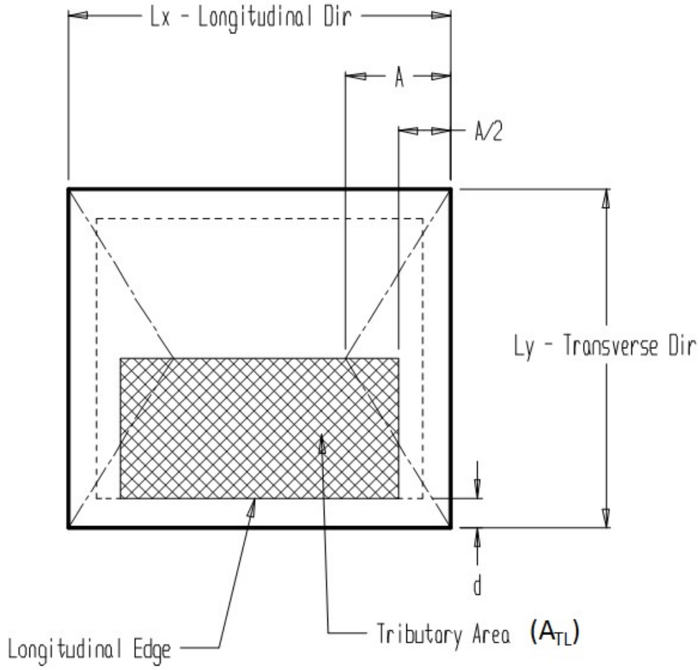
$$R_{UST} = \frac{V_u L_y}{A_{TT}}$$

Where V_u = ultimate shear strength (Section C3.3.2)

L_y = panel length in transverse direction

A_{TT} = tributary pressure area on transverse edge (Figure C-7)

$$A_{TT} = \frac{L_y (L_x - 2d)}{4}$$



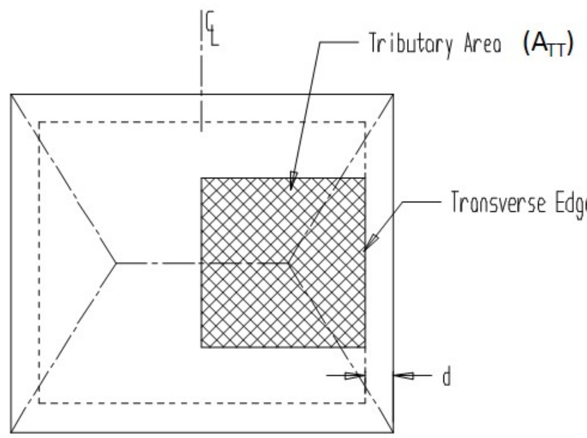


Figure C-7
Schematic Indicating Approximate Tributary Areas

Calculate tributary areas, A_{TL} and A_{TT}:

$$A_{TL} = \frac{(35.3\,ft - 12.17\,ft)}{2}\ x\ \left(40.0\,ft\ - \frac{40.0\,ft}{2}\right)$$

$A = {}^{L_x}/_2$ because all diagonal yield lines meet at center for this example.

A_{TL} = 231.3 ft^2

$$A_{TT} = \frac{35.3\ ft\ (40.0\ ft\ - 12.17\ ft)}{4}$$

A_{TT} = 245.6 ft^2

Calculate pressure capacities, R_{USL} and R_{UST}:

R_{USL} = (94,185 lb/ft) (40.0 ft) / (231.3 ft^2)

R_{USL} = 16,286 lb/ft^2

R_{UST} = (94,185 lb/ft) (35.3 ft) / (245.6ft^2)

R_{UST} = 13,536 lb/ft^2

Limiting shear pressure strength, R_{US}, is therefore:

R_{US} = 13,536 lb/ft^2

F_{US} = 13,536 lb/ft^2 x (35.3 ft) x (40.0 ft) = 19.11E6 lb

C3.5 Controlling Behavior (Flexure or Shear)

F_{UF} = 58.50E6 lb (Flexure)

F_{US} = 19.11E6 lb (Shear)

As $F_{US} < F_{UF}$, shear controls the design of the floor panel

C3.6 Floor Panel Stiffness

This section describes the approach for estimating panel stiffness. Section C3.6.1 is based on the assumption that the panel will achieve its ultimate flexural strength. However, if shear pressure strength (R_{US}) is less than the flexural pressure strength (R_{UF}), then the approach discussed in Section C3.6.2 should be used to estimate panel stiffness.

C3.6.1 Flexural Stiffness

An approximate relationship for determining panel stiffness is provided in [23].

$$K_e = \frac{1}{\gamma} x\ \frac{D}{{L_y}^4}$$

Where 1/γ is interpolated below and D is the flexural rigidity of the panel:

$$D = \frac{Ed^3}{12(1-\nu^2)}$$

$$E = 57{,}000\sqrt{f'_c}$$

E = 3.60E6 psi (5.19E8 lb/ft^2) ; d = 73 in (6.08 ft) ; ν = 0.15

D = 9.96E9 lb-ft

Reduce D a factor of 2.0 to account for cracking in accordance with ASCE 43-05 [5]

D = 4.98E9 lb-ft

L_y = 35.3 ft ; Lx = 40 ft

L_y/L_x = 35.3 ft / 40 ft = 0.88

Stiffness tables for rectangular plates with various edge conditions can be found in various references [21, 23]. For this example, Army TM-855 [23] was utilized to estimate floor slab stiffness. The tables in TM-855, which provide estimates of stiffness for plates with various aspect ratios, were curve-fit for the few cases analyzed herein. For the case of a panel fixed on all four sides, the elastic stiffness is approximated as:

<u>Calculate Elastic Stiffness, K_e</u>

$$K_e = (190.06e^{1.749\,\alpha})\ x\ \frac{D}{{L_y}^4}$$

Where α= L_y/L_x = 0.88 L_y = 35.3 ft D = 4.98E9 lb-ft

K_e = 2.85E6 lb/ft^2/ft (237,851 lb/ft^2 /in)

K_e = (237,851 lb/ft^2/in) x (35.3 ft) x (40 ft) = 33.58E7 lb/in

<u>Calculate Elastic displacement, Y_e</u>

$Y_e = F_e/K_e$

Y_e = (6.51E6 lb) / (33.58E7 lb/in) = 0.02 in

For the case of the panel fixed on all four sides, the approximate elastic-plastic stiffness is:

<u>Calculate Elastic-Plastic Stiffness, K_{ep}</u>

$$K_{ep} = (44.39e^{1.793\alpha})\ x\ \frac{D}{{L_y}^4}$$

Where $\alpha = L_y/L_x = 0.88$; Ly= 35.3 ft ; D= 4.98E9 lb-ft

K_{ep} = 693,018 lb/ft²/ft (57,751 lb/ft²/in)

K_{ep} = (57,751 lb/ft²/in) x (35.3 ft) x (40 ft) = 81.55E6 lb/in

Calculate Elastic-Plastic Displacement, Y_{ep}

$$Y_{ep} = Y_e + \frac{(F_{UF} - F_e)}{K_{ep}}$$

Y_{ep} = 0.66 in

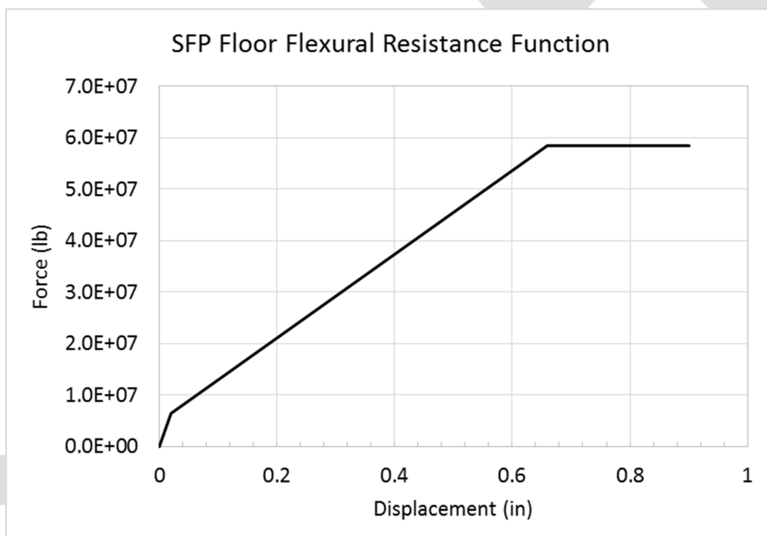


Figure C-8
SFP Floor Flexural Resistance Function

Equivalent System Parameters

As shown in Figure C-1, the panel stiffness is approximated as a bilinear force-displacement relationship. An equivalent linear system, as described in [21], can be developed by equating the area under the bilinear resistance function (shown as K_e and K_{ep}) to a single linear function, K_E. Figure C-9, below, compares the equivalent linear and bi-linear resistance functions.

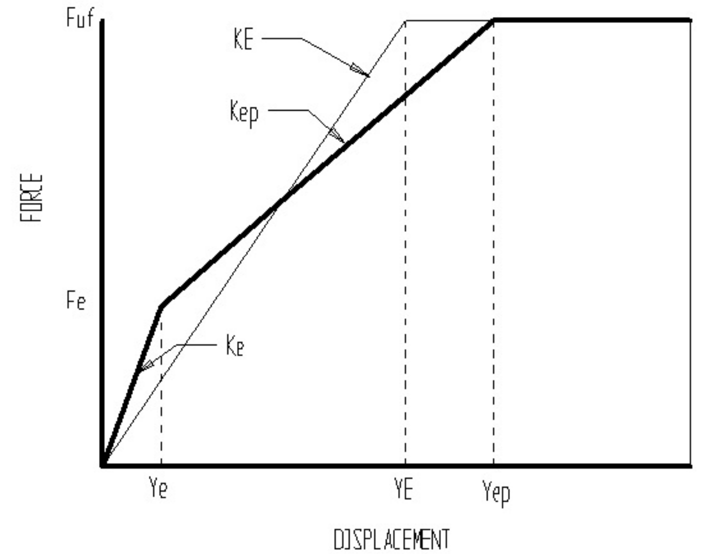


Figure C-9
Comparison of equivalent linear resistance function (K_E) to bi-linear resistance (K_e and K_{ep})

Area of bilinear resistance curve, A_{BL}, is equal to:

$$A_{BL} = \frac{1}{2} F_e\, Y_e + F_e\left(Y_{ep} - Y_e\right) + \frac{1}{2}\left(F_{UF} - F_e\right)\left(Y_{ep} - Y_e\right)$$

A_{BL}= 2.1E7 lb-in (Area under bilinear resistance curve)

Solve for an equivalent system displacement, Y_E:

$$\frac{1}{2} F_{UF}\, Y_E + F_{UF}\left(Y_{ep} - Y_E\right) = A_{BL}$$

$$Y_E = 2\,\frac{\left(F_{UF} Y_{ep} - A_{BL}\right)}{F_{UF}}$$

Y_E= 0.60 in

<u>Calculate Equivalent Stiffness, K_E</u>

$K_E = F_{UF}/Y_E$

K_E = (58.50E6 lb)/(0.60 in) = 9.70E7 lb/in

C3.6.2 Equivalent Stiffness of a Shear Controlled Panel

In cases where the panel strength is controlled by shear rather than flexure, the equivalent stiffness should be based on the ultimate shear strength, F_{US}. The equivalent stiffness based on shear, K_{ES}, is equal to the secant modulus at the peak panel shear strength, F_{US}.

$$K_{ES} = \frac{F_{US}}{Y_e + \dfrac{F_{US} - F_e}{K_{ep}}}$$

K_{ES} = 1.10E8 lb/in

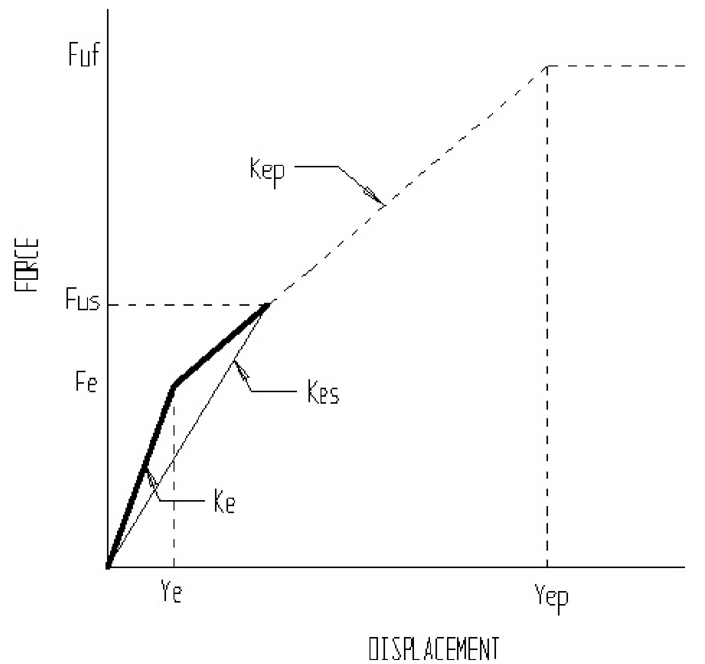


Figure C-10
Equivalent linear resistance function (K_{ES}) for a shear controlled panel

C3.7 Calculation of Panel Frequency

<u>Calculate Floor Slab Fundamental Frequency</u>

Table C-4
SFP component weights

Component / Item	Weight (lb)	Notes
Floor slab weight	1.3E6	
Fuel Assemblies and Racks	2.4E6	Consistent with NRC scoping study assumption of 2.4E6 lb (1,700 psf).
Water	3.3E6	Assumes 1.5 ft freeboard and all water participating
Total	7.0E6	Consistent with NRC scoping study assumption of 6.9E6 lb

Total mass = M_t = (7.00E6 lb) / (386 in/sec^2)

M_t = 18,135 lb-sec^2/in

Given that the floor slab is controlled by shear strength (rather than flexure), a load-mass factor of 0.68, consistent with elastic response, is utilized in accordance with Table 5.4 of Biggs [21]

Load-mass factor , K_{LM} = 0.68 (Biggs [21], Table 5.5)

Assume K_E = K_{ES} (since shear strength controls)

System Period, T = $2\pi \sqrt{\frac{K_{LM}\ M_t}{K_E}}$

T = 0.067 sec

Panel frequency = 1/T = 15.0 Hz

For selecting seismic spectral accelerations, consider peak broadening ± 15%. Chose three frequencies, 12.8 Hz, 15.0 Hz, and 17.3 Hz.

C3.8 Pressure Demands

Calculate floor slab, Rack, Assembly Inertia Force; F_{floor}

In cases where the floor design is limited by shear strength, the seismic acceleration demands consistent with 5% damping are appropriate. In cases where the design is limited by flexural response, and where increased cracking is expected, demands based on 7% damping are appropriate.

In this example, the floor slab design is controlled by shear strength. The vertical ISRS (5% damped) at the elevation of the SFP floor elevation is shown as the red curve (ref Figure C-11). Using the broadened frequency range (12.8 Hz and 17.3 Hz), the approximate peak vertical spectral acceleration is 2.3g. A load-mass factor of 0.68, consistent with elastic response, is utilized in accordance with Table 5.4 of Biggs [21].

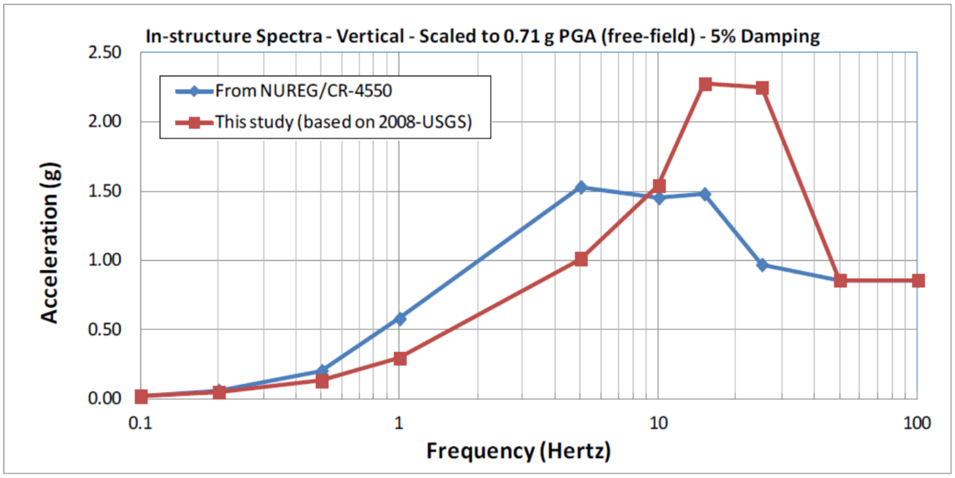


Figure C-11
Maximum vertical spectral acceleration value, at 16.9 Hz, assumed in calculating floor inertial forces

Calculate Floor Inertial Pressure, E_{FI}

$E_{FI} = (W_{slab} + W_{assemblies+racks}) \times (2.3\ g) \times 0.68$

$E_{FI} = (1.3E6\ lb + 2.4E6\ lb) \times (2.3g) \times 0.68$

$E_{FI} = (5.79E6\ lb)/(35.3\ ft \times 40\ ft) = 4{,}100\ lb/ft^2$

Calculate Hydrodynamic Floor Pressure, E_{FH}

The vertical hydrodynamic floor pressure is assumed to equal the mass of the water accelerating confined within the SFP (3.3E6 lb). The spectral acceleration assumed is equal to the worst-case acceleration values of 12.8 Hz, 15.0 Hz, and 17.3 Hz, (which represent ± 15% peak broadening). This corresponds to a 5% damped spectral acceleration of approximately 2.3g. Based on this value of spectral acceleration, the vertical hydrodynamic pressure is:

$E_{FH} = (3.3E6\ lb \times 2.3g)/(35.3\ ft \times 40\ ft) = 5{,}375\ lb/ft^2$

The vertical pressure, E_{FV}, acting on the floor slab is equal to:

$E_{FV} = E_{FI} + E_{FH}$

$E_{FV} = 4{,}100\ lb/ft^2 + 5{,}375\ lb/ft^2 = 9{,}475\ lb/ft^2$

Hydrodynamic wall pressures are derived in Section C4.8 of this report. The hydrodynamic wall pressures are calculated using ACI guidance pertaining to concrete tank structures [16].

The horizontal wall pressure, which has a maximum value of 1,939 psf in the EW direction (ref Table C-13), is conservatively applied as a uniform pressure on the SFP floor. The contribution of load to the SFP floor is:

<u>Summary of Hydrodynamic Demands (Table C-13)</u>

NS hydrodynamic pressure	1,712 lb/ft^2 (bottom elevation)
EW hydrodynamic pressure	1,939 lb/ft^2 (bottom elevation)

The controlling hydrodynamic pressure acting on the SFP wall is:

E_{WH} =1,939 lb/ft^2

This pressure is assumed to vary as a cosine wave with maximum amplitude at the wall face and zero amplitude at the mid-span of the pool (x=20 ft in this case). This is consistent with the assumption that half the water mass is participating for each wall.

$$E_{WH}(x) = E_{WH}\ \cos\left(\frac{\pi x}{2L}\right)$$

where L=SFP width (corresponding to controlling direction)

At the distance of 6.08 ft from the face of the SFP wall (corresponding to the floor slab critical section), the horizontal pressure is:

$$E_{WHcr} = 1{,}939\ psf\ \left[\cos\left(\frac{\pi}{2}\ \frac{6.08\ ft}{40\ ft}\right)\right]$$

E_{WHcr} = 1,884 psf

Combined vertical seismic pressure using 100-40-40 rule:

$E_{FC} = E_{FV} + 0.4(E_{WHcr})$

E_{FC} = 9,475 lb/ft^2 + 0.4(1,884) lb/ft^2)

E_{FC} = 10,228 lb/ft^2 (assumed uniform pressure distribution)

<u>Calculate Dead Load, D_L</u>

D_L = 7.0E6 lb / (35.3 ft x 40 ft) = 4,958 lb/ft^2

<u>Total Pressure Demand</u>

$U_{TOT} = E_{FC} + D_L$

U_{TOT} = 10,228 lb/ft^2 + 4,958 lb/ft^2 = 15,186 lb/ft^2

C3.9 Slab Displacement and Flexural Margin

SFP Floor Displacement

The vertical displacement of the floor due to the total pressure demand is:

$\Delta_v = U_{TOT}/K_{ES}$

U_{TOT} = 15,186 lb/ft² x (40.0 ft) x (35.3 ft) = 21.44E6 lb

Δ_v = (21.44E6 lb) / (1.10E8 lb/in) = 0.19 inches

The panel displacement is more than 0.19 inches, which exceeds the estimated elastic displacement of 0.02 inches. Thus, there will likely be concrete cracking at the panel supports. However, due to the small panel displacements, these cracks will have small widths, and shear forces will still be resisted by the concrete aggregate. The flexural demand-to-capacity ratio is approximately 0.37, which indicates that the reinforcement at the center of the panel is not significantly challenged. It should be noted that for this panel, which has no assumed shear reinforcement, the shear limit state would be reached long before the flexural strength could be realized.

C3.10 Margin Factors

The strength factor, Fs, is equal to:

$$F_s = \frac{C - D_{NS}}{D_s}$$

Where

C = limiting panel pressure strength

D_{NS} = non-seismic demands (e.g., dead loads)

D_S= seismic demands

$$F_s = \frac{13{,}536\, psf - 4{,}958\, psf}{10{,}228\, psf}$$

F_S = 0.83

Table C-5
Margin factors assumed to calculate floor HCLPF

Margin Factors	
F_m; material margin factor	1

F_{am}; analysis method	1
F_{ca}; code allowable	1
F_{ma}; margin to code allowable	1
F_d; damping	1
F_{iea}; inelastic energy absorption	1
F_s; strength factor	0.83
F_{sm}; margin increase factor	0.83

Based on Table C-5 information, and the observation that the subject site has a GMRS PGA of 0.7g and a peak SA of 1.6g in the 10-20 Hz range (Figure C-18), the spent fuel pool floor has a HCLPF of 0.58g PGA and 1.33g $SA_{10\text{-}20}$ (0.83 x 1.6g).

The HCLPF capacity of the SFP is defined at approximately 95% confidence of less than about 5% probability of failure. As long as β_r/β_u lies in the range of 0.5 to 2.0, this HCLPF capacity can closely be approximated by the mean 1% failure probability capacity. As the NRC SFP Scoping Study is based on mean response and material parameters, and the analysis did not result in structural failure, the 1% failure probability capacity is approximately 0.7g PGA. This capacity approximately 20% higher than the HCLPF PGA capacity of 0.58g calculated in this appendix.

C3.11 Comparison to NRC SFP Analysis

Comparison of results between the NRC Scoping Study and the SFP Analytical Model were performed. The results of the SDOF model, for SFP floor response, compared well to those described in the NRC Scoping Spent Fuel Pool Scoping Study. Section C6.0 of this report describes the comparisons.

C4.0 SFP Wall Evaluation

The critical wall span is assumed to be long-span wall with a span of 40 feet (Figure C-12). The thickness of the wall varies with height from 60 inches to 72 inches. For the purpose of this calculation, the thickness is assumed to be the average, or 66 inches.

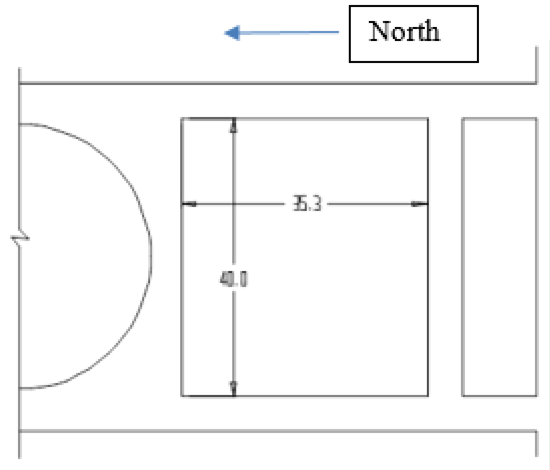


Figure C-12
Plan view of SFP showing wall spans

C4.1 Materials

The material properties for the SFP wall are assumed to be same as those used for the floor.

Table C-6
Assumed Material Properties

Design Parameter	Value Used For This Calculation
Concrete Density	150 lb/ft^3
Nominal Concrete Strength	4,000 psi
Rebar Strength (Grade 60)	60,000 psi
Young’s Modulus (un-cracked)	3.6E6 psi

C4.2 Concrete Reinforcement

Steel reinforcement ratios were estimated based on the SFP design drawings. Table C-7 provides a summary of the reinforcement design:

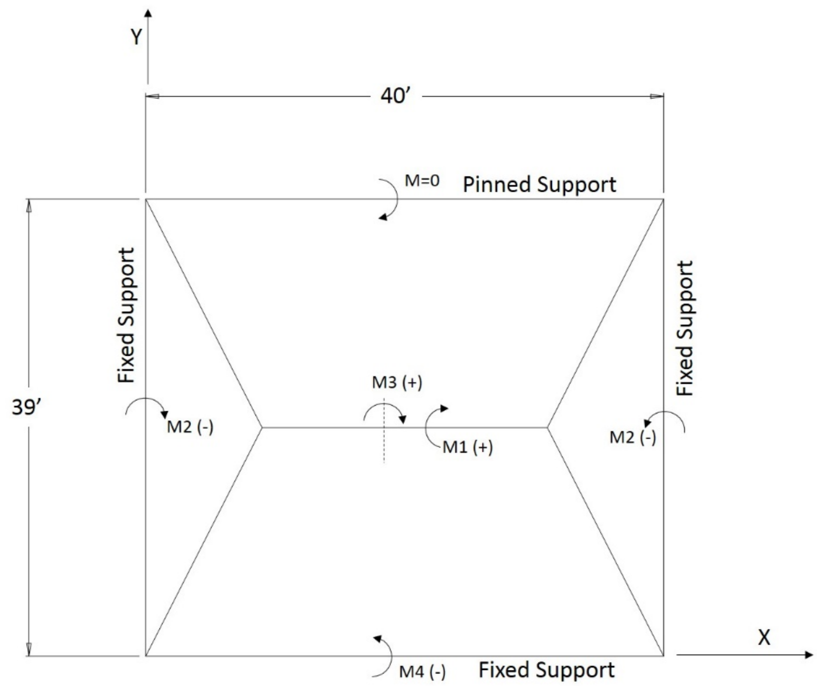


Figure C-13
Example yield line pattern indicating assumed wall boundary conditions and locations of (+) and (-) flexural reinforcement. Reinforcement per Table C-7.

Table C-7
Panel Reinforcement Assumptions (Based on Design Drawings)

Panel Location	Reinforcement	Reinforcement Ratio
M1; Total Positive Moment at Center	#11@9"	0.0025
M2; Total Negative Moment at Ctr of Side Edge	#11@12"	0.0019
M3; Total Positive Moment at Center	#11@9"	0.0025
M4; Total Negative Moment at Ctr of Bottom Edge	(3) #11@9" bundled	0.010

C4.3 Wall Panel Section Strength

Both flexure and shear section strength is calculated for each panel.

C4.3.1 Section Flexural Strength

The nominal panel flexural strength can be calculated as:

$$M_n = \rho f_y b d^2 \left(1 - 0.59 \frac{\rho f_y}{f'_c}\right) \text{ (Nilson, Winter) [22]}$$

Factored strength, $M_u = \Phi M_n$; $\Phi = 0.9$ [24]

Table C-8
Panel Ultimate Flexural Capacities

Panel Location	Total Factored Strength (Mu); Φ=0.9	Factored Strength (Mu) per unit length
M1; Total Positive Moment at Center	2.60E8 lb-in	5.41E5 lb-ft/ft
M2; Total Negative Moment at Ctr of Side Edge	1.93E8 lb-in	4.13E5 lb-ft/ft
M3; Total Positive Moment at Center	2.53E8 lb-in	5.41E5 lb-ft/ft
M4; Total Negative Moment at Ctr of Bottom Edge	9.68E8 lb-in	2.02E6 lb-ft/ft

C4.3.2 Section Shear Strength

The nominal shear strength of the panel can be calculated from ACI 349 [24]

V_n = 2x (4,000 psi)$^{0.5}$ x 64 in x 12 in = 97,145 lb/ft

$V_u = \Phi V_n$; Φ=0.85 for shear

V_u = 82,573 lb/ft

C4.4 Panel Strength

Both flexure and shear strength is calculated for each panel to confirm the limiting failure mode.

C4.4.1 Flexural Strength

The approach to estimate the pressure capacity of the wall panel is based on a yield line analysis. Figure C-13 shows an example yield line pattern. The assumption is that internal work is equivalent to the external work caused by the pressure [22]. The approach is similar to that described for the SFP floor slab (Section C3.4.1). The angle of the diagonal and the offset of the horizontal yield line were varied until the highest internal moment was obtained. From this analysis, an angle of 65 degrees maximized the required internal moment. The corresponding offset of the horizontal yield line from the bottom of the SFP is approximately 23.5 feet. Based on this, the work for the supports was computed to be:

<u>Calculate Internal Work</u>

Side supports 2.15E6 lb-ft

Bottom support 3.46E6 lb-ft

Top support 0.0 lb-ft (assumed no moment capacity)

On this basis, it is assumed that the side supports will yield prior to the bottom support.

The total internal work, W_i, is: 9.88E6 lb-ft

Calculate External Work

The external work, W_e, from the application of 1 lb/ft^2 is: 624.2 lb-ft

Calculate Maximum Resistance, R_{UF}

The maximum pressure resistance is:

$R_{UF} = (W_i/W_e) \times 1$ lb/ft^2

R_{UF} = (9.88E6 lb-ft) / (624.2 lb-ft x 1 lb/ft^2) = 15,828 lb/ft^2

F_{UF} = 15,828 lb/ft^2 x (39 ft) x (40 ft) = 24.69E6 lb

Calculate Elastic Resistance, R_e

$R_e = (W_{BOUND}/ W_i) \times R_{UF}$

R_e= (2.15E6 lb-ft / 9.88E6 lb-ft) x 15,828 lb/ft^2 = 3,444 lb/ft^2

F_e= (3,444 lb/ft^2) x (39 ft) x (40 ft) = 5.37E6 lb

C4.4.2 Shear Strength

The wall panel out-of-plane shear strength is based on an average shear force over the length of each edge. As the panel is designed as a two-way wall panel, the critical section for shear is assumed to be at a distance equal to the panel depth. The uniform pressure shear strength, R_{USL}, in the longitudinal direction is defined as:

$$R_{USL} = \frac{V_u\, L_x}{A_{TL}}$$

Where V_u = ultimate shear strength (Section C4.3.2)

L_x = panel longitudinal direction

A_{TL} = tributary pressure area on longitudinal edge (Figure C-7)

$$A_{TL} = \frac{(L_y - 2d)}{2}\ x\ (L_x - A)$$

Where A = distance to yield line intersection (ref figure C-7)

The uniform pressure shear strength, R_{USL}, in the transverse direction is defined as:

$$R_{UST} = \frac{V_u L_y}{A_{TT}}$$

Where V_u = ultimate shear strength (Section C4.3.2)

L_x = panel longitudinal direction

A_{TT} = tributary pressure area on transverse edge (Figure C-7)

$$A_{TT} = \frac{L_y(L_x - 2d)}{4}$$

<u>Calculate tributary areas, A_{TL} and A_{TT}:</u>

For the wall panel;

L_x = 40.0 ft

L_y = 39.0 ft

d = 5.3 ft

A = 23.5 ft x $\tan^{-1}(65°)$ = 10.9 ft

A_{TL} = 411.5 ft^2

A_{TT} = 286.0 ft^2

<u>Calculate pressure capacities, R_{USL} and R_{UST}:</u>

R_{USL} = (82,573 lb/ft) (40.0 ft) / (411.5 ft^2)

R_{USL} = 8,026 lb/ft^2

R_{UST} = (82,573 lb/ft) (39.0 ft) / (286.0 ft^2)

R_{UST} = 11,260 lb/ft^2

Limiting shear pressure strength, R_{US}, is therefore:

R_{US} = 8,026 lb/ft^2

F_{US} = 8,026 lb/ft^2 x (39.0 ft) x (40.0 ft) = 12.52E6 lb

C4.5 Controlling Behavior (Flexure or Shear)

F_{UF} = 24.70E6 lb (Flexure)

F_{US} = 12.52E6 lb (Shear)

As $F_{US} < F_{UF}$, shear controls the design of the wall panel

C4.6 Wall Flexural Stiffness

This section describes the approach for estimating panel stiffness. Section C4.6.1 is based on the assumption that the panel will achieve its ultimate flexural strength. However, if shear strength is less than the flexural strength, then the approach discussed in Section C4.6.2 should be used to estimate panel stiffness.

C4.6.1 Flexural Stiffness

<u>Characterize Stiffness of the Wall Panel</u>

Flexural Rigidity, D

$$D = \frac{Eh^3}{12(1-\nu^2)}$$

$$E = 57{,}000\sqrt{f'_c}$$

E = 3.60E6 psi (518.4E6 lb/ft²) ; h = 64 in (5.3 ft) ; ν = 0.15

D = 6.71E9 lb-ft

Reduce Rigidity by a factor of 2.0 in accordance with [5]

D= 3.36E9 lb-ft

L_y = 39 ft ; L_x = 40 ft

L_y/L_x = 39 ft / 40 ft = 0.98

<u>Calculate Elastic Stiffness, K_e</u>

In the case of a panel with fixed supports on three sides and pinned at top, the elastic stiffness can be approximated as:

$$K_e = (98.841e^{2.054\,\alpha})\ x\ \frac{D}{{L_y}^4}$$

Where $\alpha = L_y/L_x$ = 0.98

K_e = 1.06E6 lb/ft²/ft (88,547 lb/ft²/in)

K_e = (88,547 lb/ft²/in) x (39.0 ft) x (40.0 ft) = 138.1E6 lb/in

<u>Calculate elastic displacement, Y_e</u>

$Y_e = F_e/K_e$

Y_e = (5.37E6 lb) / (138.1E6 lb/in) = 0.04 in

Calculate Elastic-Plastic Stiffness, K_{ep}

In this case, the panel elastic-plastic stiffness can be approximated as:

$$K_{ep} = (44.394e^{1.794\alpha}) \; x \; \frac{D}{{L_y}^4}$$

K_{ep} = 3.70E5 lb/ft²/ft (30,832 lb/ft²/in)

K_{ep} = (30,832 lb/ft²/in) x (39 ft) x (40 ft) = 48.10E6 lb/in

Calculate Elastic-Plastic Displacement, Y_{ep}

$$Y_{ep} = Y_e + \frac{(F_{UF} - F_e)}{K_{ep}}$$

Y_{ep} = 0.44 in

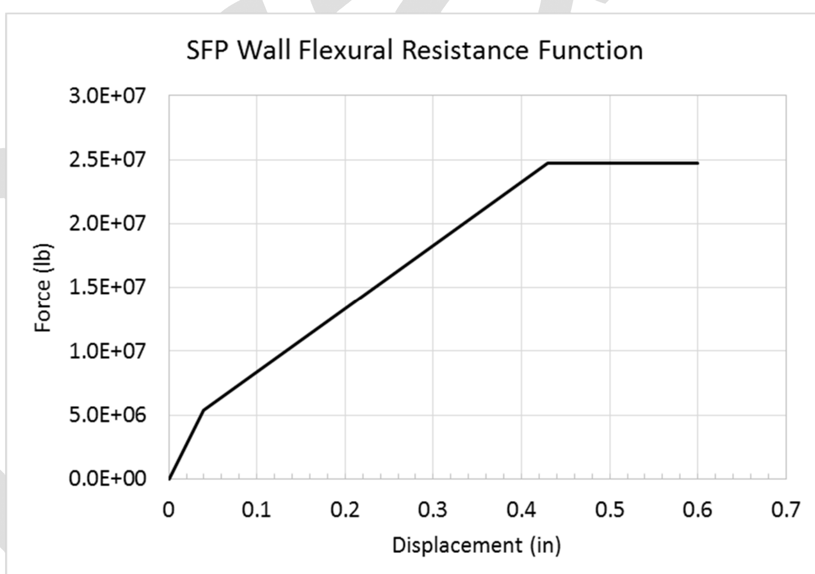


Figure C-14
SFP Wall Resistance Function

Equivalent System Parameters

The bilinear system shown in Fig C-14 is idealized as an equivalent linear system by equating the area under the bilinear resistance curve (ref Figure C-9):

Area of bilinear resistance curve, A_{BL}, is equal to:

$$A_{BL} = \frac{1}{2} F_e Y_e + F_e\left(Y_{ep} - Y_e\right) + \frac{1}{2}\left(F_{UF} - R_e\right)\left(Y_{ep} - Y_e\right)$$

A_{BL} = 6.14E6 lb-in (Area under bilinear resistance curve)

Solve for an equivalent system displacement, Y_E:

$$\frac{1}{2} F_{UF} Y_E + F_{UF}\left(Y_{ep} - Y_E\right) = A_{BL}$$

$$Y_E = 2\frac{\left(F_{UF} Y_{ep} - A_{BL}\right)}{F_{UF}}$$

Y_E= 0.38 in

Calculate Equivalent Stiffness, K_E

$K_E = F_{UF}/Y_E$

K_E = 64.37E6 lb/in

C4.6.2 Equivalent Stiffness of a Shear Controlled Panel

In cases where the panel strength is controlled by shear rather than flexure, the equivalent stiffness should be based on the ultimate shear strength, R_{US}. The equivalent stiffness based on shear, K_{ES}, is equal to the secant modulus at the peak panel shear strength, R_{US}. This approach is similar to Section C3.6.2.

$$K_{ES} = \frac{F_{US}}{Y_e + \dfrac{F_{US} - F_e}{K_{ep}}}$$

K_{ES} = 66.78E6 lb/in

C4.7 Frequency

Calculate Wall Panel Fundamental Frequency

Table C-9
Panel Weights

Component / Item	Weight (lb)	Notes
Wall Weight	1.3E6	
Water	1.7E6	Assumes 1.5 ft freeboard and 50% water participating on the wall
Total	3.0E6	

Total mass = M_t = (3.0E6 lb) / (386 lb-sec²/in)

M_t = 7,772 lb-sec²/in

Given that the floor slab is controlled by shear strength (rather than flexure), a load-mass factor of 0.63, consistent with elastic response, is utilized in accordance with Table 5.4 of Biggs [21]

Load-mass factor, K_{LM} = 0.63 (Biggs, Table 5.5)

System Period, $T = 2\pi \sqrt{\frac{K_{LM}\ M_t}{k_E}}$

Assume $K_E = K_{ES}$ = 66.78E6 lb/in (since shear strength controls)

T = 0.054 sec

Panel frequency = 1/T = 18.6 Hz

For selecting seismic spectral accelerations, consider peak broadening ± 15%. Chose three frequencies, 15.8 Hz, 18.6 Hz, and 21.4 Hz.

C4.8 Pressure Demands

Calculate Wall Inertia Pressure; E_{WI}

The approach for estimating the seismic response of the wall is to take the average of the horizontal accelerations at the top and bottom SFP elevations. However, the SFP Scoping report provides a 5% horizontal FRS at the mid-height of the wall (ref Figure C-15); which is a more direct indication of response. For this calculation, the zero period acceleration value of 1.25g was assumed. A load factor of 0.63, consistent with elastic response, is utilized in accordance with Table 5.4 of Biggs [21].

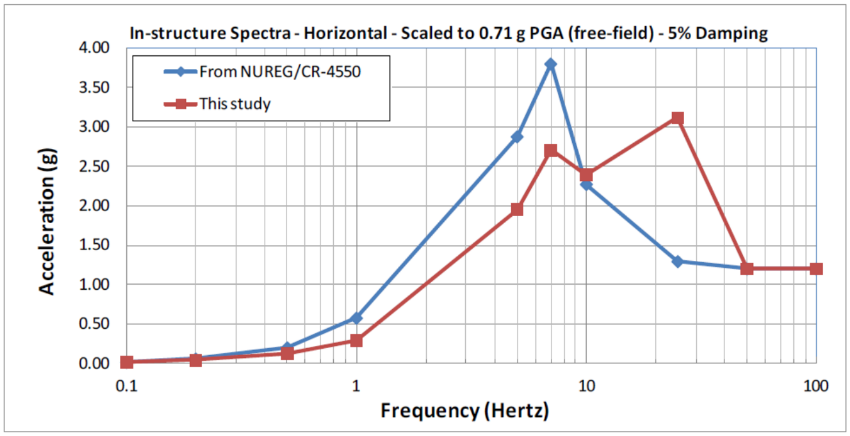


Figure C-15
Horizontal spectral acceleration at mid-height of SFP

Calculate Wall Inertial Pressure, E_{WI}

Wall weight: 1.30E6 lb

E_{WI} = (W_{wall} x 1.25g x 0.63) / (39 ft x 40 ft)

E_{WI} = 653 lb/ft^2

Calculation of Hydrodynamic Wall Pressure, E_{WH}

The hydrodynamic wall pressures are calculated using ACI 350.3/350.3R-15 [16]. This ACI standard provides guidance for the seismic analysis and design of liquid-containing concrete structures. The document identifies key differences from traditional methodologies, namely: (1) instead of assuming a rigid tank directly accelerated by ground acceleration, the document assumes amplification of response due to flexibility of the tank, (2) combines impulsive and convective response by SRSS, (3) includes effects of vertical acceleration, and (4) includes an effective mass coefficient, applicable to the mass of the walls.

The first step in the calculation of hydrodynamic demands is to estimate the dynamic properties for the simplified dynamic model postulated in ACI-350. These properties are different for each direction of sloshing. The steps are to calculate inertial and convective masses, estimate heights to center of mass, calculate sloshing frequencies, resultant forces, and corresponding pressure distributions.

Impulsive and Convective Masses

The computation of equivalent impulsive and convective masses for rectangular tanks is performed in accordance with ACI-350, equations 9-1 and 9-2:

$$\frac{W_i}{W_l} = \frac{\tanh[0.866\left(\frac{L}{H_l}\right)]}{0.866(\frac{L}{H_l})}$$

$$\frac{W_c}{W_l} = 0.264\left(\frac{L}{H_l}\right)\tanh[3.16\left(\frac{H_l}{L}\right)]$$

Table C-10
Ratios of impulsive and convective masses for NS and EW SFP directions

Direction of Seismic Loading	L (ft)	H_l (ft)	W_i/W_l	W_c/W_l
NS	35.3	38	0.83	0.24
EW	40	38	0.79	0.28

The computation of height to centers of gravity for rectangular tanks is performed in accordance with ACI-350, equations 9-3 and 9-5:

Inertial mass center-of-gravity height

For tanks with $L/H_l < 1.33$:

$$\frac{h_i}{H_l} = 0.5 - 0.09375\,(\frac{L}{H_l})$$

For tanks with $L/H_l > 1.33$;

$h_i/H_l = 0.375$

Convective mass center-of-gravity height

For all tanks,

$$\frac{h_c}{H_l} = 1 - \frac{\cosh\left[3.16\left(\frac{H_l}{L}\right)\right] - 1}{3.16\left(\frac{H_l}{L}\right)\sinh\left[3.16\left(\frac{H_l}{L}\right)\right]}$$

Table C-11
Summary of impulsive and convective center of gravity heights

Direction of Seismic Loading	L (ft)	H_l (ft)	h_i/H_l	h_c/H_l
NS	35.3	38	0.41	0.72
EW	40	38	0.40	0.70

Sloshing Frequencies

Estimation of sloshing frequencies are performed in accordance with the SPID equation 7-1 [9]:

$f_c = (1/2\pi)\,[(3.16g\,/L_p)\tanh(3.16H_l/L_p)]^{0.5}$

where L_p = pool length in the direction of shaking

H_l= water depth

g = gravity

Table C-12
Summary of NS and EW sloshing frequencies

Direction of Seismic Loading	L_p (ft)	H_l (ft)	Sloshing Frequency (Hz)
NS	35.3	38	0.27
EW	40	38	0.25

Calculation of Resultant Impulsive Force (Horizontal)

$P_i = W_{tot} \text{ x } W_i/W_l \text{ x } Sa_{h/2}$

Where W_{tot} = total water mass

W_i/W_l ; impulsive mass fraction

$Sa_{h/2}$ = spectral acceleration at mid-height of the SFP (5% damped)

At the mid-height of the SFP, the zero period acceleration based on the 5% damped horizontal response spectra is approximately 1.25g (ref Figure C-15). This is the assumed impulsive acceleration of the SFP water in the horizontal direction. The assumed pressure distribution for impulsive loading is derived from ACI-350, Figure R5.3:

$$P_{iy} = \frac{\frac{P_i}{2}\left[4H_l - 6h_i - (6H_l - 12h_i)\,x\,\left(\frac{y}{H_l}\right)\right]}{H_l^{\,2}}$$

P_i = impulsive force

H_l = pool depth

h_i = center of gravity height of impulsive mass

y = height from bottom of SFP

Calculation of Resultant Convective Force (Horizontal)

$P_c = W_{tot}$ x W_c/W_l x Sa

Where W_{tot} = total water mass

W_i/W_l ; convective mass fraction

Sa = spectral acceleration at sloshing frequency (0.5% damped)

The spectral acceleration based on the 0.5% damped response spectra is 0.32g, based on the approximate scaling relationship: (scaled from Figure C-18).

$Sa_{0.5\%} = Sa_{5.0\%}$ x $(5.0/0.5)^{0.5}$

$Sa_{0.5\%} = Sa_{5.0\%}$ x 3.16

The maximum $Sa_{5.0\%}$ is approximately 0.1g (ref Figure C-18). Therefore, the $Sa_{0.5\%}$ = 0.316g. Convective pressures shown in Tables C-13 and C-14 are based on this spectral acceleration.

The assumed pressure distribution for convective loading is derived from ACI-350, Figure R5.3.

$$P_{cy} = \frac{\frac{P_c}{2}\left[4H_l - 6h_c - (6H_l - 12h_c)\ x\ \left(\frac{y}{H_l}\right)\right]}{{H_l}^2}$$

P_c = convective force

H_l = pool depth

h_c = center of gravity height of convective mass

y = height from bottom of SFP

Table C-13
Summary of inertial and convective pressures at bottom of SFP

Direction of Seismic Loading	Inertial Force (lb)	Convective Force (lb)	L (ft)	H_l (ft)	y (ft)	P_i (psf)	P_c (psf)
NS	3,417,693	258,401	40	38	0	1,712	0
EW	3,266,632	292,011	35.3	38	0	1,939	0

Table C-14
Summary of inertial and convective pressures at mid-height of SFP

Direction of Seismic Loading	Inertial Force (lb)	Convective Force (lb)	L (ft)	H_l (ft)	y (ft)	P_i (psf)	P_c (psf)
NS	3,417,693	258,401	35.3	38	19.5	1,256	89
EW	3,266,632	292,011	40	38	19.5	1,058	88

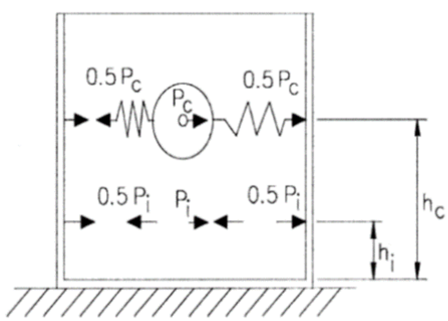


Figure C-16
SFP hydrodynamic model

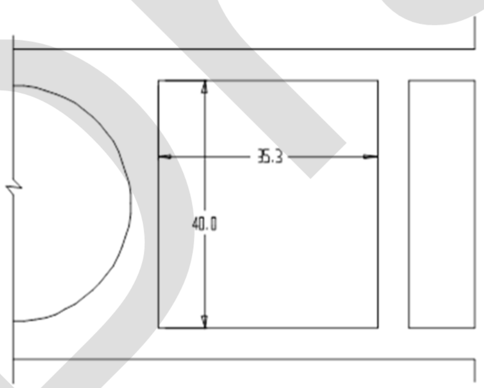


Figure C-17
Plan view of SFP showing sloshing directions

The horizontal hydrodynamic wall pressures are those values that are computed for the mid-height of the SFP wall (Table C-14).

<u>Summary of Hydrodynamic Wall Pressures (Table C-14)</u>

EW hydrodynamic wall pressure: 1,058 lb/ft²

NS hydrodynamic wall pressure: 1,256 lb/ft^2

The controlling hydrodynamic pressure, E_{WH}, is 1,256 lb/ft^2

The vertical hydrodynamic floor pressure, E_{FH}, is equal to:

E_{FH} = 5,375 lb/ft^2 / 2.0 = 2,687 lb/ft^2

(Reduced by 2.0 factor to account for hydrostatic variation).

The horizontal pressure, $E_{W,}$ acting on the wall is:

$E_W = E_{WI} + E_{WH}$

E_W = 653 lb/ft^2 + 1,256 lb/ft^2 = 1,909 lb/ft^2

The combined pressure, E_{WC}, is:

E_{WC} = 1,909 lb/ft^2 + 0.4(2,687 lb/ft^2)

E_{WC} = 2,983 lb/ft^2 (assumed uniform pressure distribution)

<u>Calculate Dead Load (Hydrostatic), D</u>

$D_L = \rho_{water}$ x H

ρ_{water} = 62.4 lb/ft^3; H = 38 ft

D_L = 2,371 lb/ft^2 ;

(Conservative in that maximum hydrostatic pressure is assumed to act uniformly on entire wall panel)

<u>Calculate Total Load, U_{TOT}</u>

$U_{TOT} = E_{WC} + D_L$

U_{TOT} = 2,983 lb/ft^2 + 2,371 lb/ft^2

U_{TOT} = 5,354 lb/ft^2

U_{TOT} (force) = 5,354 lb/ft^2 x (39.0 ft) x (40.0 ft) = 8.35E6 lb

C4.9 Displacement and Flexural Margin

The horizontal displacement of the wall due to the total pressure demand is:

$\Delta_v = U_{TOT}/K_{ES}$

Δ_v = (8.35E6 lb) / (66.78E6 lb/in) = 0.12 inches

Under the assumed pressure demand of 5,354 lb/ft^2, the panel displacement is 0.12 inches, which exceeds the estimated elastic displacement of 0.04 inches. Thus, there will likely be concrete cracking at the panel supports. However, due to the small panel displacements, these cracks will have small widths, and shear forces will still be resisted by the concrete aggregate. The flexural demand-to-capacity ratio is approximately 0.34, which indicates that the reinforcement at the center of the panel is not significantly challenged. It should be noted that for this panel, which has no assumed shear reinforcement, the shear limit state would be reached long before the flexural strength could be realized.

C4.10 Margin Factors

The strength factor, F_s, is equal to:

$$F_s = \frac{C - D_{NS}}{D_s}$$

Where

C = limiting panel pressure capacity

D_{NS} = non-seismic demands (e.g., dead loads)

D_S= seismic demands

$$F_s = \frac{8{,}026\, psf - 2{,}371\, psf}{2{,}983\, psf}$$

F_S = 1.89

Table C-15
Margin factors assumed to calculate wall HCLPF

Margin Factors	
F_m; material margin factor	1
F_{am}; analysis method	1
F_{ca}; code allowable	1
F_{ma}; margin to code allowable	1
F_d; damping	1
F_{iea}; inelastic energy absorption	1
F_s; strength factor	1.89
F_{sm}; margin increase factor	1.89

Based on Table C-15 information, and the observation that the subject site has a peak GMRS PGA of 0.7g and a peak SA of 1.6g in the 10-20 Hz range (Figure C-18), the spent fuel pool wall has a HCLPF of 1.32g PGA and 3.02g SA_{10-20} (1.89 x 1.6g).

C5.0 Provided Response Spectra

The below response spectra are provided in the NRC SFP Scoping Study [15], and are assumed as input for the floor and wall panel analyses described in this report.

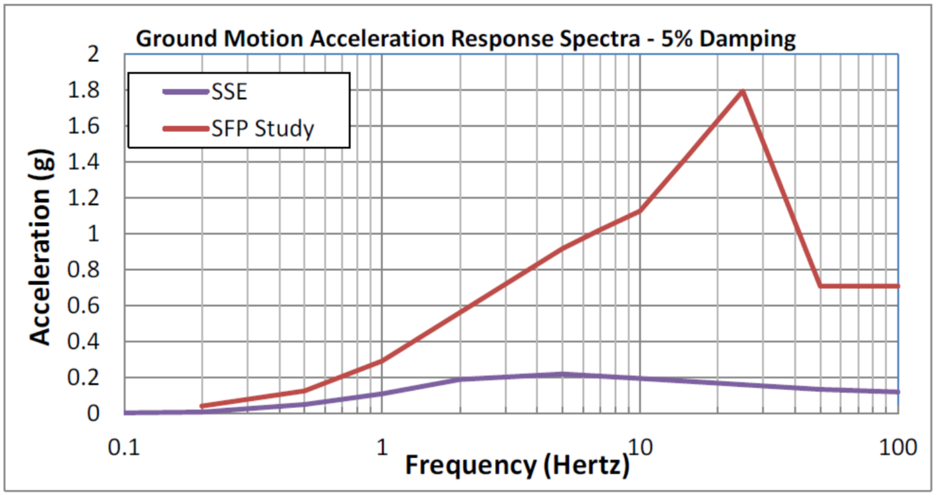


Figure C-18
NRC SFP Scoping Study foundation input response spectrum (Horizontal)

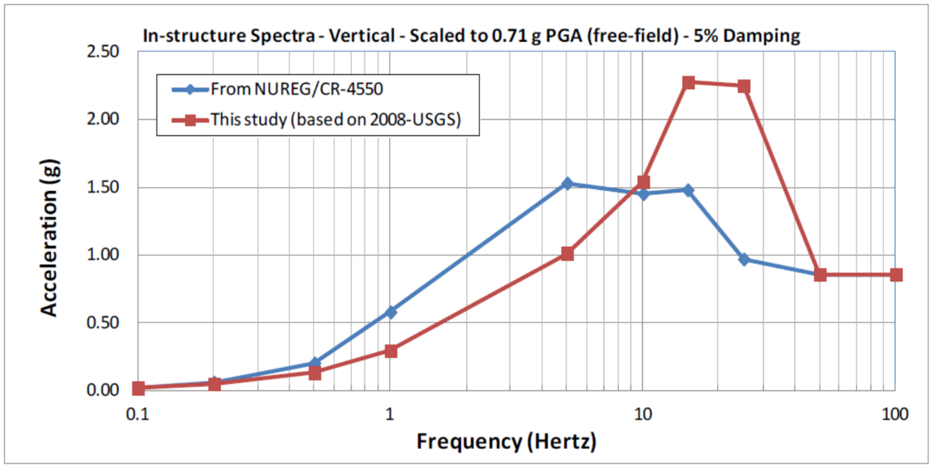


Figure C-19
Vertical ISRS at bottom elevation of SFP (NRC SFP Scoping Study)

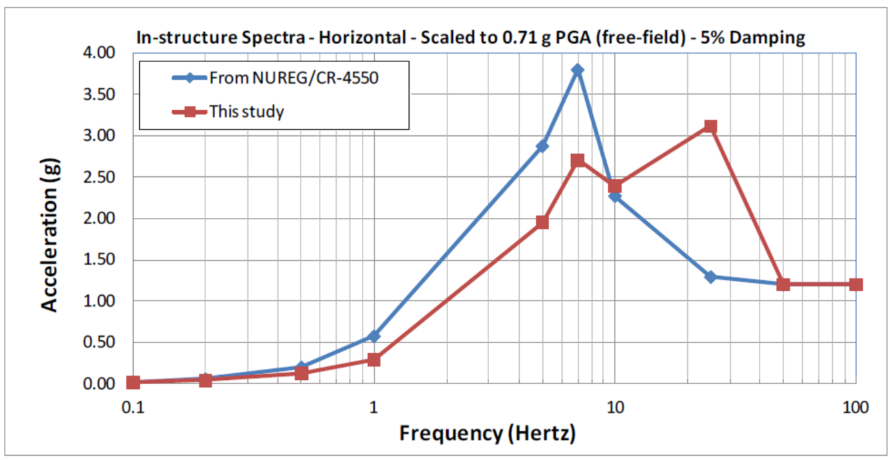


Figure C-20
Horizontal ISRS at mid-height of SFP (NRC SFP Scoping Study)

C6.0 Results

Using the Conservative Deterministic Fragility Method (CDFM), the HCLPF of the Peach Bottom Spent fuel pool was calculated to be 0.58g PGA and 1.33g $SA_{10\text{-}20}$. The assumed Seismic Margin Earthquake corresponds to the seismic ground motion described in the NRC Spent Fuel Pool Scoping Study [15]. The fragility of the pool structure is controlled by out-of-plane shear demands on the floor slab. The floor slab fundamental frequency and peak displacement were calculated to be 15.0 Hz and 0.19 in, respectively. These results compared well those described in the NRC Scoping Spent Fuel Pool Scoping Study (Table C-16). The SFP wall was also limited by out-of-plane shear strength. The wall is estimated to have a fundamental frequency of 18.6 Hz, a peak displacement of 0.12 in, and a HCLPF 1.32g PGA and 3.02g $SA_{10\text{-}20}$.

On the basis that the SFP seismic model is based on methods used in the design of reinforced concrete structures, checks for both out-of-plane shear and flexure failure modes, and has results that compare reasonably well with the NRC analysis [15], it is proposed that the model has sufficient robustness to be used in the generic evaluation of SFP designs with rectangular-shaped concrete walls and floor panels.

Table C-16
Comparison of model results with NRC Spent Fuel Pool Scoping Study

Parameter	SFP Seismic Model	NRC Scoping Study
Floor frequency	15.0	14
Dead load pressure	4,958 psf	4,900 psf
Vertical hydrodynamic impulsive pressure	5,375 psf	4,800 psf
Seismic vertical pressure	9,475 psf	7,990 psf
Peak displacement (Floor)	0.19 in	0.4 in
HCLPF	0.58g PGA	Not reported, but estimated to be approximately 0.7g PGA