# 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 [1] 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 [1] 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.

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 (10-20 Hz or more).
- 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, comprised 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 [7]. 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 capacity is derived using slab yield-line theory practiced in standard structural design [8]. The ultimate panel strength is based on the panel span aspect ratio (long side/short side), and nominal flexural section capacity accounting for both positive and negative steel reinforcement. Out-of-plane shear capacity, based on ACI-349 provisions, is also evaluated.

The NRC study [1] 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 [2] describes that the CDFM is developed assuming that (1) the seismic margin earthquake equipment response to the SME will be median centered, and (3) the assessment of component capacity will be conservative. The combination of a conservatively defined seismic demand, a median centered response to the demand, and a conservative strength prediction is considered to result in a HCLPF.

The method of seismic modeling is consistent with the approach in ASCE 4-98 [3]. A median-centered model is developed, consistent with the CDFM approach. Consistent with EPRI TR-103959 [4], the median damping for reinforced concrete is assumed to be 7% 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 [5].

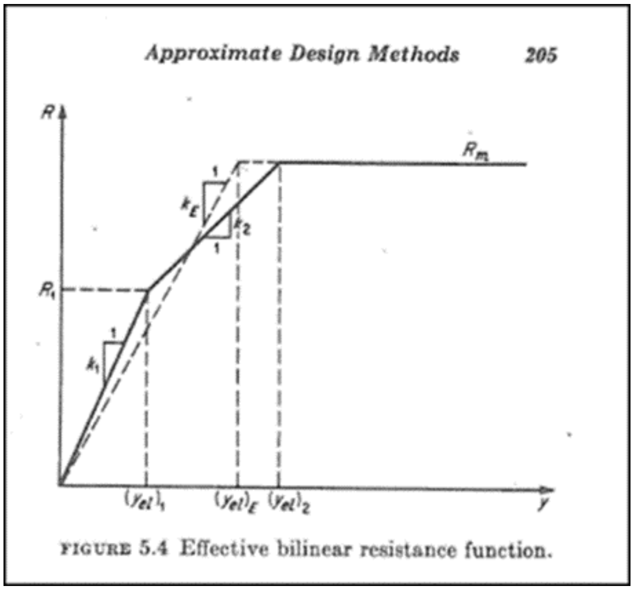



*Figure C-1*
*Example Elastic-Plastic Response Model*

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 [1] 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 [6]. Responses from the horizontal and vertical directions are combined using the 100-40-40 rule [3]. Displacements are checked to confirm they are small and shear demands are compared to ACI-349 code limits [11]. 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.

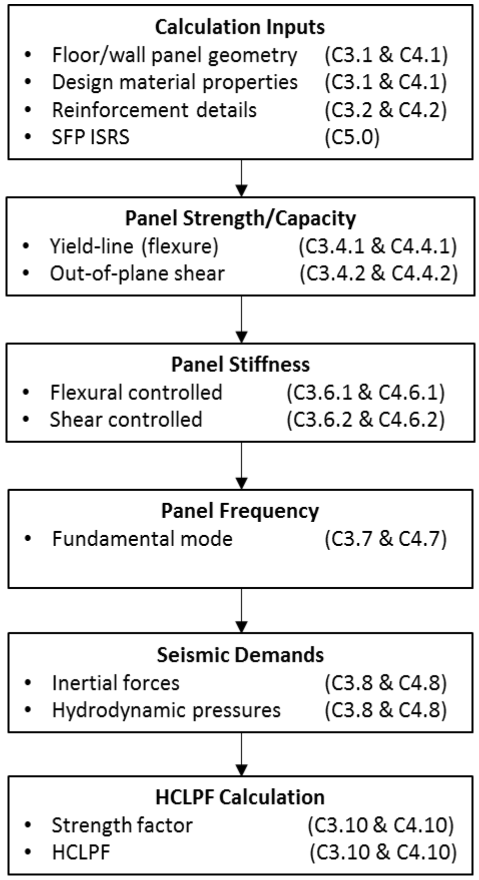



*Figure C-2*
*SFP HCLPF process and corresponding appendix sections*

## 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 capacity.

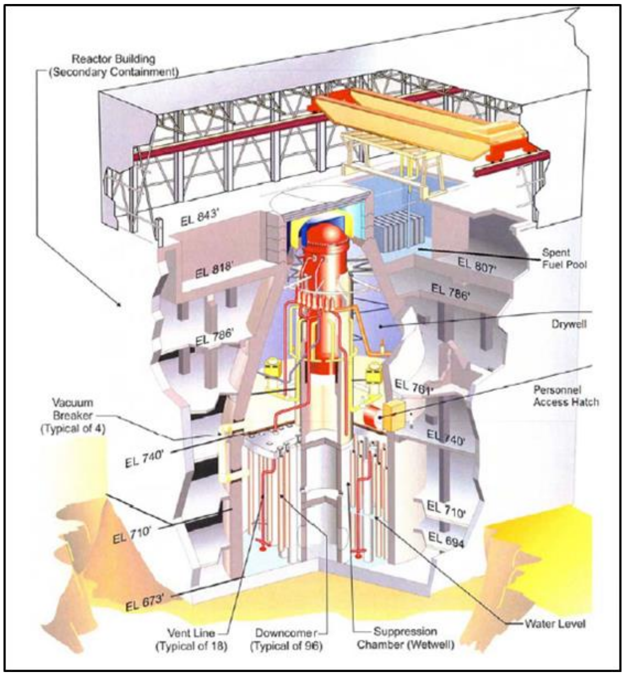



*Figure C-3*
*Schematic for Typical BWR Configuration with Elevated Spent Fuel Storage Pool*

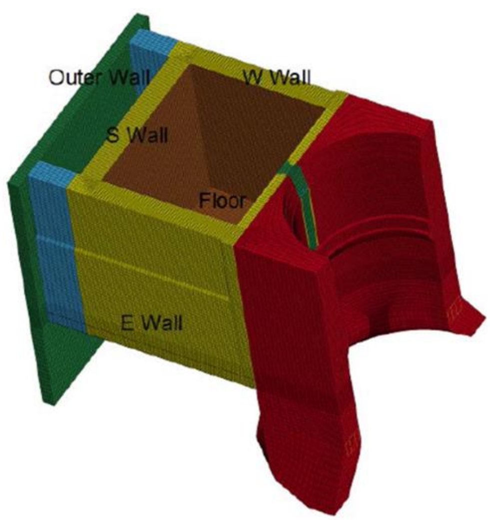



*Figure C-4*
*Peach Bottom Spent Fuel Pool (Source: NRC Spent Fuel Pool Scoping Study)*

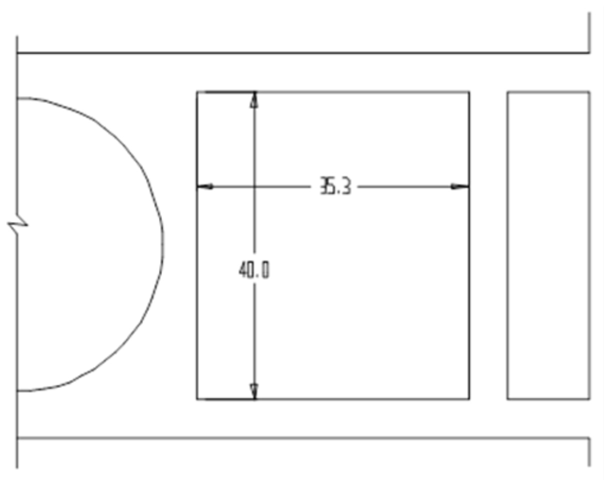



*Figure C-5*
*Plan View of Spent Fuel Pool Indicating Approximate Dimensions*

## 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 ratio was less than 1/800; 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.

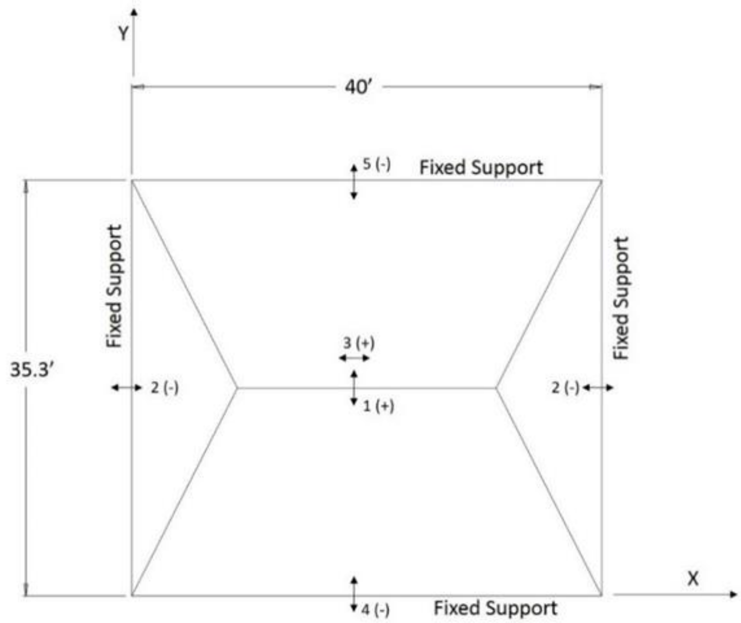



*Figure C-6*
*SFP floor slab schematic indicating example yield line pattern and directions of positive (+) and negative (-) reinforcement*

### C3.1 Material Properties

As the CDFM approach requires nominal design values to be used, 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 (direction // to Side Edge) | #11@9” | 0.0023 |
| M2; Total Negative Moment at Center of Side Edge | (2) #11@9” | 0.0045 |
| M3; Total Positive Moment at Center (direction // to long edge) | Bundle (3) #11@12”; two layers | 0.010 |
| M4; Total Negative Moment at Center of Long Edge 4 | #11@12” | 0.0017 |
| M5; Total Negative Moment at Center of Long Edge 5 | #11@12” | 0.0017 |

### C3.3 Floor Panel Section Strength

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

#### C3.3.1 Section Flexural Strength

The nominal panel flexural strength can be calculated as:

$$M_n = \rho f_y b d^2 \left(1 - 0.59 \frac{\rho f_y}{f'_c}\right) \text{ [8]}$$

Factored strength, $M_u = \Phi M_n$ ; $\Phi$=0.9 [11]

**Table C-3 Panel Ultimate Flexural Capacities (Factored)**

| Panel Location | Total Factored Strength ($M_u$); $\Phi$=0.9 | Factored Strength (Mu) per unit length |
|---|---|---|
| M1; Total Positive Moment at Center (direction // to Side Edge) | 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 (direction // to long edge) | 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 |
| M5; Total Negative Moment at Center of Top Edge | 2.30E8 lb-in | 4.79E5 lb-ft/ft |

### C3.3.2 Section Shear Capacity

The nominal shear capacity of the panel can be calculated from ACI 349 [11]

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

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

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

$V_u$ = 94,185 lb/ft

### C3.4 Panel Capacity

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

### C3.4.1 Flexural Capacity

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 required by the panel structures is equivalent to the external work caused by the pressure loading. The approach described by Nilson and Winter was utilized [8]. 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 capacity 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 highest internal moment was obtained. From this analysis, the angle of 41.4 degrees maximized the required internal moment, resulting in all diagonal yield lines meeting at the center of the panel. Based on this, the total internal work provided by the panel supports is estimated to be 1.95E7 lb-ft. The limiting panel edge contributes 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/ft$^2$ is: 470.7 lb-ft

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

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

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

$R_{UF}$ = (1.95E7 lb-ft) / (470.7 lb-ft) x 1 lb/ft$^2$ = 41,427lb/ft$^2$

$F_{UF}$ = 41,427 lb/ft$^2$ x (35.3 ft) x (40.0 ft) = 58.49E6 lb

<u>Calculate Cracking Resistance, Re</u>

First cracking resistance corresponds to pressure at which external work is equivalent internal work of limiting side support:

$R_e$= ($W_{side}$/ $W_i$) x $R_u$

$R_e$= (2.17E6 lb-ft / 1.95E7 lb-ft) x 41,427 lb/ft$^2$ = 4,610 lb/ft$^2$

$F_e$= (4,610 lb/ft$^2$) x (35.3 ft) x (40 ft) = 6.51E6 lb

### C3.4.2 Panel Shear Capacity

The floor panel out-of-plane shear capacity 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 of equal to the slab depth. The uniform pressure shear capacity, $R_{USL}$, in the longitudinal direction is defined as:

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

Where $V_u$ = ultimate shear capacity (Section C3.3.2)
$L_x$ = panel longitudinal direction
$A_{TL}$ = tributary pressure area on longitudinal edge (Figure C-7)

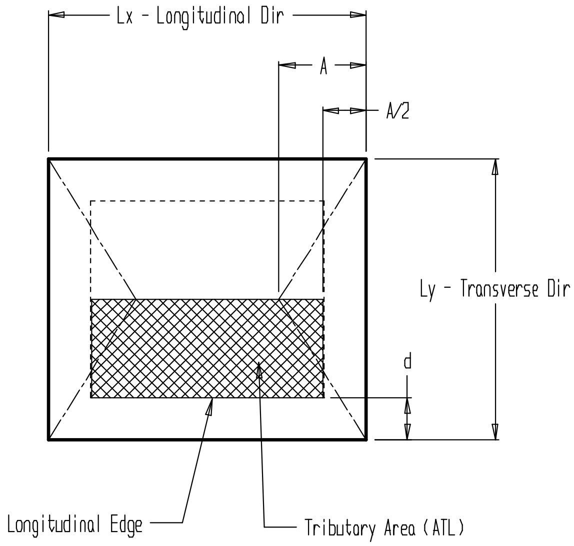
$$A_{TL} = \frac{\left(L_y - 2d\right)}{2}\ x\ \left(L_x - \frac{L_y}{2}\right)$$

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

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

Where Vu = ultimate shear capacity (Section C3.3.2)
$L_x$ = panel longitudinal direction
$A_{TT}$ = tributary pressure area on transverse edge (Figure C-7)

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



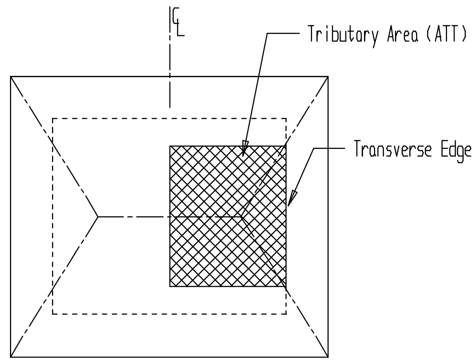




*Figure C-7*
*Schematic Indicating Approximate Tributary Areas*

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

$$A_{TL} = \frac{(35.3\,ft - 12.0\,ft)}{2}\ x\ \left(40.0\,ft\ - \frac{35.3\,ft}{2}\right)$$

$A_{TL}$ = (11.7 ft) x (22.4 ft) = 262.1 ft$^2$

$$A_{TT} = \frac{35.3\,ft\,(40.0\,ft\ - 12.0\,ft)}{4}$$

$A_{TT}$ = 247.1 ft$^2$

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

$R_{USL}$ = (94,185 lb/ft) (40.0 ft) / (262.1 ft$^2$)

$R_{USL}$ = 14,374 lb/ft$^2$

$R_{UST}$ = (94,185 lb/ft) (35.3 ft) / (247.1ft$^2$)

$R_{UST}$ = 13,455 lb/ft$^2$

Limiting shear pressure capacity, $R_{US}$, is therefore:

$R_{US}$ = 13,455 lb/ft$^2$

$F_{US}$ = 13,455 lb/ft$^2$ x (39.0 ft) x (40.0 ft) = 19.00E6 lb

### C3.5 Controlling Behavior (Flexure or Shear)

$F_{UF}$ = 58.49E6 lb (Flexure)

$F_{US}$ = 19.00E6 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 capacity. However, if shear pressure capacity ($R_{US}$) is less than the flexural pressure capacity ($R_{UF}$), 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 [10].

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

Where D is the flexural rigidity of the panel:

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

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

E = 3.60E6 psi (518.40E6 lb/ft$^2$) ; h = 73 in (6.08 ft) ; $\nu$ = 0.15

D = (518.40E6 lb/ft$^2$) x (6.08 ft)$^3$ / (12(1-0.15$^2$) = 9.93E9 lb-ft

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

D = 4.97E9 lb-ft

$L_y$ = 35.3 ft ; $L_x$ = 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. For this example, Army TM-855 [10] was utilized. These tables, 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:

Calculate Elastic Stiffness, $K_e$

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

Where $\alpha$= $L_y/L_x$ = 0.88 and $L_y$ = 35.3 ft

$K_e$ = 885.8 x (4.97E9 lb-ft) / (35.3 ft)$^4$ = 2.84E6 lb/ft$^2$/ft (236,700 lb/ft$^2$/in)

$K_e$ = (236,700 lb/ft$^2$/in) x (35.3 ft) x (40 ft) = 334.22E6 lb/in

Calculate Elastic displacement, $Y_e$

$Y_e = R_e/K_e$

$Y_e$ = (6.51E6 lb) / (334.22E6 lb/in) = 0.02 in

For the elastic-plastic stiffness, it is assumed that all supports have yielded and therefore have pinned boundary conditions. For this case, the elastic-plastic stiffness is approximated as:

Calculate Elastic-Plastic Stiffness, $K_{ep}$

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

Where $\alpha$= $L_y/L_x$ = 0.88 and $L_y$= 35.3 ft

$K_{ep}$ = 215 x (4.97E9 lb-ft) / (35.3 ft)$^4$ = 688,170 lb/ft$^2$/ft (57,348 lb/ft$^2$/in)

$K_{ep}$ = (57,348 lb/ft$^2$/in) x (35.3 ft) x (40 ft) = 80.97E6 lb/in

Calculate Elastic-Plastic Displacement, $Y_{ep}$

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

$Y_{ep}$ = 0.02 in + (58.49E6 lb – 6.51E6 lb) / (80.97E6 lb/in) = 0.66 in

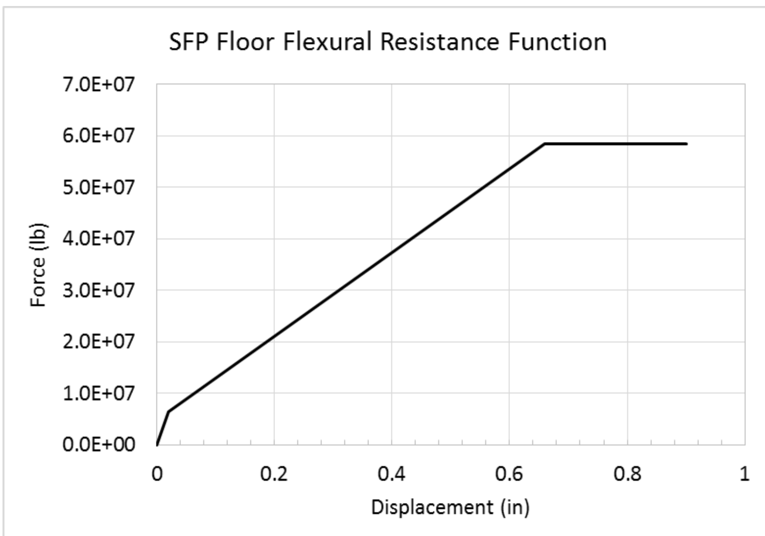



*Figure C-8*
*SFP Floor Flexural Resistance Function*

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 [7], 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.

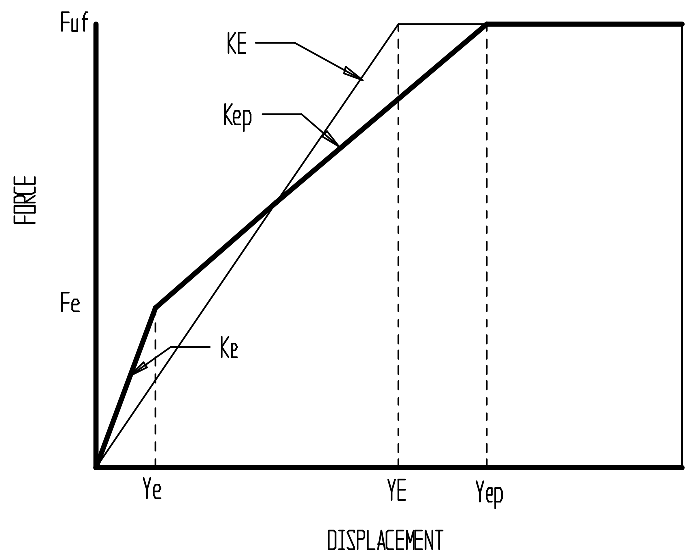



*Figure C-9*
*Comparison of equivalent linear resistance function (KE) to bi-linear resistance (Ke and Kep)*

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

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

$A_{BL}$= (0.5)(6.51E6 lb)(0.02 in)+(6.51E6 lb)(0.66 in – 0.02 in)+(0.5) x (58.49E6 lb -6.51E6 lb)(0.66 in – 0.02 in)

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

Solve for an equivalent system displacement, $Y_E$:

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

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

$Y_E$= 2(58.49E6 lb x 0.66 in – 2.1E7 lb)/(58.49E6 lb) = 0.30 in

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

$K_E = F_{UF}/Y_E$

$K_E$ = (58.49E6 lb)/(0.30 in) = 19.50E7 lb/in

### C3.6.2 Equivalent Stiffness of a Shear Controlled Panel

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

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

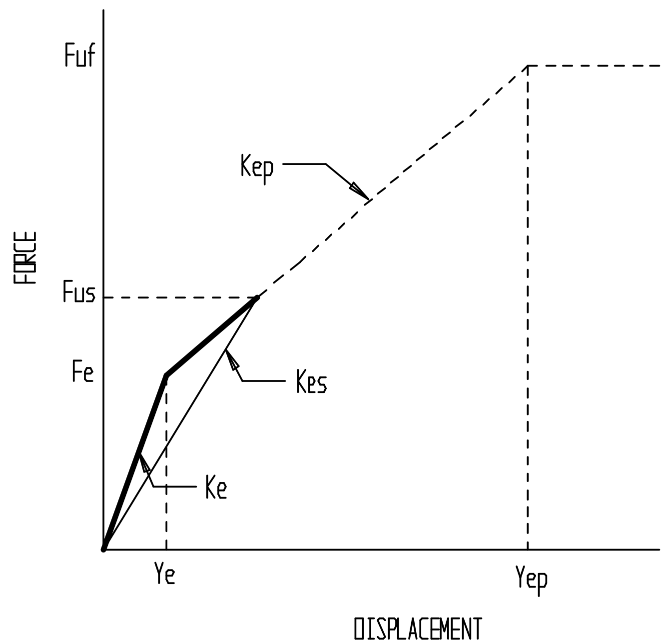



*Figure C-10*
*Equivalent linear resistance function ($K_{ES}$) for a shear controlled panel*

$$K_{ES} = \frac{19.00E6\ lb}{0.02\ in + \dfrac{19.00E6\ lb - 6.51E6\ lb}{80.97E6\ lb/in}}$$

$K_{ES}$ = 1.09E8 lb/in

### C3.7 Calculation of Panel Frequency

Calculate Wall Panel Fundamental Frequency

**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 lb-sec$^2$/in)

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

Load-mass factor , $K_{LM}$ = 0.7 (Biggs, Table 5.5)

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

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

$$T = 2\pi\sqrt{\frac{0.7x(18{,}135\ \text{lb}-\frac{\text{sec2}}{\text{in}})}{1.09\text{E}8\ \text{lb/in}}}$$

T = 0.068 sec

Panel frequency = 1/T = 14.7 Hz

For selecting seismic spectral accelerations, consider peak broadening ± 15%. Chose three frequencies, 12.9 Hz, 14.7 Hz, and 16.9 Hz.

## C3.8 Pressure Demands

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

For the purpose of this calculation, the high seismic demands are assumed to result in significant cracking to the SFP structure. 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 (12.9 Hz and 16.9 Hz), the approximate peak vertical spectral acceleration is 2.3 g. Due to the cracking in the SFP structure, it is assumed that damping will increase to 7%. The maximum spectral acceleration response corresponding to this damping level is approximately 1.94 g. A load-mass factor is utilized in accordance with Biggs [7].

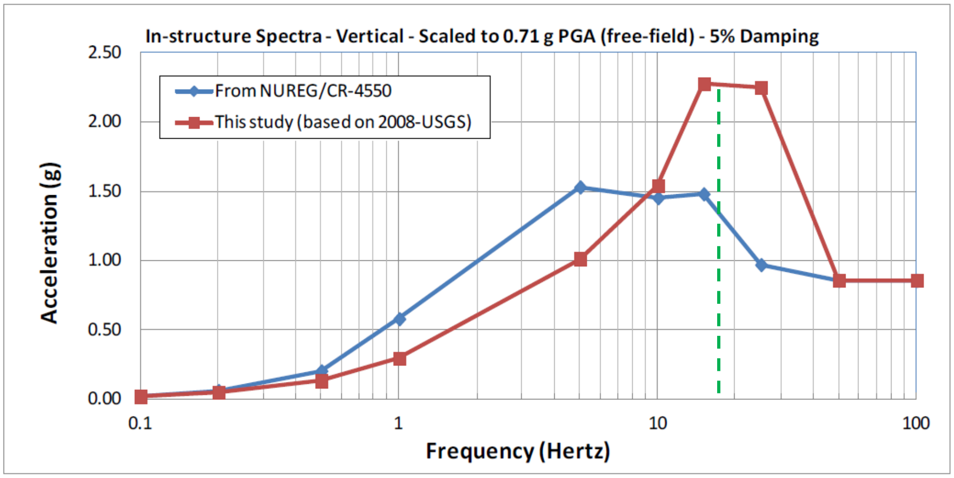



*Figure C-11*
*Maximum vertical spectral acceleration value, at 16.9 Hz, assumed in calculating floor inertial forces*

Floor slab and assembly weight: 3.7E6 lb

$F_{floor} = W_{floor}$ x 1.94 g x 0.7 ; where 0.7 is approximate mass participation factor

$F_{floor}$ = 3.7E6 lb x 1.94g x 0.7 = 5.02E6 lb

$F_{floor}$ = 5.03E6 lb / (35.3 ft x 40 ft) = 3,560 lb/ft$^2$

<u>Calculate Hydrodynamic Floor Pressures</u>

The vertical hydrodynamic floor pressure is assumed to equal the mass of the water accelerating confined within the SFP. The spectral acceleration assumed is equal to the worst-case acceleration values of 12.9 Hz, 14.7 Hz, and 16.9 Hz. (which represent ± 15% peak broadening. This corresponds to a 7% damped spectral acceleration of approximately 1.94g.

Vertical Hydrodynamic Force = 3.3E6 lb x 1.94g

Vertical Hydrodynamic Pressure = 6.4E6 lb / (35.3 ft x 40 ft) = 4,533 lb/ft$^2$

Combined Vertical Pressure = 3,560 lb/ft$^2$ + 4,533 lb/ft$^2$ = 8,093 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 [6]

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:

Calculation of Seismic Demands, E

| | |
|---|---|
| EW hydrodynamic pressure | 1,711 lb/ft$^2$ (bottom elevation) |
| NS hydrodynamic pressure | 1,940 lb/ft$^2$ (bottom elevation) |

The controlling hydrodynamic pressure, NS direction , is assumed to vary as a sine wave with maximum amplitude at the wall face and zero amplitude at the mid-span of the pool (x=20 ft). At a distance of 6.0 ft from the face of the E and W supports, the horizontal pressure is approximated as:

$$P_{NSH} = \ 1{,}940\ psf\ \left[\sin\left(\frac{\pi}{2}\ \frac{20\ ft - 6\ ft}{20\ ft}\right)\right]$$

$P_{NSH}$ = 1,940 psf (0.891) = 1,729 psf

Vertical inertia and hydrodynamic pressure 8,092 lb/ft$^2$

E = 8,093 lb/ft$^2$ + 0.4(1,729 lb/ft$^2$)

E = 8,785 lb/ft$^2$ (assumed uniform pressure distribution)

Calculate Dead Load, D

D = 7.0E6 lb / (35.3 ft x 40 ft) = 4,957 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:

$\Delta_v = U/K_{ES}$

$\Delta_v$ = (19.00E6 lb) / (1.09E8 lb/in) = 0.17 inches

Under the assumed pressure demand of 13,742 lb/ft$^2$, the panel displacement is 0.17 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 0.33, 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 capacity could be realized.

## C3.10 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

$$F_s = \frac{13{,}455\ psf - 4{,}957\ psf}{8{,}785\ psf}$$

Fs = 0.97

**Table C-5 Margin factors assumed to calculate floor HCLPF**

| Margin Factors | |
|---|---|
| Fm; material margin factor | 1 |
| Fam; analysis method | 1 |
| Fca; code allowable | 1 |
| Fma; margin to code allowable | 1 |
| Fd; damping | 1 |
| Fiea; inelastic energy absorption | 1 |
| Fs; strength factor | 0.97 |
| Fsm; margin increase factor | 0.97 |

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 0.68g PGA and 1.55g $SA_{10-20}$ (0.97 x 1.6 g).

The HCLPF capacity of the SFP is defined at approximately 95% confidence of less than about 5% probability of failure. As long as $B_r/B_u$ 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 is similar to the HCLPF PGA capacity of 0.68g 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.

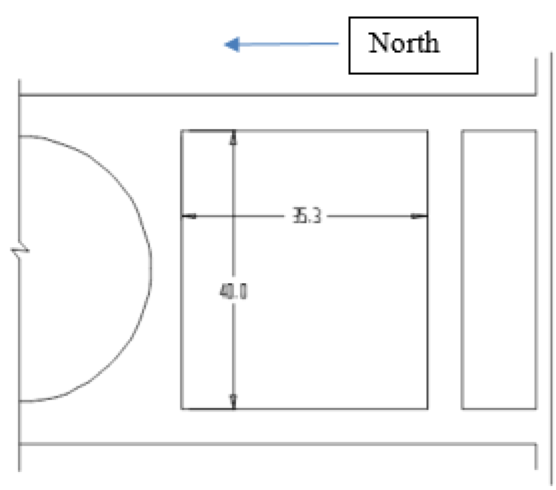



*Figure C-12*
*Plan view of SPF showing wall spans*

### 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:

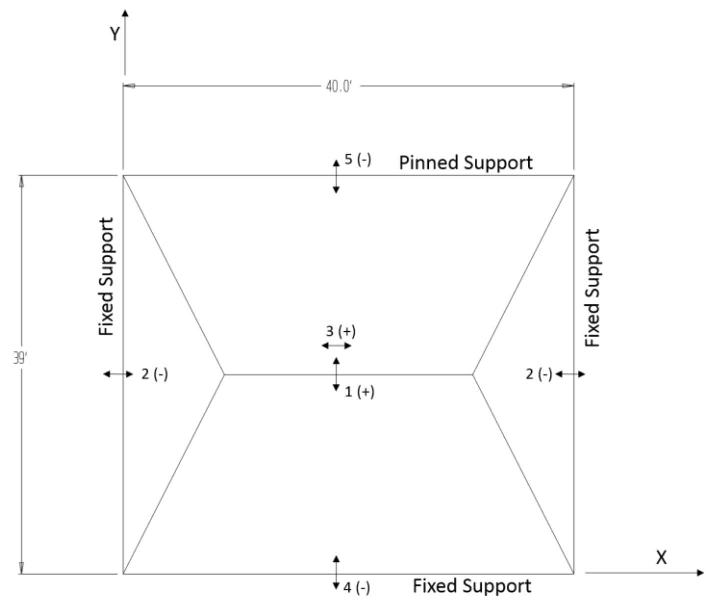



*Figure C-13*
*Example yield line pattern indicating assumed wall boundary conditions and locations of (+) and (-) reinforcement*

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

| Panel Location | Reinforcement | Reinforcement Ratio |
|---|---|---|
| M1; Total Positive Moment at Center (direction // to Side Edge) | #11@9” | 0.0025 |
| M2; Total Negative Moment at Ctr of Side Edge | #11@12” | 0.0019 |
| M3; Total Positive Moment at Center (direction // to long edge) | #11@9” | 0.0025 |
| M4; Total Negative Moment at Ctr of Bottom Edge | (3) #11@9” bundled | 0.010 |
| M5; Total Negative Moment at Ctr of Top Edge | Assume None | NA |

## C4.3 Wall Panel Section Strength

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

### C4.3.1 Section Flexural Strength

The nominal panel flexural strength can be calculated as:

$$M_n = \rho f_y b d^2 \left(1 - 0.59 \frac{\rho f_y}{f'_c}\right) \text{ (Nilson, Winter) [8]}$$

Factored strength, $M_u = \Phi M_n$ ; $\Phi$=0.9 [11]

**Table C-8 Panel Ultimate Flexural Capacities**

| Panel Location | Total Factored Strength ($M_u$); $\Phi$=0.9 | Factored Strength (Mu) per unit length |
|---|---|---|
| M1; Total Positive Moment at Center (direction // to Side Edge) | 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 (direction // to long edge) | 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 |
| M5; Total Negative Moment at Ctr of Top Edge | NA | NA |

### C4.3.2 Section Shear Capacity

The nominal shear capacity of the panel can be calculated from ACI 349 [11]

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

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

$V_u$ = 82,573 lb/ft

## C4.4 Panel Capacity

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

### C4.4.1 Flexural Capacity

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 [8]. 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 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

<u>Calculate External Work</u>

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

<u>Calculate Maximum Resistance, Ru</u>

The maximum pressure resistance, $R_u$, is:

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

$R_{UF}$ = (9.88E6 lb-ft) / (624.2 lb-ft x 1 lb/ft$^2$) = 15,833 lb/ft$^2$

$F_{UF}$ = 15,833 lb/ft$^2$ x (39 ft) x (40 ft) = 24.70E6 lb

<u>Calculate Cracking Resistance, Re</u>

First cracking resistance corresponds to pressure at which external work is equivalent internal work of side supports:

$R_e$= ($W_{sides}$/ $W_i$) x $R_u$

$R_e$= (2.15E6 lb-ft / 9.88E6 lb-ft) x 15,833 lb/ft$^2$ = 3,445 lb/ft$^2$

$F_e$= (3,445 lb/ft$^2$) x (39 ft) x (40 ft) = 5.37E6 lb

### C4.4.2 Shear Capacity

The wall panel out-of-plane shear capacity 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 capacity, $R_{USL}$, in the longitudinal direction is defined as:

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

Where $V_u$ = ultimate shear capacity (Section C4.3.2)
$L_x$ = panel longitudinal direction
$A_{TL}$ = tributary pressure area on longitudinal edge (Figure C-7)

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

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

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

Where Vu = ultimate shear capacity (Section C4.3.2)
$L_x$ = panel longitudinal direction
$A_{TT}$ = tributary pressure area on transverse edge (Figure C-7)

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

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

For the wall panel;
Lx = 40.0 ft
Ly = 39.0 ft
d = 5.3 ft

$$A_{TL} = \frac{(39.0\,ft - 10.6\,ft)}{2}\ x\ \left(40.ft\ - \frac{39.0\,ft}{2}\right)$$

$A_{TL}$ = (14.2 ft) x (20.5 ft) = 291.1 ft$^2$

$$A_{TT} = \frac{39.0\,ft\,(40.0\,ft\ - 10.6\,ft)}{4}$$

$A_{TT}$ = 286.7 ft$^2$

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

$R_{USL}$ = (82,573 lb/ft) (40.0 ft) / (291.1 ft$^2$)

$R_{USL}$ = 11,346 lb/ft$^2$

$R_{UST}$ = (82,573 lb/ft) (39.0 ft) / (286.7 ft$^2$)

$R_{UST}$ = 11,232 lb/ft$^2$

Limiting shear pressure capacity, $R_{US}$, is therefore:

$R_{US}$ = 11,232 lb/ft$^2$

$F_{US}$ = 11,232 lb/ft$^2$ x (39.0 ft) x (40.0 ft) = 17.52E6 lb

## C4.5 Controlling Behavior (Flexure or Shear)

$F_{UF}$ = 24.70E6 lb (Flexure)

$F_{US}$ = 17.52E6 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 capacity. However, if shear capacity is less than the flexural capacity, 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

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

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

E = 3.60E6 psi (518.4E6 lb/ft$^2$) ; h = 64 in (5.3 ft) ; ν = 0.15

D = (518.4E6 lb/ft$^2$) x (5.3 ft)$^3$ / (12(1-0.15$^2$) = 6.58E9 lb-ft

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

D= 3.29E9 lb-ft

$L_y$ = 39 ft ; $L_x$ = 40 ft

$L_y/L_x$ = 39 ft / 40 ft = 0.98

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

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 α= $L_y/L_x$ = 0.98

$K_e$ = 740 x (3.29E9 lb-ft) / (39 ft)$^4$ = 1.05E6 lb/ft$^2$/ft (87,698 lb/ft$^2$/in)

$K_e$ = (87,698 lb/ft$^2$/in) x (39 ft) x (40 ft) = 136.81E6 lb/in

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

$Y_e = F_e/K_e$

$Y_e$ = (5.37E6 lb) / (136.81E6 lb/in) = 0.04 in

Calculate Elastic-Plastic Stiffness, $K_{ep}$

For the elasto-plastic stiffness, it is assumed that all supports are pinned. In this case, the panel stiffness can be approximated as:

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

Where $\alpha = L_y/L_x$

$K_{ep}$ = 258 x (3.29E9 lb-ft) / (39 ft)$^4$ = 3.77E5 lb/ft$^2$/ft (31,417 lb/ft$^2$/in)

$K_{ep}$ = (31,417 lb/ft$^2$/in) x (39 ft) x (40 ft) = 49.01E6 lb/in

Calculate Elastic-Plastic Displacement, $Y_{ep}$

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

$Y_{ep}$ = 0.04 in + (24.70E6 lb – 5.37E6 lb) / (49.01E6 lb/in) = 0.43 in

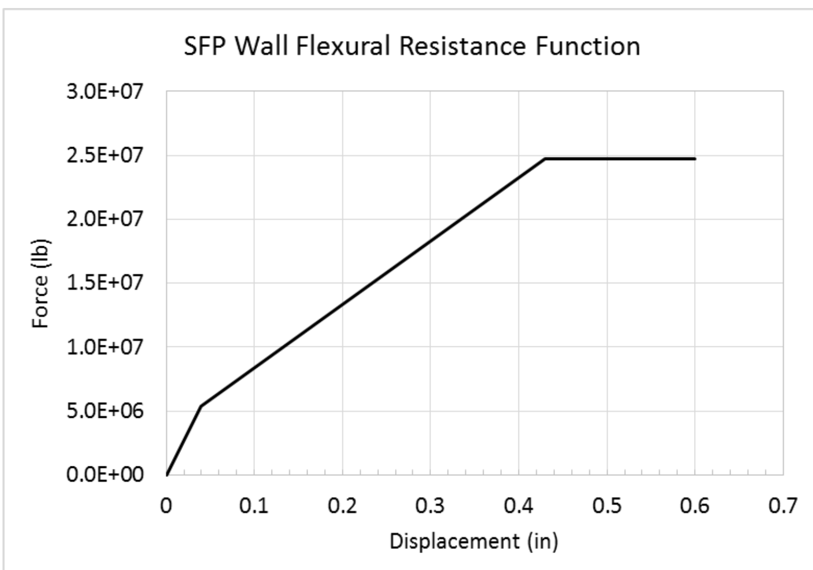



*Figure C-14*
*SFP Wall Resistance Function*

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:

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

$A_{BL}$= (0.5)(5.37E6 lb)(0.04 in)+(5.37E6 lb)(0.43 in – 0.04 in)+(0.5) x (24.70E6 lb -5.37E6 lb)(0.43 in – 0.04 in)

$A_{BL}$ = 5.97E6 lb-in (Area under bilinear resistance curve)

Solve for an equivalent system displacement, $Y_E$:

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

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

$Y_E$= 2(24.70E6 lb x 0.43 in – 5.97E6 lb-in)/(24.7E6 lb) = 0.38 in

Calculate Equivalent Stiffness, $K_E$

$K_E = F_{UF}/Y_E$

$K_E$ = (24.70E6 lb)/(0.38 in) = 65.00E6 lb/in

### C4.6.2 Equivalent Stiffness of a Shear Controlled Panel

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

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

$$K_{ES} = \frac{17.52E6\ lb}{0.04\ in + \dfrac{17.52E6\ lb - 5.36E6\ lb}{49.01E6\ lb/in}}$$

$K_{ES}$ = 60.85E6 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 | 3.3E6 | Assumes 1.5 ft freeboard and 50% water participating on each wall |
| Total | 4.6E6 | |

Total mass = $M_t$ = (4.6E6 lb) / (386 lb-sec$^2$/in)

$M_t$ = 11,917 lb-sec$^2$/in

Load-mass factor , $K_{LM}$ = 0.7 (Biggs, Table 5.5)

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

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

$$T = 2\pi \sqrt{\frac{0.7x(11{,}917\ \text{lb}-\frac{\text{sec2}}{\text{in}})}{60.85\text{E}6\ \text{lb/in}}}$$

T = 0.074 sec

Panel frequency = 1/T = 13.6 Hz

For selecting seismic spectral accelerations, consider peak broadening +/- 15%. Chose three frequencies, 11.8 Hz, 13.6 Hz, and 15.6 Hz.

## C4.8 Pressure Demands

Calculate Wall Inertia Force; $F_{wi}$

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 1.25 g was assumed and a load-mass factor (0.7) is utilized in accordance with Biggs [7].

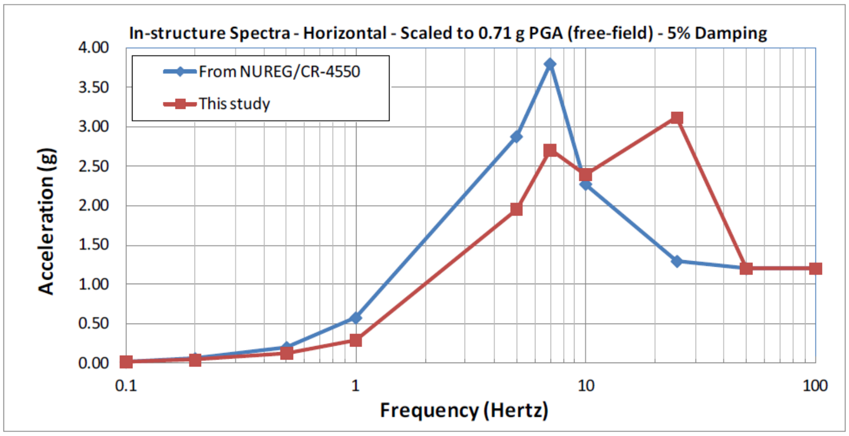



*Figure C-15*
*Horizontal spectral acceleration at mid-height of SFP*

Wall weight: 1.30E6 lb

$F_{wi} = W_{wall}$ x 1.25 g x 0.7

$F_{wi}$ = 1.140E6 lb

Calculation of Hydrodynamic Wall Pressures

The hydrodynamic wall pressures are calculated using ACI 350.3/350.3R-15 [6]. 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.

Inertial and Convective Masses

The computation of equivalent inertial 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 inertial 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:

<u>Inertial mass center-of-gravity height</u>

For tanks with $L/H_l < 1.33$:

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

For tanks with $L/H_l > 1.33$;

$H_i/H_l = 0.375$

<u>Convective mass center-of-gravity height</u>

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 inertial 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 |

<u>Sloshing Frequencies</u>

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 |

<u>Calculation of Resultant Inertial Force (Horizontal)</u>

$P_i = W_{tot} \times W_i/W_l \times SA_{h/2}$

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 1.25 g (ref Figure C-15). This is the assumed inertial acceleration of the SFP water in the horizontal direction.  The assumed pressure distribution for impulsive loading is derived from ACI-350, Figure R5.3:

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

$P_i$ = inertial force
$H_l$ = pool depth
$h_i$ = center of gravity height of inertial mass
y = height from bottom of SFP

<u>Calculation of Resultant Convective Force (Horizontal)</u>

$P_c = W_{tot}$ 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 0.32 g (scaled from Figure C-18).

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 |

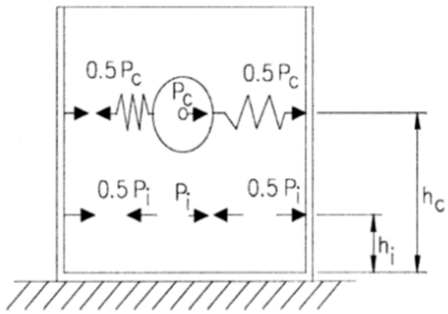



*Figure C-16*
*SFP hydrodynamic model*

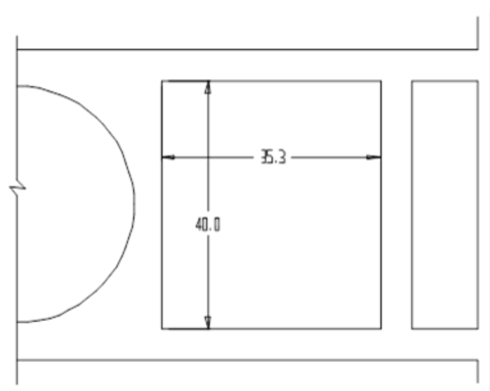



*Figure C-17*
*Plan view of SFP showing sloshing directions*

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

Calculation of Seismic Demands, E

EW hydrodynamic pressure 1,058 lb/ft2

NS hydrodynamic pressure 1,256 lb/ft2

Wall inertia 1.14E6 lb / (39 ft x 40 ft) = 731 lb/ft$^2$

Vertical hydrodynamic pressure 4,533 lb/ft$^2$ / 2.0 = 2,267 lb/ft$^2$; Reduced by 2.0 factor to account for hydrostatic variation.

E = 731 lb/ft$^2$ + 1,256 lb/ft$^2$ + 0.4(2,267 lb/ft$^2$)

E = 2,893 lb/ft$^2$ (assumed uniform pressure distribution)

Calculate Dead Load (Hydrostatic), D

D = $\rho_{water}$ x H ; $\rho_{water}$ = 62.4 lb/ft$^3$; H = 39 ft

D = 62.4 lb/ft$^3$ x 39 ft = 2,434 lb/ft$^2$ ;
(Conservative in that maximum hydrostatic pressure is assumed to act uniformly on entire wall panel)

Calculate Combined Load, U

U= E + D

U = 2,893 lb/ft$^2$ + 2,434 lb/ft$^2$

U = 5,327 lb/ft$^2$

U = 5,327 lb/ft$^2$ x (39.0 ft) x (40.0 ft) = 8.31E6 lb

### C4.9 Displacement and Flexural Margin

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

$\Delta v = U/K_{ES}$

$\Delta v$ = (8.31E6 lb) / (60.85E6 lb/in) = 0.14 inches

Under the assumed pressure demand of 5,327 lb/ft$^2$, the panel displacement is 0.14 inches, which exceeds the estimated elastic displacement of 0.05 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 0.4, 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 capacity 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

$$F_s = \frac{11{,}232\,psf - 2{,}434\,psf}{2{,}893\,psf}$$

$F_s$ = 3.04

**Table C-15 Margin factors assumed to calculate wall HCLPF**

| Margin Factors | |
|---|---|
| Fm; material margin factor | 1 |
| Fam; analysis method | 1 |
| Fca; code allowable | 1 |
| Fma; margin to code allowable | 1 |
| Fd; damping | 1 |
| Fiea; inelastic energy absorption | 1 |
| Fs; strength factor | 3.04 |
| Fsm; margin increase factor | 3.04 |

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 2.13g PGA and 4.9g $SA_{10\text{-}20}$ (3.04 x 1.6 g).

### C5.0 Provided Response Spectra

The below response spectra are provided in the NRC SFP Scoping Study [1], and are assumed as input for the floor and wall panel analyses described in this report.

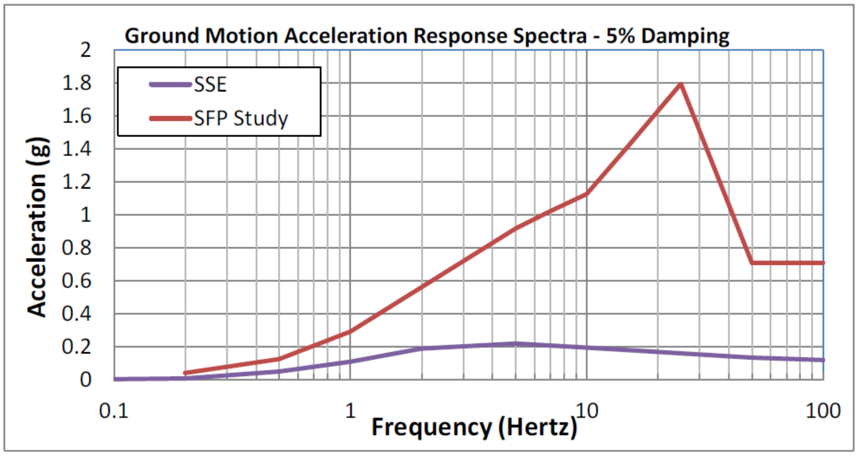



*Figure C-18*
*NRC SFP Scoping Study foundation input response spectrum (Horizontal)*

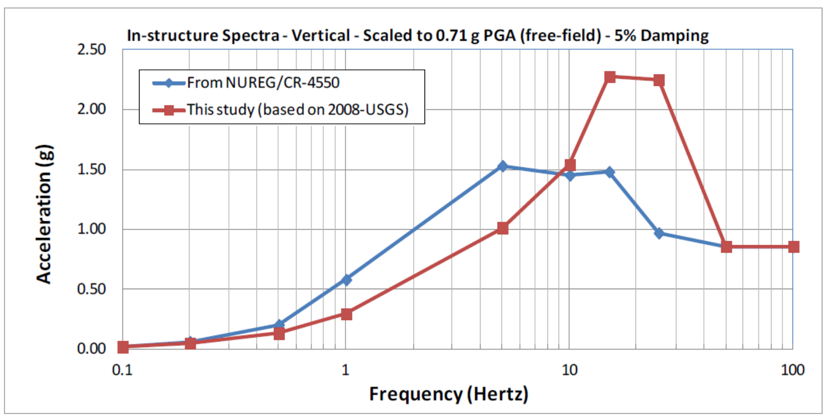
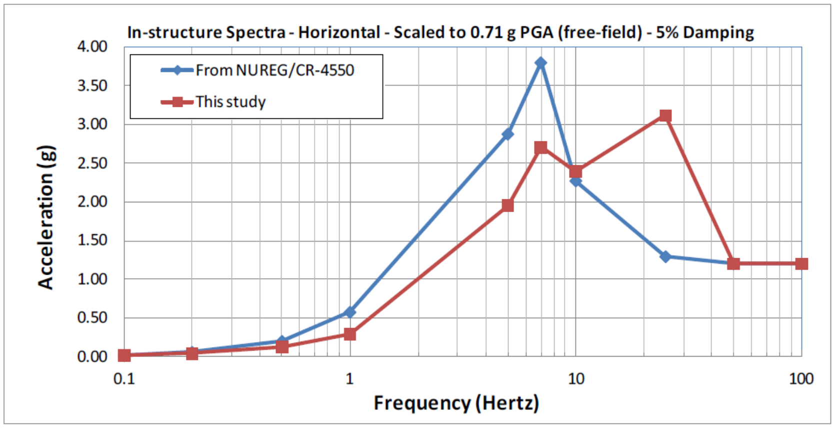



*Figure C-19*
*Vertical ISRS at bottom elevation of SFP (NRC SFP Scoping Study)*




*Fgure 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 0.68g PGA and 1.55g $SA_{10-20}$. The assumed Seismic Margin Earthquake corresponds to the seismic ground motion described in the NRC Spent Fuel Pool Scoping Study [1]. 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 14.7 Hz and 0.2 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 capacity. The wall is estimated to have a fundamental frequency of 13.6 Hz, a peak displacement of 0.14 in, and a HCLPF 2.13g PGA and 4.9g $SA_{10-20}$.

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 well with the NRC analysis [1], 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 | 14.7 | 14 |
| Dead load pressure | 4,960 psf | 4,900 psf |
| Vertical hydrodynamic impulsive pressure | 4,533 psf | 4,800 psf |
| Seismic vertical pressure | 8,093 psf | 7,990 psf |
| Peak displacement (Floor) | 0.2 in | 0.4 in |
| HCLPF | 0.68g PGA | Not reported, but estimated to be approximately 0.7g PGA |

### C7.0 References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