**FENOC**

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August 9, 2010
L-10-235

10 CFR 50.90

ATTN: Document Control Desk
U. S. Nuclear Regulatory Commission
Washington, DC 20555-0001

SUBJECT:
Beaver Valley Power Station, Unit No. 2
Docket No. 50-412, License No. NPF-73
Response to Request for Additional Information for License Amendment Request No. 08-027, Unit 2 Spent Fuel Pool Rerack (TAC No. ME1079)

By letter dated April 9, 2009 (Reference 1), as supplemented by letters dated June 15, 2009 (Reference 2), January 18, 2010 (Reference 3), March 18, 2010 (Reference 4), May 3, 2010 (Reference 5), May 21, 2010 (Reference 6), and June 1, 2010 (Reference 7). FirstEnergy Nuclear Operating Company (FENOC) requested an amendment to the operating license for Beaver Valley Power Station (BVPS) Unit No. 2. The proposed amendment would revise the Technical Specifications to support the installation of high density fuel storage racks in the BVPS Unit No. 2 spent fuel pool (SFP).

During a May 27, 2010 teleconference, the Nuclear Regulatory Commission staff (NRC) requested FENOC docket supplemental responses to Request for Additional Information (RAI) numbers 2, 5, 6, 14, 16, 17, and 19 (originally provided in Reference 5). Supplemental responses to RAI numbers 2, 5, 6, 14, 16, 17, and 19 are provided in Attachment 1. Attachment 2 provides FENOC's response to the NRC's June 11, 2010 RAI (Reference 8). During a July 7, 2010 teleconference, the NRC requested that FENOC docket a supplemental response to RAI number 8 (originally provided in Reference 6). The supplemental response to RAI number 8 is included in Attachment 3.

The information provided by this submittal does not invalidate the no significant hazard evaluation submitted by Reference 1. There are no regulatory commitments contained in this letter. If there are any questions or if additional information is required, please contact Mr. Thomas A. Lentz, Manager – FENOC Fleet Licensing, at 330-761-6071.

A001
NRR

I declare under penalty of perjury that the foregoing is true and correct. Executed on August 9, 2010.

Sincerely,

Paul A. Harden

Attachments:
1. Response to May 27, 2010 NRC Request for Supplemental Information
2. Response to June 11, 2010 NRC Request for Additional Information
3. Response to July 7, 2010 NRC Request for Supplemental Information

References:
1. FENOC Letter L-09-086, "License Amendment Request No. 08-027, Unit 2 Spent Fuel Pool Rerack," dated April 9, 2009 (Accession No. ML091210251).
2. FENOC Letter L-09-162, "Additional Technical Information Pertaining to License Amendment Request No. 08-027 (TAC No. ME1079)," dated June 15, 2009 (Accession No. ML091680614).
3. FENOC Letter L-10-001, "Response to Request for Additional Information for License Amendment Request No. 08-027, Unit 2 Spent Fuel Pool Rerack (TAC No. ME1079)" dated January 18, 2010 (Accession No. ML100191805).
4. FENOC Letter L-10-082, "Response to NRC Staff Request for Additional Information Regarding Criticality Analyses Supporting a Spent Fuel Pool Re-rack for Unit 2 (TAC No. ME1079)," dated March 18, 2010 (Accession No. ML100820165).
5. FENOC Letter L-10-121, "Response to Request for Additional Information for License Amendment Request No. 08-027 (TAC No. ME1079)," dated May 3, 2010 (Accession No. ML101260059).
6. FENOC Letter L-10-151, "Response to Request for Additional Information for License Amendment Request No. 08-027 (TAC No. ME1079)," dated May 21, 2010 (Accession No. ML101460057).
7. FENOC Letter L-10-130, "Remainder of Responses to NRC Staff Request for Additional Information Regarding Unit 2 Spent Fuel Pool Rerack Criticality Analyses (TAC No. ME1079)", dated June 1, 2010 (Accession No. ML101610118).
8. NRC Letter dated June 11, 2010, titled "BEAVER VALLEY POWER STATION, UNIT NO. 2 - REQUEST FOR ADDITIONAL INFORMATION RE: SPENT FUEL POOL RERACK LICENSE AMENDMENT (TAC NO. ME1079)" (Accession No. ML101380546).

cc: NRC Region I Administrator
NRC Senior Resident Inspector
NRR Project Manager
Director BRP/DEP
Site Representative (BRP/DEP)

On May 27, 2010, a Nuclear Regulatory Commission staff (NRC) and FirstEnergy Nuclear Operating Company (FENOC) telephone conference was held to discuss RAI responses provided in FENOC letter dated May 3, 2010 (Reference 3) related to the Beaver Valley Power Station (BVPS) Unit No. 2 spent fuel pool (SFP) rerack license amendment request No. 08-027. To complete its review, the NRC requested that FENOC docket supplemental information for the response to RAI numbers 2, 5, 6, 14, 16, 17, and 19. Supplemental information is provided for each RAI.

**Supplement to RAI 2:**

The structural design analysis of the spent fuel racks to be installed in the SFP at BVPS Unit No. 2 evaluates the potential for buckling failure under seismic loading at two critical locations:

i) a perimeter cell wall adjacent to the corner of the rack (just above the rack base plate), that experiences the maximum vertical compression due to the axial force and bending moments on the rack cell structure caused by the earthquake, and

ii) the cell walls at the top of the rack that are oriented perpendicular to the ¼ inch thick reinforcement bar, which absorb the lateral impact between adjacent racks or between a rack and the SFP wall during a seismic event.

The first location is the focus of RAI 2.c in Reference 3, and the second location is the focus of RAI 11 in Reference 2 and RAIs 2.a and 2.b in Reference 3. The buckling evaluations performed for these two locations are completely independent from each other, and they use two different methods of analysis. However, the acceptance criterion for both evaluations is provided in paragraph NF-3321.1(b) of the American Society of Mechanical Engineers Boiler and Pressure Vessel Code (ASME Code), Section III, Subsection NF (Reference 15). Specifically, "the allowable stress shall be limited to two-thirds of the critical buckling stress." The following paragraphs describe the method of analysis, the analytical model, and the loading for each critical location.

Perimeter Cell Wall Just Above Rack Base Plate (Location i)

For this location, the cell geometry and the boundary conditions allow the critical buckling stress for the cell wall to be calculated using the classical buckling solution for a rectangular plate under uni-axial compression given in Chapter 9 of *Theory of Elastic Stability* by Timoshenko and Gere (Reference 4) and Table 35 of *Roark's Formulas for Stress & Strain* (Reference 5). The width of the plate is 8.8 inches, which equals the inner dimension of the storage cell. At the base of the rack, the cell wall acts alone in compression for a vertical length of only 2-7/8 inches. Above this point the neutron absorber sheathing is welded to the cell wall and provides additional reinforcement (which is not considered) against buckling. For conservatism, the unbraced length of the cell wall is taken as 6 inches, and the length/width ratio for the cell wall plate is computed as 8.8 inches /6 inches equals 1.47 (as opposed to 6 inches/8.8 inches = 0.68) to reduce the critical buckling stress. Since the cell wall plate is continuous on three sides and the fourth side is welded to the rack base plate, the boundary conditions for the rectangular plate are considered clamped. Per Table 35 (Case 1b) of Reference 5, the critical buckling stress for this plate configuration is computed as:

$$\sigma_{cr} = K\frac{E}{1-\nu^2}\left(\frac{t}{b}\right)^2 = 15{,}928\,psi$$

where K = 7.23 for length/width ratio of 1.47

E = 27.6 $\times 10^6$ pounds per square inch (psi)

v = 0.3

t = 0.075 inches

b = 8.8 inches

The critical buckling stress is compared with the maximum compressive stress on the outermost cell wall under safe shutdown earthquake (SSE) conditions, which is determined using the Holtec proprietary code DYNARACK. The maximum compressive stress under SSE conditions is 9,380 psi (which is given in Subsection 5.6.10.1 of Enclosure C to Reference 1). Thus, the safety factor (SF) against cell wall buckling at the base of the cells is:

$$SF = \frac{15{,}928\,psi}{9{,}380\,psi} = 1.70$$

Since the safety factor is greater than 1.5, the compressive load on the outermost cell wall is less than two-thirds of the critical buckling stress as required by paragraph NF-3321.1(b) of the ASME Code.

<u>Cell Walls at Top of Rack Perpendicular to Reinforcement Bar (Location ii)</u>

The buckling potential of the cell walls at the top of the rack, which are subject to lateral impact loads, are investigated using the computer code LS-DYNA as described in the response to RAI 11.c in Reference 2. The response to RAI 2.a in Reference 3 incorrectly identified the computer code as ANSYS. This is incorrect since LS-DYNA was the computer code used. LS-DYNA is used, as opposed to a classical plate buckling solution, due to the complex nature of the loading and the boundary conditions. Figure 1 below shows the LS-DYNA model used for this buckling evaluation.

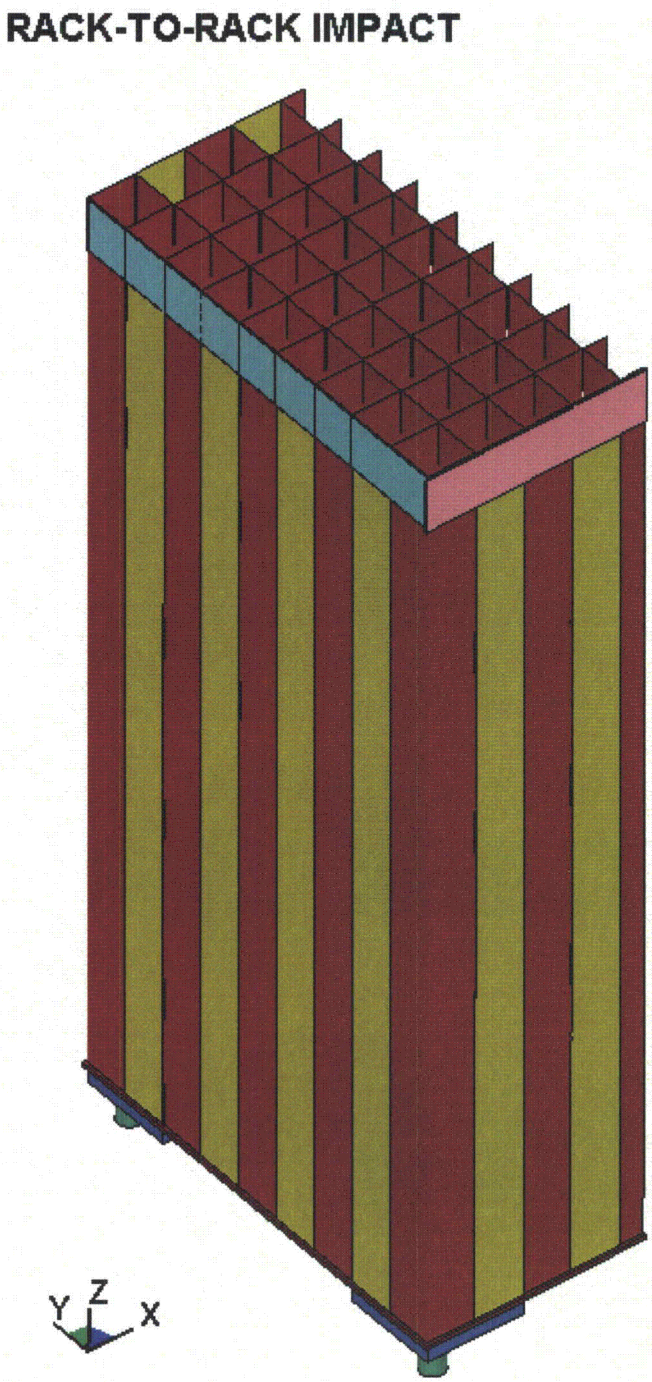



Figure 1 – LS-DYNA Model for Cell Wall Buckling Evaluation

The ½-symmetric model includes all major components of the rack, namely the cell walls, the base plate, the support pedestals, and the reinforcement bars. The model also includes a 10 inch deep by 41.5 inch wide rigid plate (shown in pink in Figure 1), which represents the reinforcement bar on the adjacent rack involved in the impact (referred to hereinafter as rigid impactor). The rack-to-rack impact is simulated in LS-DYNA by applying a pressure time-history on the +y face of the rigid impactor, which is constrained to move only in the y-direction. The peak magnitude of the pressure time-history is arbitrarily set equal to twice the maximum rack-to-rack impact force predicted by DYNARACK under SSE conditions. The duration of the pressure pulse is equal to the duration of the maximum rack-to-rack impact from the DYNARACK simulation. The side of the rack opposite the rigid impactor is fixed against translation.

Figure 2 below shows the displacement time-history for the rigid impactor obtained from the LS-DYNA solution. The onset of buckling is marked by the sudden increase in lateral displacement, which occurs at 0.0325 seconds. The corresponding impact load at t = 0.0325 seconds is 153.5 thousand pounds-force (kips), which is considered to be the critical buckling load for the spent fuel racks under lateral impact. The maximum rack-to-rack impact load predicted by DYNARACK is only 101.8 kips. Therefore, the safety factor against buckling is:

$$SF = \frac{153.5 kips}{101.8 kips} = 1.51$$

This satisfies the minimum requirement of 1.5 per paragraph NF-3321.1(b) of the ASME Code.

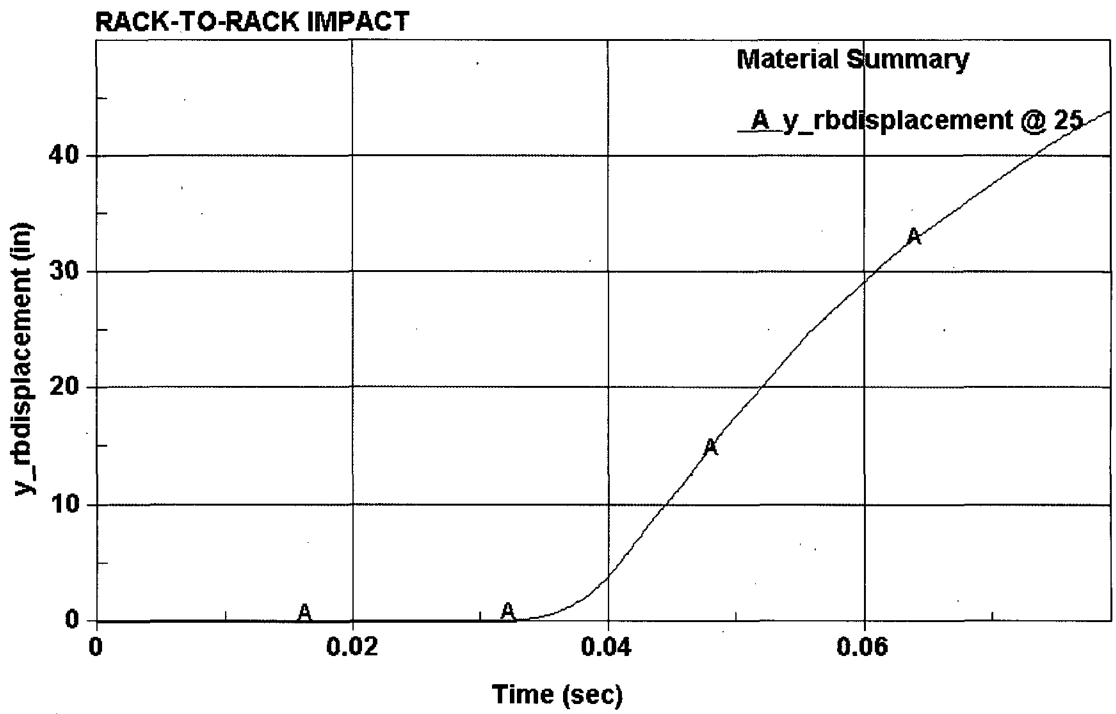



Figure 2 – Lateral Displacement of Rigid Impactor from LS-DYNA Solution

**Supplement to RAI 5:**
In the original response to RAI 5 in Reference 3, global bending of SFP slab was not evaluated due to the slab's small span-to-depth ratio (351 inches/120 inches = 2.93), and the fact that it is founded on grade. However, to quantify the flexural behavior of the spent fuel slab during a rack drop and provide a more complete evaluation, the slab has been conservatively analyzed as a beam on an elastic foundation with a concentrated load at its center using the formula given in Table 7 of *Roark's Formulas for Stress & Strain* (Reference 5). The length of the beam is set equal to the floor span in the short axis direction, which controls one-way bending, and the width of the beam is set equal to the minimum width of the dropped rack. Both ends of the beam are assumed to be fixed where the floor slab intersects the SFP walls. The rack impact load is obtained from the LS-DYNA simulation of the rack drop event (Reference 14). The following table summarizes the data used for the beam analysis.

| Item | Value |
|---|---|
| Beam Length (inches) | 351 |
| Beam Width (inches) | 90.6 |
| Beam Depth (inches) | 120 |
| Subgrade Modulus (kips/ft$^3$) | 260 |
| Concrete Compressive Strength (psi) | 3,000 |
| Modulus of Elasticity of Beam (psi) | $3.12 \times 10^6$ |
| Rack Impact Load [pounds-force (lbf)] | 624,360 |

The calculated moment at the center of the beam, due to the rack drop event (Fd), is combined with the maximum slab bending moments due to dead load (D) and thermal load (To) from References 6 and 16. The moment contributions from each load category, as well as the combined moment, are tabulated below.

| Item | Value (lbf-in/in) |
|---|---|
| Moment due to Dead Load (D) | $1.287 \times 10^5$ |
| Moment due to Thermal Load (To) | $4.939 \times 10^5$ |
| Moment due to Rack Drop (Fd) | $3.002 \times 10^5$ |
| Combined Moment (D + To + Fd) | $9.228 \times 10^5$ |

The combined moment is very conservative since the maximum slab moments due to dead load, thermal load, and rack drop are summed together (in absolute value) irrespective of their location on the slab.

From Reference 6, the ultimate moment capacity of the BVPS Unit No. 2 SFP slab is $1.080 \times 10^6$ lbf-in/in. Thus, for the load combination D + To + Fd, the safety factor against bending failure of the SFP slab is 1.17. Therefore, the postulated rack drop event will not cause a global failure of the SFP slab.

**Supplement to RAI 6:**
A revised analysis has been performed to combine thermal and seismic stresses for the cell-to-cell welds (which are most vulnerable to differential heating across adjoining cells). Both Section 3.8.4 of the Standard Review Plan (SRP) (Reference 7) and Generic Letter (GL) 78-11 (Reference 8) clearly state:

> The temperature gradient across the rack structure that results from the differential heating effect between a full and an empty cell should be indicated and incorporated in the design of the rack structure.

The above condition causes shear stresses to develop in the cell-to-cell welds due to the differential thermal growth between neighboring cells. Per Subsection 5.6.10.2 of Enclosure C to Reference 1, the maximum calculated shear stress in the cell-to-cell welds due to an isolated hot cell is 14,082 psi, which occurs at the top of the rack. Under seismic conditions, the cell-to-cell welds also develop stress due to the flexural behavior of the rack cell structure and the fuel-to-cell wall impact forces. The maximum shear stress in the cell-to-cell welds due to seismic loading is 6,756 psi, which is reported in Subsection 5.6.7 of Enclosure C to Reference 1. When these two stress results are conservatively summed, the combined stress on the weld is still less than the minimum acceptance limit specified in GL 78-11 (that is, 1.5 times normal limits or the lesser of 2Sy [yield stress] and Su [ultimate tensile stress]). The normal (Level A) stress limit for a weld is 0.3 × Su per ASME Code Subsection NF. Thus, in accordance with GL 78-11, the applicable weld stress limit for evaluating thermal plus seismic stresses is 0.45 × Su. For SA-240 304 material at 200 degrees Fahrenheit, 0.45 × Su is equal to 29,790 psi, which is greater than the combined weld stress of 20,838 psi (14,082 psi plus 6,756 psi). In summary, the combined stress (thermal plus seismic) on the cell-to-cell welds for the BVPS Unit No. 2 spent fuel racks is less than the limit given in GL 78-11.

**Supplement to RAI 14:**
The only interaction between the spent fuel racks and the U-shaped beam structure occurs at three discrete locations where specially designed bearing pads are installed to allow a single support pedestal on racks D2, B3, and B4 to rest on top of the sub-base structure (response to RAI 14 in Reference 3 contains more information). At the other 60 support pedestal locations, the nominal clearance gap between the externally threaded portion of the adjustable pedestal (Figure 2.6.3 of Enclosure C to Reference 1) and the U-shaped beam structure is large enough to preclude any contact between the support pedestals and the U-shaped beam structure during a seismic event.

Based on the proposed spent fuel rack layout, the nominal clearance between an adjustable pedestal and the nearest U-shaped beam is at least 3 1/4 inches. Per Section 9.1.1 of Reference 10, the maximum horizontal (sliding) displacement at the base of any BVPS Unit No. 2 spent fuel rack is only 1 inch under SSE load conditions. The internally threaded portion of the adjustable pedestal is located 9 1/8 inches above the SFP floor, which exceeds the height of the U-shaped beam structure and eliminates the potential for lateral impacts. Therefore, other than the three specially designed bearing pad locations, the spent fuel rack pedestals will not interact with the U-shaped beam structure during an earthquake.

The minimum calculated weld safety factor of 1.42 given in the response to RAI 14 in Reference 3 is based on the AISC (American Institute of Steel Construction) allowable stress on welds (that is, 0.30 × nominal tensile strength of the weld). This result, which exceeds the minimum acceptable limit of 1.0, is conservative since the applied load is equal to the maximum pedestal vertical load under seismic (SSE) loading. Per Section A5.2 of the AISC Code (Reference 11), the allowable stresses may be increased by 1/3 when stresses are produced by seismic loading (which is not credited in the analysis for conservatism).

**Supplement to RAI 16:**
FENOC has reviewed the existing sliding and overturning analysis for the BVPS Unit No. 2 Fuel Building. The existing analysis entirely neglects the effects of the spent fuel racks plus fuel when determining the safety factors against sliding and overturning. Subsequently, FENOC performed an assessment of the safety factors against sliding and overturning when including the effects of the proposed new spent fuel racks plus fuel. The resulting safety factors for both the sliding and overturning evaluations were determined to be slightly higher. Therefore, the existing safety factors against sliding and overturning as previously reported in the RAI response are conservative and bounding.

**Supplement to RAI 17:**

To address the staff's comments/concerns, the shallow fuel drop accident has been re-analyzed using LS-DYNA to incorporate the following changes:

1) the stress-strain curve for the weld material is switched from a bi-linear, engineering stress-strain curve to a power law, true stress-strain curve;
2) the uni-axial failure strain limits for the base metal (SA-240 304L) and the weld material (Type 308) are the 98 percent exceedance probability failure strain values based on the data provided in Table B.1 of NUREG-1864 (Reference 12);
3) a lower bound triaxiality factor is applied to the uni-axial failure strain limits at the start of the simulation;
4) a strain rate amplification curve developed from the test data in Reference 13 is applied to both the base metal and the weld material.

Figure 17-1 shows the stress-strain curves for the base metal and the weld material that are input to LS-DYNA for the shallow fuel drop accident analysis. As shown in the figure, the uni-axial failure strain limits for the base metal and the weld material are 0.724 and 0.493, respectively. The uni-axial failure strain limits are calculated according the following relationship:

$$\varepsilon_f = ln\left(\frac{1}{1-q_{98}}\right)$$

where $q_{98}$ is the 98 percent exceedance probability value (i.e., mean value minus two standard deviations) for the reduction in area based on the data provided in Table B.1 of Reference 12 after adjusting for a reference temperature of 150°F.

Figure 17-2 shows the strain rate amplification curve assigned to the base metal and the weld material based on the test data in Reference 13. Based on the derivation presented in the response to RAI 17 in Reference 3, a triaxiality factor of 0.6065 is applied to the uni-axial failure strain limits for the base metal and the weld material to bound the worst-possible bi-axial stress state.

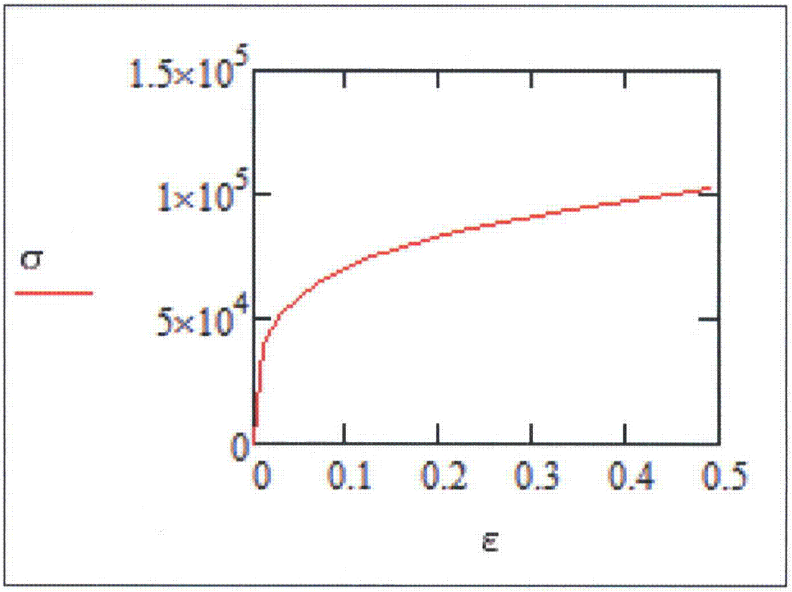
The new results for the shallow fuel drop accident analysis are shown in Figures 17-3 to 17-5 and summarized in the table below.

| Shallow Drop LS-DYNA Analysis Results | |
|---|---|
| Impact Duration (sec) | 0.09 |
| Peak Impact Force (lbf) | 30,748 |
| Fuel Assembly Vertical Displacement (in) | 5.29 |
| Plastic Deformation Measured from the Rack Top (in) | 17.92 |

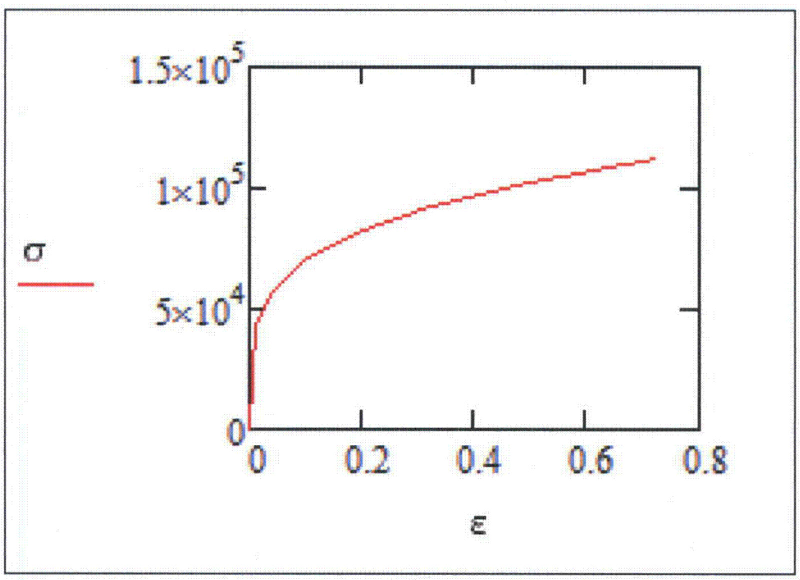
The LS-DYNA analysis results demonstrate that the plastic deformation in the rack cell wall resulting from a shallow drop accident does not extend down into the "neutron absorber zone," which is defined as the vertical length of the cell blanketed by the fixed neutron absorber panel. For the BVPS Unit No. 2 spent fuel racks the minimum distance from the top of the rack to the top edge of the neutron absorber panel (that is, neutron absorber zone) is 19.75 inches. From the LS-DYNA solution, the dropped fuel assembly moves downward crushing the cell wall to a depth of only 5.29 inches, and the plastic strain in the cell wall diminishes to zero at a distance of 17.92 inches below the top of the rack as shown in Figure 17-5. Since the depth of damage by either measure is less than 19.75 inches, the neutron absorber panels do not suffer any damage, and therefore the shallow drop accident has no adverse effect on the criticality safety analysis for the BVPS Unit No. 2 spent fuel racks. In fact, the criticality safety analysis takes no credit for the uppermost 1.3 inches of neutron absorber length, which extends above the active fuel region (assuming the maximum active fuel height and worst-case tolerances). Therefore, taking into consideration the criticality safety analysis, the more precise limit on the permanent deformation to the spent fuel rack due to a shallow drop accident is no greater than 21.05 inches (19.75 inches plus 1.3 inches) measured from the top of the rack. Based on this criterion, the computed safety factor for the shallow drop accident is:

$$SF = \frac{21.05''}{17.92''} = 1.17$$

This demonstrates that the functional capability of the rack (that is, to maintain fuel in sub-critical storage configuration) is not adversely affected by the shallow fuel drop accident, and therefore the acceptance limit given in Table 1 of Reference 7 is satisfied.




(a)




(b)

Figure 17-1: True Stress Strain Curves
(a) Weld; (b) Base Material

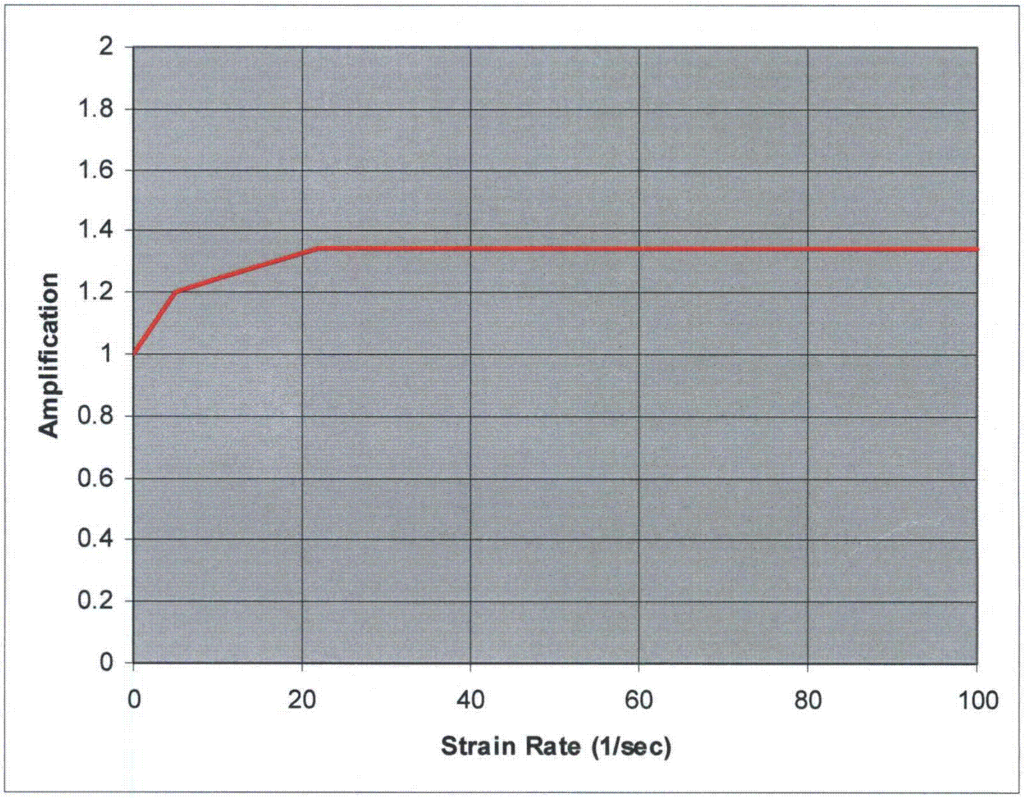



Figure 17-2: Strain Rate Amplification Curve of the Rack Material

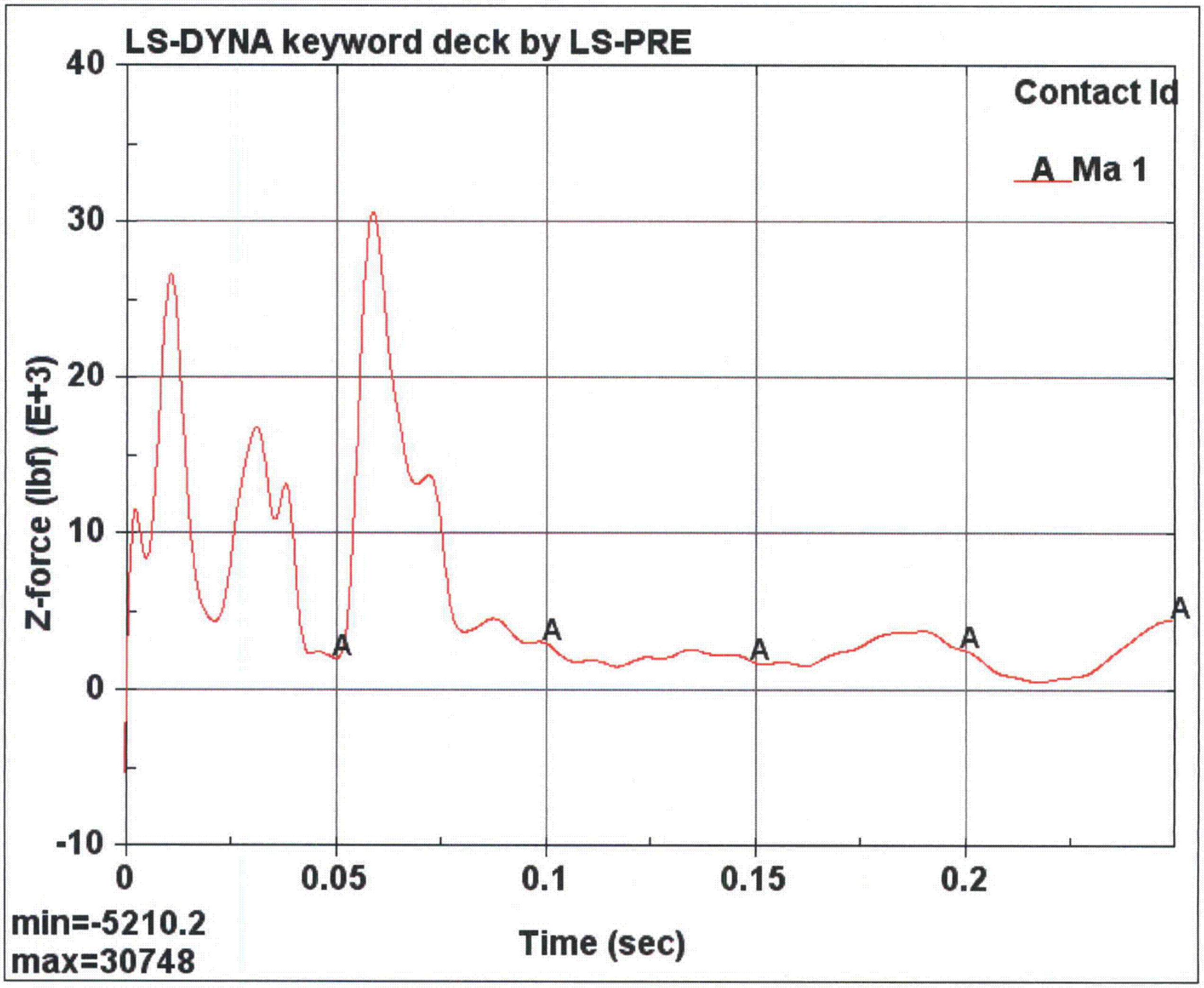



Figure 17-3: Impact Force Time History

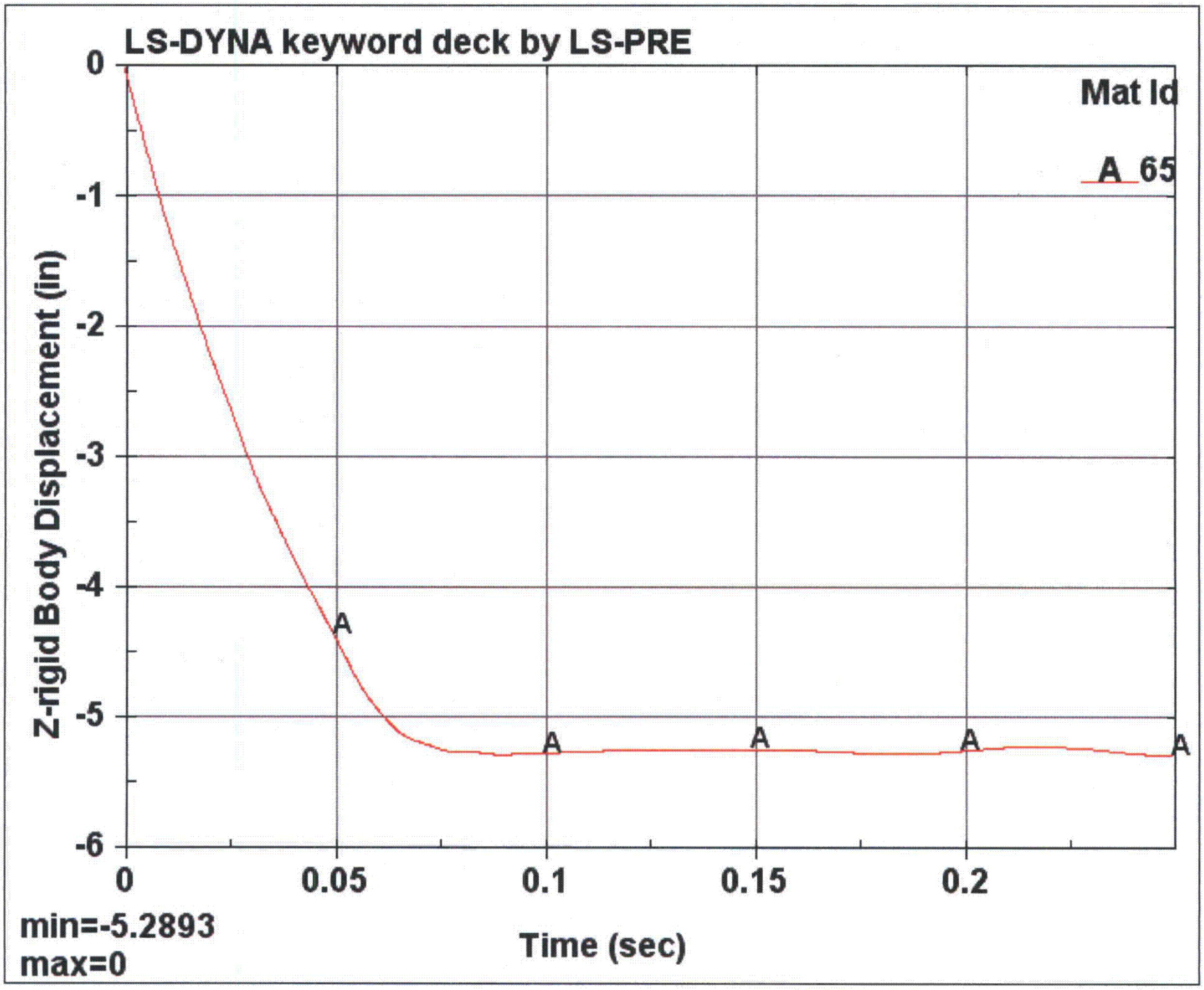



Figure 17-4: Vertical Displacement Time History of the Dropped Fuel During Impact

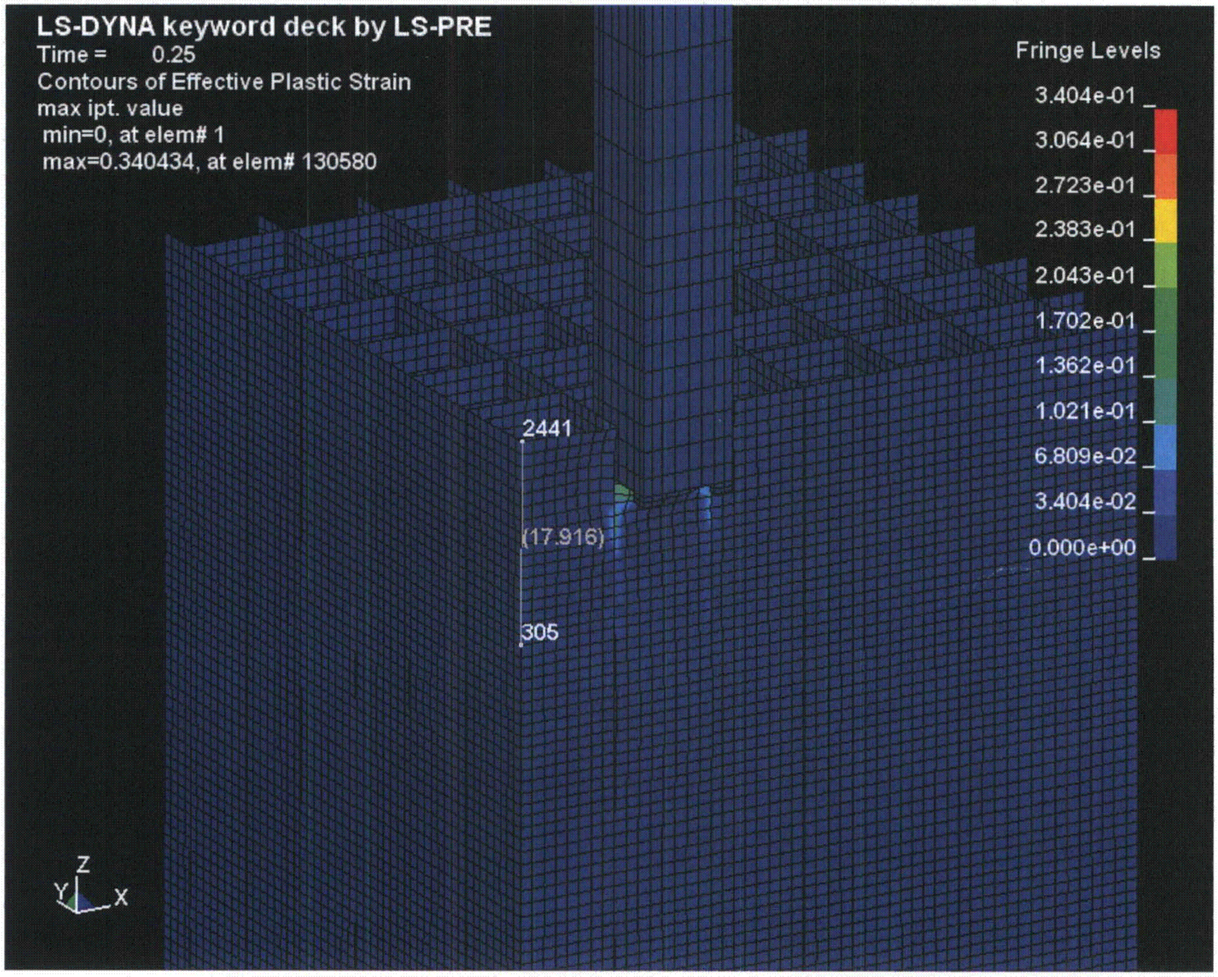



Figure 17-5: Deformed Shape of the SFP Rack after the Shallow Drop Accident

**Supplement to RAI 19:**
The 4,000 psi concrete compressive strength was used as input only for the rack drop analysis performed in previous revisions of Reference 14. The structural evaluation of the reinforced concrete SFP at BVPS Unit No. 2, under the increased loads from the spent fuel racks, uses the design basis minimum concrete strength of 3,000 psi (Reference 6).

The rack drop analysis has been re-performed using the design basis minimum strength of 3,000 psi. Apart from the change in concrete strength, no other changes to the LS-DYNA simulation model have been made. The results of the re-analysis are summarized below.

The rack drop accident analysis results for 3,000 psi concrete are shown in Figures 19-1 to 19-3 and summarized in the table below. The maximum plastic strain of the SFP floor liner is well below the failure strain of 0.4 for the liner material, indicating that there is no loss of water after the rack drop event. Nevertheless, the SFP floor slab will experience limited local damage as shown in Figure 19-3 due to excessive compressive stress resulting from the impact load at the pedestal/liner interface. In addition, the dropped rack also experiences local plastic deformation in the cells adjacent to the pedestal.

| **Rack Drop Analysis Results Based on 3,000 psi Concrete** | |
|---|---|
| Impact Duration (seconds) | 0.62 |
| Peak Impact Force on the SFP Floor (pounds-force) | $1.59 \times 10^{5}$ |
| Maximum Plastic Strain of the SFP Liner | $8.82 \times 10^{-3}$ |
| Number of Plastically Deformed Cells Near Pedestal | 8 |

A dropped rack will suffer some local plastic deformation and the accident will result in limited local damage in the concrete slab. However, the SFP floor liner will not be punctured.

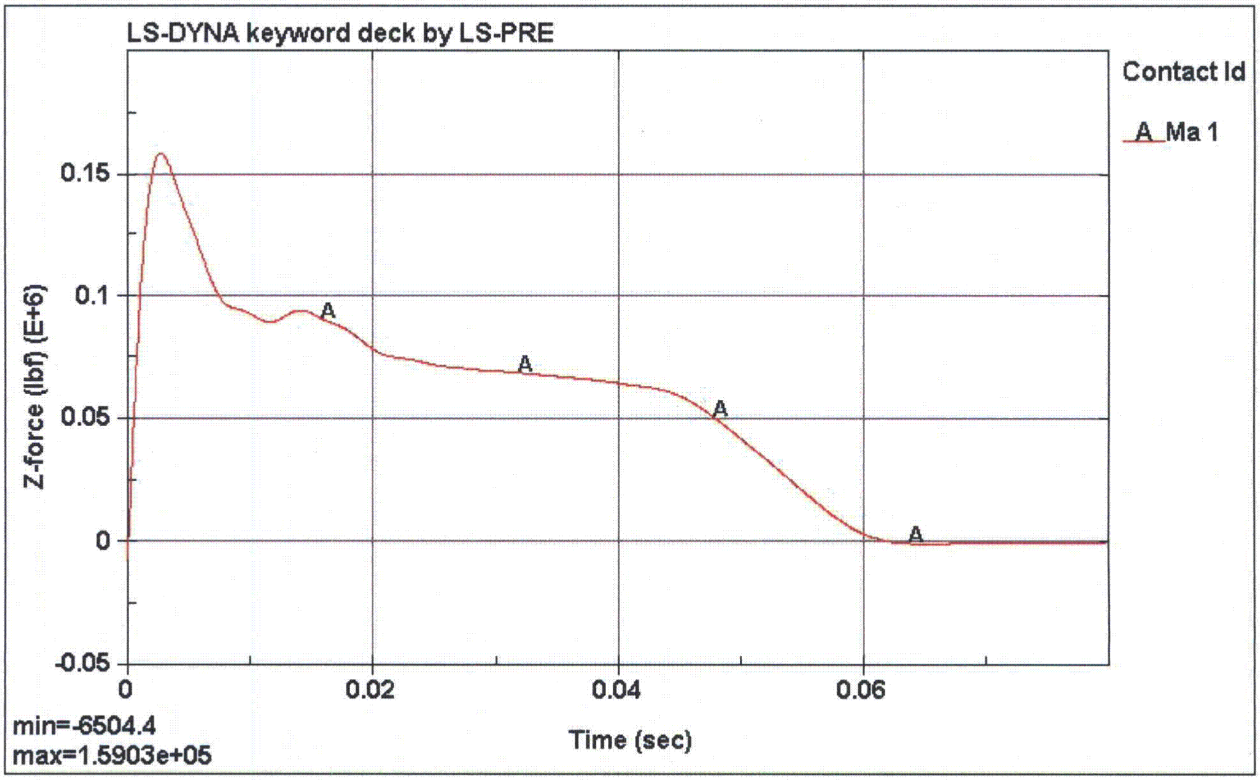



Figure 19-1: Rack Drop Impact Force Time History

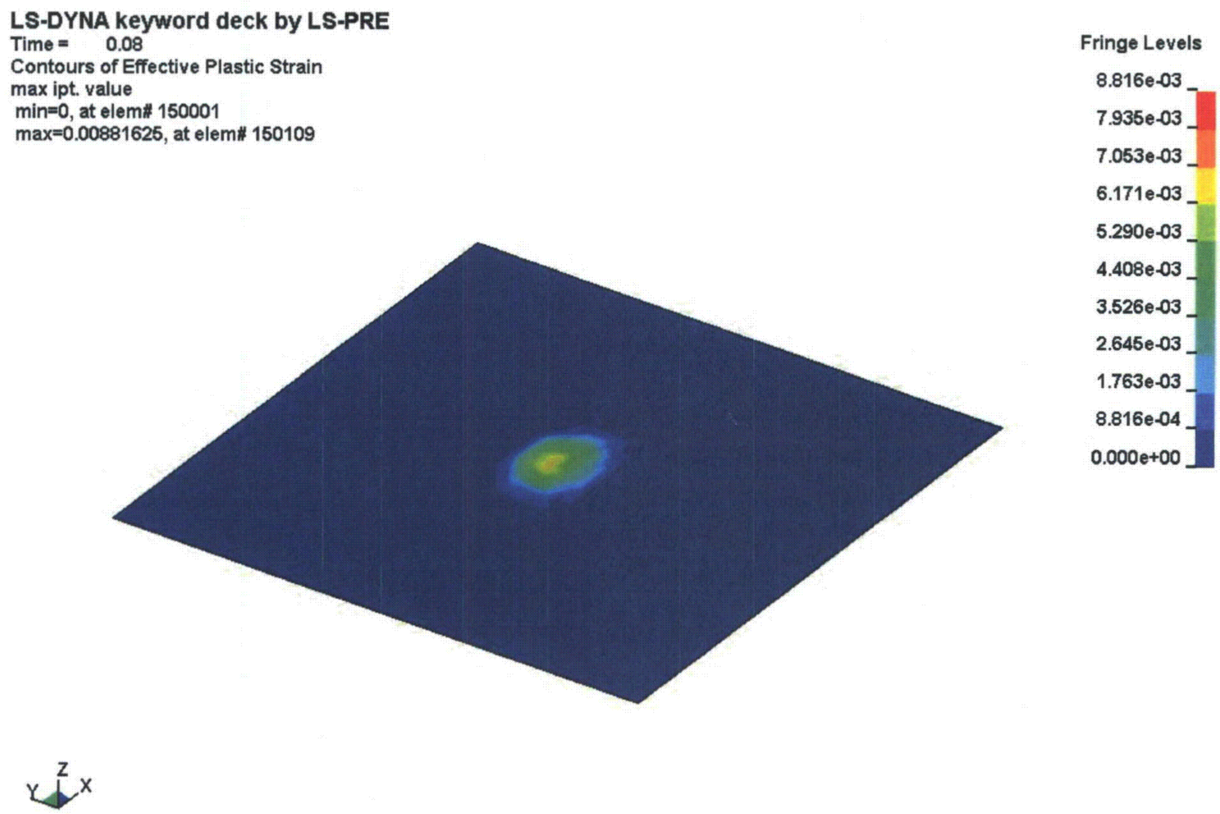



Figure 19-2: Maximum Plastic Strain of the SFP Floor Liner

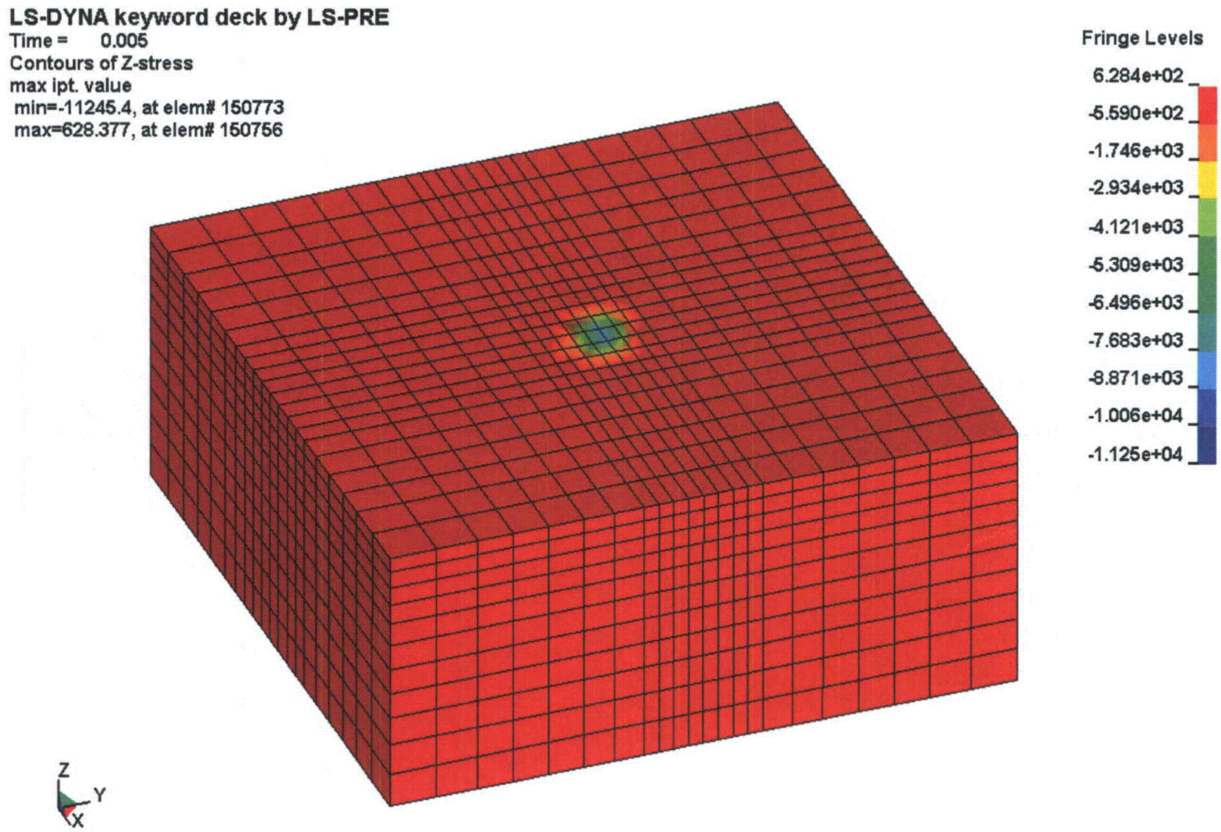



Figure 19-3: Maximum Stress of the SFP Concrete Floor

**References:**

1. FENOC Letter L-09-086, "License Amendment Request No. 08-027, Unit 2 Spent Fuel Pool Rerack," dated April 9, 2009 (Accession No. ML091210251).
2. FENOC Letter L-10-001, "Response to Request for Additional Information for License Amendment Request No. 08-027, Unit 2 Spent Fuel Pool Rerack (TAC No. ME1079)" dated January 18, 2010 (Accession No. ML100191805).
3. FENOC Letter L-10-121, "Response to Request for Additional Information for License Amendment Request No. 08-027 (TAC No. ME1079)," dated May 3, 2010 (Accession No. ML101260059).
4. Timoshenko, S. and Gere, J., Theory of Elastic Stability, McGraw-Hill Book Company, 2nd Edition.
5. Young, W.C., Roark's Formulas for Stress & Strain, McGraw-Hill Book Company, 6th Edition.
6. Holtec Report No. HI-2084131, "Fuel Pool Structural Evaluation of Beaver Valley Unit 2." Revision 5.
7. NUREG-0800, Section 3.8.4, "Other Seismic Category I Structures," Revision 2, March 2007.
8. Generic Letter 78-011, "USNRC OT Position for Review and Acceptance of Spent Fuel Storage and Handling Applications," dated April 14, 1978, and January 18, 1979 amendment.
9. Holtec Drawing No. 5606, "Unit 2 Spent Fuel Storage Racks Replacement," Revision 6.
10. Holtec Report No. HI-2084123, "Structural/Seismic Analysis for Beaver Valley Unit 2," Revision 4.
11. AISC Steel Construction Manual, 9th Edition.
12. NUREG-1864, "A Pilot Probabilistic Risk Assessment of a Dry Cask Storage System at a Nuclear Power Plant," March 2007.
13. D.K. Morton, S.D. Snow, T.E. Rahl and R.K. Blandford, "Impact Testing of Stainless Steel Material at Room and Elevated Temperatures," ASME Pressure Vessel and Piping Conference, San Antonio, TX, PVP2007-26182, ASME, NY, NY, July 2007.
14. Holtec Report No. HI-2084010, "Mechanical Drop Accident Analyses Supporting Beaver Valley Unit 2 Reracking Project," Revision 2.
15. American Society of Mechanical Engineers Boiler and Pressure Vessel Code (ASME Code), Section III, Subsection NF, 1998.
16. Holtec Report HI-90567, "Fuel Pool Structural Analysis of Beaver Valley Unit 1 with Maximum Density," Revision 2.

By letter dated June 11, 2010 (Reference 1), the NRC issued an RAI regarding Section 6.6 of the Holtec Report No. HI-2084175 which describes a methodology for calculating bulk spent fuel pool (SFP) temperature. The staff's request is provided below in bold text followed by the FENOC response.

1. **Section 6.6 of the Holtec Report No. HI-2084175, "Licensing Report for Beaver Valley Unit 2 Rerack", which supports the BVPS Unit No. 2 license amendment request, described a methodology for calculating bulk SFP temperature. This methodology included an allowance for evaporative cooling from the SFP surface. The NRC staff found this to be a non-conservative change in the methodology for determination of the maximum decay heat rate that may be placed in the SFP, compared to that included in the current licensing basis.**

   **Identify the methodology that will be used to control the total decay heat rate of fuel placed in the SFP. Explain how the methodology will maintain current licensing basis margins to pool temperature limits and pool time to boil. Provide appropriate test data and benchmarking to support any assumptions or heat transfer modes included in the methodology that are different from those included in the current licensing basis.**

Response: Credit for evaporative cooling has been removed from the BVPS Unit No. 2 Spent Fuel Pool Rerack Bulk Thermal Analysis HI-2084128. The revised calculation takes no credit for any passive heat losses. The analyses in HI-2084128 (all revisions including the latest one) use the industry accepted ORIGEN2 code for determining fuel assembly decay heat as a function of time. The revised calculation includes a determination of the maximum allowable component cooling water inlet temperature as a function of full core offload initiation time. The calculated maximum allowable component cooling water inlet temperature is less restrictive than the established administrative control. FENOC committed to establish the current administrative control in a letter to the NRC dated October 29, 2001.

In the revised calculation, total decay heat is a function of fuel design, reactor design, and operations only. The calculation assumes bounding parameters (maximum burnup, maximum reactor power and 100 percent capacity factor) for all projected future operations to maximize the total decay heat. The net SFP heat load (which goes to the SFP heat exchangers) is the total decay heat load plus the contribution from any other heat sources (that is, pump motor heat), with no deduction due to heat losses to the SFP solid structures or evaporative, convective, or radiative heat losses to the fuel building atmosphere.

The table below provides a comparison of a few key results between the current design basis calculation and the revised rerack SFP bulk thermal calculation.

| Parameter | Current Design Basis Calculation Result | Revised Rerack Calculation Result |
|---|---|---|
| Normal Full-Core Offload Refueling Condition | | |
| Peak Fuel Pool Bulk Temperature | 170.0°F | 170.0°F |
| Bounding Minimum Time-to-Boil | 3.42 hours | 2.24 hours |
| Abnormal Full-Core Offload Refueling Condition | | |
| Peak Fuel Pool Bulk Temperature | 172.7°F | 170.3°F |
| Bounding Minimum Time-to-Boil | 2.58 hours | 1.87 hours |

As shown in the table above, the calculated peak fuel pool bulk temperature results for the revised rerack calculation are the same as or better than the results for the current design basis calculation; however, the calculated bounding minimum time-to-boil results for the revised rerack calculation are lower than the results for the current design basis calculation. These time-to-boil results are acceptable because the normal full-core offload refueling condition time is greater than two hours and the abnormal full-core offload refueling condition time represents a worst-case scenario. In both cases, in the unlikely event of a failure of forced cooling to the SFP, there would still be adequate time to initiate corrective measures before boiling began in the SFP.

**References:**

1. **NRC Letter dated June 11, 2010, titled "BEAVER VALLEY POWER STATION, UNIT NO. 2 - REQUEST FOR ADDITIONAL INFORMATION RE: SPENT FUEL POOL RERACK LICENSE AMENDMENT (TAC NO. ME1079)" (Accession No. ML101380546).**

On July 7, 2010, a teleconference was held between the Nuclear Regulatory Commission staff (NRC) and FirstEnergy Nuclear Operating Company (FENOC) to discuss RAI number 8 response provided in FENOC letter dated May 21, 2010 (Reference 4) related to the Beaver Valley Power Station (BVPS) Unit 2 spent fuel pool (SFP) rerack license amendment request No. 08-027. To complete its review, the NRC requested that FENOC docket supplemental information in response to RAI number 8. The information requested is provided below.

**Supplement to RAI 8:**
The following table summarizes the punching (two-way) shear load, the two-way shear capacity, and the corresponding safety factor for each of the SFP walls.

| SFP Wall | Shear Load (kips) | Shear Capacity (kips) | Safety Factor |
|---|---|---|---|
| East | 2,040 | 4,644 | 2.28 |
| West | 3,045 | 10,308 | 3.38 |
| North | 4,226 | 20,503 | 4.85 |
| South | 2,720 | 17,444 | 6.41 |

The two-way shear evaluation conservatively assumes that each wall is supported on only three sides (top edge is free). Accordingly, the shear capacity values in the above table are equal to the total sum capacity of the three-sided support perimeter of each wall evaluated at a distance d/2 from its perimeter edges (d is the distance from the extreme compression fiber to the center of the tension reinforcement). In accordance with Section 11.10.3 of Reference 5, the two-way shear capacities are computed based on a permissible concrete shear stress of:

$$v_c = 4\sqrt{f_c'} = 219 psi$$

where $f_c'$ is the design compressive strength of concrete of 3,000 psi. The tabulated shear load values are equal to the maximum perpendicular shear load on each wall based on the following factored load combinations, whichever is more limiting:

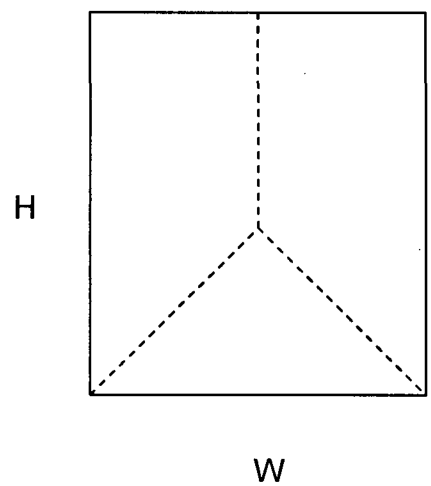
1) 1.4D + 1.9E
2) D + To + E'

where D = dead loads
E = loads generated by the operating basis earthquake
E' = loads generated by the SSE
To = normal thermal loads

The following table summarizes the results for the one-way shear evaluation of the spent fuel pool (SFP) walls.

| SFP Wall / Edge | Shear Load (kips) | Shear Capacity (kips) | Safety Factor |
|---|---|---|---|
| East / Vertical | 882 | 950 | 1.08 |
| West / Bottom | 1,109 | 1,483 | 1.34 |
| North / Bottom | 2,059 | 3,836 | 1.86 |
| South / Bottom | 856 | 2,422 | 2.83 |

For the one-way shear evaluation, the total shear load on each wall is divided between the vertical edges and the bottom edge based on their tributary area. This means that for a wall of height H and of width W the load is divided as follows:




Bottom Edge: $$\beta = \frac{W^2/4}{WH} = \frac{W}{4H}$$

Vertical Edge: $$\alpha = \frac{1-\beta}{2}$$

This approach yields a conservative result for the load on the vertical edges since the hydrostatic pressure on the wall tends to bias the load towards the bottom edge. To insure that the load on the bottom edge is also conservative, a factor of two is applied to the value of $\alpha$ for all walls except the east wall. For the east wall, the load on the bottom edge is calculated more precisely by considering the pressure on the wall at the centroid of the triangular load area.

The shear capacity values in the preceding table are equal to the shear capacity across the full width of the bottom edge or the full height of the vertical edge, as applicable. In accordance with Section 11.4.1 of Reference 5, the one-way shear capacities are based on a permissible concrete shear stress of:

$$v_c = 2\sqrt{f_c'} = 109 psi$$

The SFP reinforced concrete slab is 10-feet thick and founded on grade; therefore, a shear failure of the SFP slab is not limiting. This is evident from the response to RAI number 5 (Reference 6), which shows that the slab has a safety factor greater than 13 (without taking credit for the subgrade) against punching shear failure due to the rack drop event. The safety factors against shear failure for the east-south wall and the south-east wall are bounded by the safety factor for the east wall in the above tables. This is because (a) the east-south wall and the south-east wall are at least two times thicker than the east wall and (b) the wetted area of the east-south wall and the south-east wall are significantly less than the east wall.

The maximum concrete temperature and the corresponding temperature gradients through the SFP walls and slab are calculated in Holtec report HI-2084135 (Reference 2). Since the thermal moment is directly proportional to the square of the wall (or slab) thickness and the corresponding temperature gradient, an adjustment factor is calculated for each wall/slab, which enables the thermal moments for the BVPS Unit No. 2 SFP to be calculated from the BVPS Unit No. 1 moment results (Reference 3). As described in Section 5.9 of Enclosure C in Reference 1, linear interpolation from the BVPS Unit No. 1 moment results is used to calculate the results for BVPS Unit No. 2. Once the thermal moments for BVPS Unit No. 2 are known, the applicable American Concrete Institute load factors are applied and the final safety factors (shown in Section 5.9 of Enclosure C in Reference 1) against bending (moment) failure are determined for the BVPS Unit No. 2 SFP structure.

The shear safety factors reported above are not adversely affected by the temperature rise in the pool since thermal loads tend to cause net compression in the SFP slab and wall cross-sections, which has a positive (increasing) effect on shear capacity. The SFP liner is not analyzed for the effects of the SFP temperature rise since the liner is designated as seismically qualified, but non-safety related (Seismic Category II) according to Section 9.1.2.3 of the BVPS Unit No. 2 Updated Final Safety Analysis Report.

**References:**


1. **Letter from P. P. Sena, FirstEnergy Nuclear Operating Company, to NRC Document Control Desk, "Beaver Valley Power Station, Unit No. 2 Docket No. 50-412, License No. NPF-73, License Amendment Request No. 08-027, Unit 2 Spent Fuel Pool Rerack," with Enclosure B (proprietary) and Enclosure C (non-proprietary), "Licensing Report for Beaver Valley Unit 2 Rerack," dated April 9, 2009. (ADAMS Accession Nos. ML091210251 (letter) and ML091210263 (Enclosure C))**

2. **Holtec Report HI-2084135, "Beaver Valley Unit 2 Spent Fuel Pool Structure Temperature Gradients," Revision 4.**

3. **Holtec Report HI-90567, "Fuel Pool Structural Analysis of Beaver Valley Unit 1 with Maximum Density," Revision 2.**

4. **FENOC Letter L-10-151, "Response to Request for Additional Information Related to Beaver Valley Power Station Unit No. 2 Spent Fuel Pool Rerack License Amendment Request (TAC No. ME1079)," dated May 21, 2010 (Accession No. ML101460057).**

5. **American Concrete Institute, "Building Code Requirements for Reinforced Concrete (ACI 318-71)."**

6. **FENOC Letter L-10-121, "Response to Request for Additional Information Related to Beaver Valley Power Station Unit No. 2 Spent Fuel Pool Rerack License Amendment Request (TAC No. ME1079)," dated May 3, 2010.**