NRC Public Meeting 9/27/10
Open Items for Resolution and Proposed Approaches

RAI #5

Open Items

1. In August 9, 2010 supplemental response, moment due to Dead Load (D) is discussed. Provide further information relative to what is included in this load case. Specifically, provide confirmation that the weight of the water is included in the base slab evaluation.

   Proposed Approach: For the base slab evaluation presented in the August 9, 2010 supplemental response, the Dead Load (D) includes the self-weight of the reinforced concrete slab, the weight of the SFP water, and the buoyant weight of the spent fuel racks plus fuel. With regard to the SFP water weight, a uniform pressure of 17.33 psi was applied to the top surface of the SFP slab to account for the 40-foot height of water in determining the moment due to Dead Load (D). This information will be captured in the upcoming RAI submittal.

2. The LAR states the following:
   "To confirm the structural integrity of the racks, it is necessary to demonstrate compliance with the USNRC Standard Review Plan [5.1] and the OT Position Paper [5.2]."
   Both Appendix D to SRP 3.8.4 and the OT position paper (GL 78-11) state that the ductility ratios utilized to absorb kinetic energy should be quantified.
   Provide further information and justify that the total strain in the base slab [strain due to the rack impact plus the existing strain in the base slab due to other loads (dead load, water load, etc.)] is within the acceptance limit determined based on the allowable ductility ratios specified in the BVPS-2 UFSAR.

   Proposed Approach:
   In order to address this open item, the rack drop analysis will be re-performed (in lieu of considering the slab as a beam strip on an elastic foundation and statically applying the loads). The new LS-DYNA model will have the following characteristics:

   - the full extent of the reinforced concrete SFP slab will be modeled using solid elements;
   - the tensile reinforcement at the base of the SFP slab will be modeled using a layer(s) of thin shell elements;
   - the SFP liner covering the slab will be modeled using thin shell elements;
   - if necessary, the soil underlying the SFP slab will be modeled using solid elements (initial solution will assume no credit for the underlying soil);
   - the dropped rack will be modeled in the same manner as the original LS-DYNA "local" rack drop model.

The weight of the SFP water, the spent fuel racks, and the stored fuel will be accounted for in the model by applying an equivalent pressure load to the top surface of the SFP liner. Prior to solving for the rack impact event, a 1-g acceleration vector will be globally applied to the model to initialize the stresses and strains in the SFP slab under normal load conditions (i.e., weight of the concrete slab, SFP water, spent fuel racks, and stored fuel). After the stresses and strains in the slab due to normal load conditions are fully developed in LS-DYNA, the rack drop event will be initiated in LS-DYNA while the pressure load on the SFP liner and the 1-g global acceleration remain active. This LS-DYNA model will address both the local and global response of the slab in one solution and in doing so avoid potential concerns regarding compatibility of models (i.e., local vs. global, static vs. dynamic). To verify the accuracy of the solution, the results from the new LS-DYNA model will be compared against the results from the "local" rack drop model (see RAI # 19) or an alternate calculation.

The acceptance criterion for the rack drop event will remain as previously stated in Section 7.2 of the Licensing Report (Holtec report HI-2084175). That is the rack drop event shall not result in catastrophic damage to the SFP structure leading to a rapid loss of pool water inventory. This will be demonstrated by showing that the SFP liner is not breached and the tensile strain in the steel reinforcement under the impact load remains within the acceptance limit determined based on the allowable ductility ratios (μ) specified in the BVPS-2 UFSAR Section 3.5.3, namely:

Reinforced concrete – beam column members and slabs (tension controlling) $\mu \leq 10$
Reinforced concrete – beam column members and slabs (compression controlling) $\mu \leq 1.3$
Reinforced concrete – beam column members and slabs (in region requiring shear reinforcement) $\mu = 1.3$

3. Provide the basis for the value of subgrade modulus (260 kips/ft^3). In addition, provide further information and confirm that the uncertainty in the value of the subgrade modulus has been taken into account or provide justification that excluding the variation of subgrade modulus has minimal effects on the outcome of the analysis.

Proposed Approach: The value of the subgrade modulus used in the previous analysis ($260\, kips / ft^3$) was obtained from a 1976 report prepared by Stone & Webster investigating the effects of a spent fuel rack capacity expansion on the SFP structure. The design basis minimum subgrade modulus for BVPS-2 is $150\, kips / ft^3$. This value was used in the design basis calculations for the fuel building. Therefore this value will be used (if necessary) as an input to the LS-DYNA rack drop model discussed above.

The modulus of subgrade reaction was determined from test load results on a one foot by one foot plate. From modulus of subgrade reaction for a one foot square

plate of 580 $kips / ft^3$ the subgrade reaction was determined for the Unit 2 structures. The subgrade reaction for the fuel building was determined from the size effect ratio formula for a rectangular footing. The subgrade reaction for the fuel building was calculated to be $150\, kips / ft^3$. The uncertainty associated with the value of subgrade modulus will be addressed by demonstrating that the moment and shear forces in the mat are not sensitive to this parameter.

RAI #6

Open Items

1. Appendix D to SRP 3.8.4 states that the structural acceptance criteria are those given in Table 1. OT position paper (GL 78-11) states "When subsection NF, Section III, of the ASME B&PV Code is used for the racks, the structural acceptance criteria are those given in the Table below." The tables in the OT position paper (GL 78-11) and Appendix D to SRP 3.8.4 provide consistent guidance relative to the load combinations and corresponding acceptance limits where seismic and temperature loadings are combined.
   The August 9, 2010 supplemental response discusses the shear stresses in the cell-to-cell welds due to the differential thermal growth between neighboring cells. Provide justification that the evaluation described in this response is the enveloping condition for the proposed rack design or provide supplemental information to demonstrate compliance with the guidance defined in Appendix D to SRP 3.8.4 and OT position paper (GL 78-11).

   Proposed Approach: In order to address the above open item, a bounding 3-D temperature distribution will be defined for the largest spent fuel rack, which maximizes the thermal gradient across the cell structure in the vertical and horizontal directions, and the resulting thermal stresses in the rack will be calculated using ANSYS.

   The bounding 3-D temperature distribution will be conservatively determined based on the existing calculations for the SFP bulk water temperature and maximum SFP local water temperature following an emergency full core offload (i.e., Ta). It will be conservatively assumed that the rack is half loaded with freshly discharged fuel, and the other half is empty. Thus, for the loaded cells the applied temperatures will be derived from the existing CFD analysis that establishes the maximum SFP local water temperature inside the hottest loaded storage cell and characterizes the vertical temperature gradient from the rack baseplate to the top of rack. For the empty cells, the applied temperature throughout the empty region will be equal to the calculated SFP bulk water temperature. The temperature distribution described above is considered to bound the realistic conditions in the SFP since:

   a) there will be an abrupt (step) increase in the rack metal temperature at the boundary between the empty and loaded regions of the rack;

b) the temperatures applied to the loaded cell region will be based on the maximum water temperatures inside the hottest cell as the water flows vertically upward around the fuel assembly (in reality the cell wall temperature would be slightly less than the maximum water temperature inside the cell).

The bounding temperature distribution will then be applied to a 3-D ANSYS model of the spent fuel rack, in which each storage cell location, and its cell-to-cell connections, is individually modeled. The thermal stress results obtained from ANSYS will be combined with the stress results from DYNARACK in accordance with the load combinations provided in the OT Position Paper. The combined stress results will then be compared against the acceptance limits provided in the OT Position Paper (including January 18, 1979 amendment thereto) and Appendix D to SRP 3.8.4.

RAI #8

Open Items

1. Based on load combinations for concrete structures defined in Section 3.8.3.3 of BVPS-2 UFSAR, Ta should be combined with E' and 1.25E. Load combination (2) in the August 9, 2010 supplemental response includes To (normal condition temperature) rather than Ta (accident condition temperature).
   Provide confirmation that appropriate thermal loads, according to the BVPS-2 design basis requirements, have been used in the evaluation of SFP structure.

   Proposed Approach: The load combinations per UFSAR Section 3.8.3.3 for concrete structures will be used. Specifically, the governing equations to be used are as follows;

   $U = 1.4D + 1.9E$ (Eq. 3.8-3)
   $U = 1.0D + 1.0Ta + 1.25E$ (Eq. 3.8-8)
   $U = 1.0D + 1.0Ta + 1.0E'$ (Eq. 3.8-9)
   $U = 0.75(1.4D + 1.9E + 1.7To)$ (Eq. 3.8-10)

   Where: U = required section strength
   D = dead loads
   To = normal thermal loads
   Ta = abnormal thermal loads
   E = loads generated by OBE
   E' = loads generated by the SSE

2. The August 9, 2010 supplemental response does not discuss the capacity reduction factor (Phi factor). Please confirm that appropriate Phi factors have been considered for two-way and one-way shear evaluations.

Proposed Approach: Per Section 9.2.1.3 of ACI 318-71, a capacity reduction factor, ϕ, equal to 0.85 was considered in the one-way and two-way shear evaluations discussed in the August 9, 2010 supplemental response.

3. Provide further Justification that the tributary area methodology used in one-way shear evaluation is conservative.

   Proposed Approach: To validate the tributary area methodology and quantify the degree to which it is conservative, a simple finite element model of the SFP East Wall (which is limiting based on the current one-way shear evaluation) will be created in ANSYS and analyzed for two different load cases. In the first load case, a uniform pressure will be applied over the entire face of the wall, and the nodal reaction forces will be summed along each of the fixed edges (3 sides) to determine the shear force distribution. The ANSYS results will then be compared with the results obtained using the tributary area methodology for the same size wall and applied pressure. In the second load case, a linear varying pressure load will be applied to the model to reflect the actual loading on the SFP East Wall for the load combination 1.4D + 1.9E (which is limiting based on the current one-way shear evaluation). The shear forces on the two vertical edges and the bottom edge as determined from ANSYS will be compared with the shear forces considered in the one-way shear evaluation.

4. The August 9, 2010 supplemental response states that the liner plate is not analyzed because it is designated as seismic category II. However, according to Section 3.2.1.2 of BVPS-2 UFSAR, a seismic category II classification is identified as those SSCs whose failure could adversely affect safety-related, seismic category I components. Therefore, the response does not adequately address the effects of SFP temperature rise on the liner plate integrity.

   Proposed Approach:
   The Unit 2 spent fuel pool liner consists of ¼ inch plate with vertical stiffeners spaced every 18 inches on center. The stiffeners consist of structural T-shapes welded to the liner plate. The T-shapes anchor the liner to the concrete. Welded headed studs were not used to anchor the liner plate to the concrete.
   The spent fuel pool design basis calculations for the liner were reviewed. The liner design used the maximum delta t postulated for the SFP. This was based on the boiling temperature of the pool at the site elevation. Therefore the existing liner calculation bounds the thermal loads for the SFP re-rack.

5. The August 9, 2010 supplemental response states: "The shear safety factors reported above are not adversely affected by the temperature rise in the pool since thermal loads tend to cause compression in the SFP slab and wall cross sections, which has a positive (increasing) effect on shear capacity." Contrary to this statement, the temperature rise tends to expand the SFP liner and since the liner is anchored to the concrete, the liner expansion will induce a tensile load on the wall.

Provide further information that the effects of the SFP liner heat-up has been included in the evaluation of the SFP concrete structure according to the BVPS-2 design basis requirements.

Proposed Approach: The reviewer is correct that the thermal expansion of the SFP liner will induce tensile forces in the SFP walls. It also bears noting that the temperature rise will cause the reinforced concrete SFP structure to expand as well, and since the adjoining slabs and walls are at lower temperature, the concrete expansion tends to induce compressive loads in the walls. The referenced statement from the August 9, 2010 supplemental response was based upon the latter effect.

To address the effect of the liner, a conservative estimate of the maximum induced tensile stress in each SFP wall will be made by considering the maximum temperature rise in the liner (i.e., difference between maximum SFP bulk water temperature and stress-free temperature (70°F)) and the relative coefficient of thermal expansion between stainless steel and concrete. For each wall, the permissible shear stress in the concrete will be adjusted according to formula given in Section 11.4.4 of ACI 318-71 to account for the induced tensile stress in the wall due to liner expansion. The moment safety factors for the various wall sections will also be reviewed and adjusted as necessary to account for tensile loads.

RAI #17

Open Items

1. Using the Hancock-MacKenzie model to determine failure strain is not acceptable (triaxial factor of 0.6065). The triaxial factor needs to be 0.5 based on more recent models (As discussed during the public meeting, the NRC documented its acceptance of a minimum triaxiality factor of 2.0 for cases of biaxial tension in the safety evaluation report (SER) written for Amendment 7 to the Certificate of Compliance (CoC) for the Holtec International (HI)-STORM 100 cask system (located in ADAMS at ML093620075).

   Proposed Approach: The shallow drop analysis will be re-performed in LS-DYNA using a constant triaxiality factor of 0.5. In other words, the failure strain limit for the base metal and weld material will be input in LS-DYNA as 0.5 times the 98-percent exceedance uniaxial failure strain limit and it will remain constant throughout the simulation.

2. Document the basis for using the INL paper for determining the strain rate amplification curve for the weld material. Additionally, the supplemental information will document the discussion regarding RAI 17 to provide explicit details regarding the material model curve which was used in the LS-DYNA analysis. This will include information relative to where the strain rate

amplification was applied (i.e., only the stress was increased) and where the triaxiality factor was applied to provide a clear understanding of what the value for the true failure is for the shallow-drop accident analysis. Coupled with this discussion, justification regarding the strain amplification value(s) used and at what point during the analysis these values are used will be provided. This justification should focus on how the amplification factor is applied to the true stress-strain curve in the LS-DYNA model, i.e. describing whether an iterative procedure is used in LS-DYNA to capture the strain rate value and then apply the corresponding amplification factor or whether a constant strain rate amplification factor is applied to the true stress-strain curve.

Proposed Approach: The shallow drop analysis will be re-performed using the strain rate amplification data from the INL paper for the base metal material only. No strain rate amplification factors will be applied to the weld material. The updated response will also provide the details of the material model curves, including the precise manner in which the triaxiality factor and the strain rate amplification factors are applied to the material model. In brief, the strain rate amplification curve from the INL paper (see Figure 17-2 from August 9, 2010 RAI submittal) will be input into LS-DYNA for the base metal material. Throughout the solution, the strain rate amplification factor for each base metal element will be continually updated based on the current strain rate for that element (i.e., an iterative procedure will be used). The strain amplification factor will be applied only to the stress values in true stress-strain curve for the base metal material. See Proposed Approach to Open Item 1 above for manner in which triaxiality factor will be applied.

In conjunction with the above changes in the material modeling, the finite element model of the impacted rack will be updated to include the ¼" thick x 10" deep reinforcement bar (see Supplement to RAI # 2 in August 9, 2010 submittal for more information on reinforcement bar), which is welded to the exterior cell walls of the rack (above the neutron sheathing) around the entire rack perimeter. [Note: The current shallow drop analysis conservatively ignores the presence of the reinforcement bar.] Since the reinforcement bar reinforces only the perimeter cell walls, the shallow drop analysis will be expanded to consider two drop locations, namely:

i) a vertical fuel assembly drop from a height of 24" above the rack onto a perimeter cell wall (same as current);
ii) a vertical fuel assembly drop from a height of 24" above the rack onto an interior cell wall.

The acceptance criterion for the shallow drop analysis will remain the same. That is the damage to the rack shall not impair the rack's ability to maintain the fuel in a subcritical storage configuration. In other words, the permanent damage to the rack due to the fuel assembly impact shall not extend downward to the active fuel zone (i.e., damage shall be limited to the upper 21.05" of the rack cell structure).

RAI #19

Open Items

1. RAI-19 stated the following:
   "Please confirm that concrete properties utilized in the proposed reracking LAR are in compliance with the BVPS-2 licensing basis."
   To completely respond to this RAI, the staff requests that the licensee confirm that the concrete properties (compressive strength, elastic modulus, etc.) used in all evaluations/calculations performed for the proposed BVPS-2 reracking are in accordance with the BVPS-2 licensing basis.

   Proposed Approach: The concrete properties (compressive strength, elastic modulus, etc.) used in all evaluations/calculations performed for the proposed BVPS-2 reracking are in accordance with the BVPS-2 licensing basis.

   The concrete compressive strength was used as defined in UFSAR Section 3.8.6.4.

   Concrete compressive strength; fc' = 3000 psi

   Other concrete properties used in accordance with the design basis include;

   Elastic modulus (from ACI 318-71, Section 8.3 Modulus of Elasticity)

   $E_c = 57{,}000\ \sqrt{fc'} = 57{,}000\ \sqrt{3000 psi} = 3{,}122{,}019\ lb/in^2$

   Poisson's ratio of concrete: $\nu = 0.17$

2. The August 9, 2010 supplemental response provided a table summarizing the results of 3000 psi run for rack drop.

a) For 3000 psi run, the strain in the liner plate is reported as 0.0088 and the impact force on the floor is reported as 159 kips. For the 4000 psi case, the strain in the liner plate was reported as 0.052 and the impact force on the floor was reported as 156.09 kips (624.36/4). The value of the liner plate strain for the 4000 psi case is 6 times larger than the strain value reported for the 3000 psi case. Considering the comparable values of the impact force on the floor for both the 3000 and 4000 psi cases, please provide further discussion to clarify why the strain values are so far apart.

   Proposed Approach: The reason that the plastic strain in the liner decreased by a factor of 6, despite the fact that the concrete compressive strength was decreased from 4,000 psi to 3,000 psi, is because the contact damping at the rack/SFP liner interface was also increased from 0% to 20%, in conjunction with the change in compressive strength. The original rack drop analysis, which utilized 4,000 psi concrete, conservatively assumed 0% contact damping. When the analysis was re-performed using 3,000 psi concrete, the contact damping was set to 20% (which is

the recommended value per the LS-DYNA user manual for impact problems) to obtain a more realistic solution. This change, however, was not explained to the NRC Staff in the previous RAI response, which was submitted on August 9, 2010. In fact, to the contrary the RAI response incorrectly stated: "Apart from the change in concrete strength, no other changes to the LS-DYNA simulation model have been made." It is also noted that the time step reduction factor was decreased from 0.9 to 0.8 in the revised LS-DYNA analysis to ensure a converged solution for the reduced strength concrete. This change, however, had only a minor effect on the results as compared to the change in contact damping. Both Holtec and FENOC have entered this occurrence into their respective corrective action programs.

To reconcile this issue, FENOC will re-perform the rack drop analysis using 3,000 psi concrete, 0% contact damping, and a time step reduction factor of 0.9, so that (as stated in the previous response) the only change to the LS-DYNA model is the concrete compressive strength.

b) Furthermore, the impact duration is reported as 0.62 seconds. Please confirm that 0.62 seconds is the correct value for the impact duration.

Proposed Approach: There was a typographical error in the previous response, which was submitted to the NRC on August 9, 2010. The impact duration should have been reported as 0.062 seconds, not 0.62 seconds. This will be corrected in the next RAI submittal.