Tennessee Valley Authority, 1101 Market Street, Chattanooga, Tennessee 37402

OCT 08 1992

TVA-SQN-TS-92-01 10 CFR 50.90

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

Gentlemen:

In the Matter of) Docket Nos. 50-327
Tennessee Valley Authority) 50-328

SEQUOYAH NUCLEAR PLANT (SQN) - RESPONSE TO QUESTIONS ON REQUEST FOR LICENSE AMENDMENT TO TECHNICAL SPECIFICATION (TS) - SPENT FUEL POOL STORAGE CAPACITY INCREASE

On March 27, 1992, TVA requested a license amendment to the SQN technical specifications to support increased spent fuel storage capacity.

On September 1, 1992, we received questions from NRC concerning the structural integrity analysis of the proposed spent fuel storage racks. The enclosed pages provide TVA's response to those questions.

Calculations referred to in this and previous submittals related to this amendment request were performed and issued by TVA's contractor, Holtec International. The appropriate TVA technical organizations have reviewed and concurred with the calculations. These calculations will be appropriately incorporated into the TVA calculation system prior to actual fuel rack installation.

Please direct questions concerning this issue to C. R. Davis at (615) 751-7509.

Sincerely,

D. Dick Shaley

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Enclosures 690012
cc: See page 2

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ENCLOSURE

1. Page 6-21

Provide a technical basis for the expression "Fa" for the compressive stress given in page 6-21 by means of derivation or by reference to an established code or both. Please note that the rack wall has a possibility of side sway laterally in a direction normal to the wall, i.e. the top support of the column may move laterally away from the original position with respect to the bottom support of the column when a compressive load is applied to a box type structure such as a rack. Demonstrate that the expression given in page 6-21 considers this possibility.

RESPONSE

The load due to the weight of the fuel assemblies bears directly on the baseplate of the fuel rack. Therefore, the only structural members subject to significant compression loadings are the support pedestals. The cellular portion of the rack experiences insignificant compressive loadings.

The term Fa includes the factor which accounts for the reduction in strength due to the slenderness effect of the structural member. Since the pedestals have a very low slenderness ratio, there is practically no reduction in the allowable compressive strength in contrast to the tensile strength.

The expression for Fa owes its origin to civil/structural engineering literature and first appeared in the structural engineering Code (Manual of Steel Construction, American Institute of Steel Construction, NY, NY). The ASME Code had this formula in Appendix XVII of Section III until the 1983 Edition and subsequently in Subsection NF (NF3322.2).

Because of the relatively small axial compressive stress in the rack cellular region, there is a large margin of safety against buckling in that region. This can be confirmed by perusing the maximum stress factor (above the baseplate) provided in Tables 6.7.3 through 6.7.20 for various loading scenarios.

2. Page 6-23

Stress factors are discussed in page 6-23. Provide the most highly stressed examples of R1 and R6. Identify what part of the rack these stresses correspond to and discuss the significance of the compressive stresses by providing the percentage of the compressive stress contribution to the R6.

RESPONSE

Stress factors R1 and R6 have limiting values for the 12x14 spent fuel rack. The limiting values come from Table 6.7.30 (for a case where adjacent racks are assumed to move out-of-phase).

R1	R6	Gross Cross-Section
.042	.301	Just above baseplate on a section through the entire cellular region.
.287	.484	Just above baseplate on a section through one pedestal.

For a case where the adjacent racks are assumed to move in-phase with the 12x14 rack (Table 6.7.21) the corresponding values are:

R1	R6
.039	.333
.282	.453

For the gross cross-section just above the baseplate (i.e., a cut-through the cellular region) the highest combined stress will be at a corner cell. Only .042/.301 = .1395 (14%) will be due to direct compression acting on the gross cross-section of 12x14 cells. In reality, the actual primary stress acting on the corner cell just above the baseplate will all be in compression since it is at "the extreme" fiber of the cross-section. The actual value of this stress on the outermost corner cell will be $R_6 \times (.6S_y)$ where $.6S_y$ approximates the allowable stress. Thus, the maximum compressive primary stress at the base of the outermost corner cell in the cell metal is $.333 \times .6S_y = 4995$ psi.

We note that on the gross cross-section of the cellular region of the rack (just above the baseplate) direct compression plays only a small role. Column buckling of the cellular structure as a beam is not a governing condition because there is only a small component of direct compression imposed during a seismic event (i.e., the heavy vertical fuel load is imposed directly on the baseplate and is not uniformly distributed along the cells).

For the pedestal, of course, the compressive load factors are a larger percentage (59-62%) of the total R_6. Buckling of the pedestal is not a concern since the section is extremely compact.

3. Page 6-24

The governing equation on page 6-24 does not have a damping term. Please explain when a structural damping is used. Discuss how the damping term is incorporated in the governing differential equation of motion. Also, justify the damping values used, referring to the Regulatory Guide 1.61.

RESPONSE

The matrix [Q] of the governing equation of motion includes the damping term. Structural damping follows established practice and is incorporated into the elastic portion of the model by introducing a structural damping matrix formed by associating linear structural damping coefficients of the form $c = \beta k$ with every linear spring in the model. Therefore, the Q matrix contains damping terms linearly proportional to velocity in addition to spring terms. β is a constant proportional to the specified damping percentage imposed on equipment subjected to the seismic event. As required by the Updated Final Safety Analysis Report (UFSAR), 2 percent structural damping is used for the design basis event. Four percent structural damping was used for the site specific event. The design basis event is the plant commitment in the UFSAR where maximum 2 percent damping curves are developed (page 3.7-29 of UFSAR). For the proposed rerack, TVA also imposed an additional spectra, corresponding to an SSE event, to be considered. Since this additional spectra is not part of the UFSAR, the damping value of 4 percent was obtained from Regulatory Guide 1.61 for welded structures. It turned out that even with higher damping, limiting rack behavior was controlled by the additional site specific seismic event.

4. Page 6-25

Provide a discussion regarding DYNARACK verification. The discussion should emphasize the nonlinear portion of the analysis together with some linear response aspects. Verification should include analytical calculation as well as experimental results, including full size tests.

RESPONSE

The validation manual for DYNARACK has been previously submitted on two dockets in the past year TMI Unit one and D. C. Cook). A brief outline of the validation is provided in the following.

The validation of DYNARACK is in conformance with the provisions of the Holtec Quality Procedure HQP 5.2, Computer Programs, and demonstrates that DYNARACK meets all validation requirements of USNRC-SRP 3.8.1. Section II.4(e) of SRP 3.8.1 states that computer programs used in design and analysis should be described and validated by any of the following procedures or criteria:

(i) The computer program is a recognized program in the public domain, and has had sufficient history of use to justify its applicability and validity without further demonstration.

The computer program solution to a series of test problems has been demonstrated to be substantially identical to those obtained by a similar and independently written and recognized program in the public domain. The test problems should be demonstrated to be similar to or within the range of applicability of the problems analyzed by the public domain computer program.

(iii) The computer program solution to a series of test problems has been demonstrated to be substantially identical to those obtained from cl[illegible] solution or from accepted experimental test [illegible] to analytical re[illegible] published in technical literat[illegible] The test problems should be demonstrated to be similar to or within the range of applicability of the classical problems [illegible] to justify [illegible] of the program. A summary comparison should be p[illegible] for the results obtained in the validation of each computer program.

Since DYNARACK is a private domain program, the validation problems used for DYNARACK comply with criteria (ii) and (iii) above.

In the DYNARACK Validation Report, it is shown that DYNARACK meets the following criteria:

1. All desired capabilities of the code perform as expected.

2. Results from DYNARACK are in excellent agreement with solutions obtained from other sources.

3. The fluid coupling methodology in DYNARACK is demonstrated to be in agreement with experimental results.

4. The code exhibits excellent convergence when applied to both linear and nonlinear problems.

The experimental verification of DYNARACK had to be performed on a scaled model since a full scale testing would involve very large inertia, fluid, and friction forces which would outstrip the capability of calibrated testing in any U.S. laboratory. To our knowledge, the only effort at full scale testing was in Japan, which, too, falls short of the objective because some key loadings such as the fluid coupling forces, were eliminated from the exp[illegible]ment, presumably to keep the testing effort manageable. Alth[illegible]gh attempts have been made to obtain it, the Japanese data has not [illegible] made [illegible] and it would be of limited value because of [illegible] ab[illegible]ntioned limitations.

Holtec's scaled model testing focused on the two key contributors to the dynamics of the racks--the fluid coupling and inertia forces. The results from almost 100 experiments demonstrated remarkable agreement between the predictions of the Code and the experimental data. Recognizing that empirical principles are used in constructing the DYNARACK equations of motion and that the Code has been benchmarked against a wide array of linear and nonlinear problems in dynamics, the experimental validations have further reinforced the veracity of DYNARACK. To our knowledge, DYNARACK is the only Code with such a complete underlay of validations. This Code has been used in over 1000 dynamic simulations in over two dozen nuclear plant dockets since 1980.

5. Page 6-28

Discuss the stress analysis of the various welds described in Page 6-28 and 6-29. Provide a definition of the limit force and the moment together with a numerical example of weld stress analysis of baseplate to rack and cell to cell. In particular, expand the term function (F/Fy, M/My).

RESPONSE

A copy of annotated back-up calculations for the welds is attached. These computations were performed using MathCad (commercial calculation program) and show how final results reported in the licensing document were achieved. The limit analysis interaction formula is

$$\frac{F}{F_y} + \frac{M}{M_y} \leq 1$$

where F, M are applied compressive force and net bending moment applied to a J-weld section, F_y = limit force = $W_y A_w$ and M_y = limit moment calculated on basis of ideal plasticity with W_y = yield stress and A_w = weld effective stress area. Use of a straight line interaction formula implied by the foregoing equation is conservative, as i neglect of the gussets in the calculation of plastic section moduli.

6. Page 8-5

Provide the total weight of the structure for the spent fuel pool (concrete, racks, fuel assemblies, water and other). What is the increase in various loads in going from the original design to the proposed high density design? Provide the amount of increase in stresses due to the new loads. In cases where the stresses are decreased in spite of the increase in the load, state the reasons for such an outcome (such as difference in analysis methods).

RESPONSE

The attached table shows that there is a 6.3 percent increase in total bearing weight caused by the proposed rerack. If we do not count the concrete in the table, then the percent increase becomes

$$\% \text{ Increase} = 100 \times \frac{7939.2 - 6264.9}{6264.9} = 25.2\%$$

The previous analysis of the pool structure used a mixture of analytical calculations on reduced models plus some finite element computations on selected portions of the structure. The new analysis used a total finite element based analysis. The criteria used for acceptance is comparison of moments and shears with American Concrete Institute allowable values. The limiting section of the structure, both in the current and proposed configuration, is the 18" intermediate wall between the cask pit and the main pool. It is difficult to quantify the actual increase in moment at critical sections because of a lack of 1 to 1 correspondence in the models. The new analysis includes amplification of the response due to low resonant frequency modes on the intermediate wall (the wall separating the cask pit from the spent fuel pool). The increase in moment is acceptable with the established requirement that the cask pit remain flooded.

7. Page 8-9

Provide a detailed discussion, in terms of numerical values, as to how the maximum stress of ∠2992 psi is obtained on the liner. Discuss the desig ı criterion that is based un an ultimate strength. The discussion should include the data basis for the ultimate strength and how the ultimate strength addresses bearing, tearing (fracture), denting or any other type of failure mode of the lir⁻r.

RESPONSE

The maximum liner stress of 22992 psi is obtained from a finite element analysis of a portion of the li: ɜr subject to imposed loads in the vertical and tangential direction. The purpose of this analysis is to assess whether reracking imposes the potential for liner damage due to the increased loads. The estimate of liner stress is obtained by considering the highest peak load from any pedestal in the pool during the governing seismic event to be applied uniformly over a load patch equal to the nominal size of a bearing pad. For conservatism, it is also assumed that friction forces are applied equa` to .8 x the peak normal load in each of two directions. (It is recalled that the bounding value of the interface coefficient of friction for stainless sheet in water is 0.8) The liner is simulated as a 1/4" thick plate in contact with an ela :ic foundation (the concrete). A representative section of the liner is considered and it is assumed that the load patch is applied near one corner of the liner section considered (roughly 5" away from a weld seam).

The corresponding elastostatic solution encompassing the three components of load is obtained and the maximum bending stress in the liner determined from the finite element analysis. The result for maximum elastic plate bending stress is 22992 psi. As expected, this maximum stress is near the edge of the seam weld.

The rimary inten⁺ of the ⁿnalysis is to calculate maximum stress leve. in the liner and at ⌐ welds to assess potential overstress and possible rupture of the liner. There is no criteria established for assessment o. liner stress level in the NRC OT Postion Paper; the margin with respect to the liner ultimate stress provides a measure of the safety against in-plane rupture.

In this case, since the stresses remain low, in the elastic range, rupture of the liner is ⁻ot possible.

8. Page 8-9

The concept of cumulative damage factor (CDF) is used in addressing the adequacy of the pool liner. Provide a basis for the use of CDF by reference, noting that the nature of seismic loading represents a low cycle fatigue with relatively high stresses.

RESPONSE

It is recognized that the vibratory motion of the rack due to the seismic event induces cyclic stresses in the pool liner. If the amplitude of the cyclic stress is above the endurance limit, then the most likely actuating mechanism for failure is low cycle fatigue. The governing design code for high density racks, Subsection NF of Section 3, Class 3 does not contain techniques for fatigue analysis. We refer to ASME, Section III, Subuection NB-3222.4 for the appropriate methodology.

The use of a fatigue criterion for liner assessment is another measure that is useful for considering implications of the rerack. Since fatigue analysis methods are not spelled out in the NF section of the Code, we refer to ASME, Section III, subsection NB-3222.4 for Class 1 components.

The procedure outlined in NB-3222.4 also refers to and requires use of Sections NB-3222.2, NB-32228.5, NB-3215, and NB-3216. Appropriate fatigue curves for obtaining cyclic life versus alternating stress range are given in Section III for austenitic steel.

Another reference where the concept of the Cumulative Damage Factor (CDF) is comprehensively explained is the text by David Burgreen, "Design Methods for Power Plant Structures," Arcturus Publishers (1975).

We use the time-history results of pedestal loads from the whole pool multi-rack analysis to determine the peak impact vertical load and make a conservative estimate of friction loads at the same instant. Per the requirements of the fatigue method, stress intensities are computed from the finite element analysis, and cycles are estimated from the time-history pedestal load files. In this case, since the stresses in the liner are low (see response number 7), the cumulative damage factor is less than 0.1 (allowable = 1.0).

ATTACHMENT FOR RESPONSE TO NRC QUESTION #5

WELD STRESSES BETWEEN CELL AND BASEPLATE

Let c be cell width, t be cell thickness, lw be weld length per side of cell, tw be weld size. Assume welding on 4 sides.

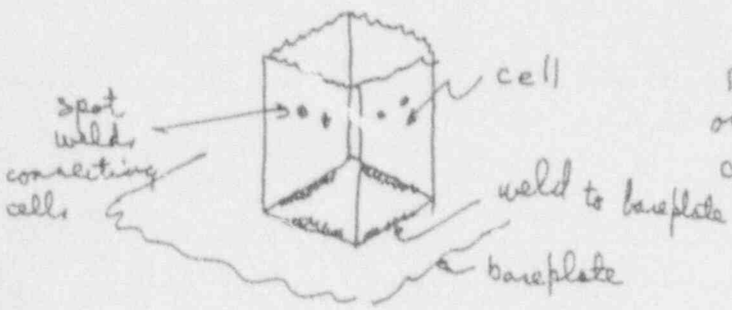
$c := 8.75 \qquad t := .060 \qquad l_w := 7 \qquad t_w := .060$

$A_{cell} := c \cdot t \cdot 4 \qquad A_{cell} = 2.1$ sq.in.

$A_{weld} := l_w \cdot t_w \cdot .7071 \cdot 4 \qquad A_{weld} = 1.188$ sq.in.

$R := \frac{A_{cell}}{A_{weld}} \qquad R = 1.768$

For TVA Sequoyah, the critical value for R6 just above the baseplate, is obtained for run do12x14a.rf8. From the summary tables 6.7.2 in the licensing report, we obtain:



R6 value allows one to calculate stress in cell wall. Stress in weld is obtained by ratioing cell area to baseplate weld area

$R_6 := .333$

$s_{bp} := R_6 \cdot 15000 \cdot R$ $\qquad s_{bp} = 8.83 \cdot 10^3$

This is less than the allowable weld shear stress.
For faulted conditions, we use .42 x ultimate stress as the shear limit per ASME NF which refers you to Appendix D for dealing with faulted conditions. Note that this weld stress is less than the weld stress allowable from the NF table for normal operation. (21000-24000 psi)

$s_u := 71000$ psi* so that

the weld allowable shear for SSE (faulted conditions) is

$t_a := .42 \cdot s_u$ $\qquad t_a = 2.982 \cdot 10^4$

psi

WELD BETWEEN SUPPORT FOOT PAD AND BASEPLATE

The weld between baseplate and support pedestal is checked by using limit analysis. This weld is a groove weld.* Additional weld area is provided by gussets applied at ninety degree locations.
The formula used for the limit analysis is (basic pedestal is circular):

* Actually, weld has been changed to J groove from a V groove.

F/Fy + (Mx**2+My**2)**1/2*1/MY <= 1 where

F,Mx,My are the calculated moments, FY,MY are the yield force and moment for the weld section. We now calculate the appropriate quantities.

The allowable weld limit stress is taken as $\qquad t_a := .42 \cdot s_u$

WE NEGLECT THE EFFECT OF THE GUSSETTS!!!!. We will include the gussets in this calculation only if we need them for the limit qualification

$c := 4.5$ in. $\qquad t_w := .625$ $\qquad t_a = 2.982 \cdot 10^4$ psi

$A := .7071 \cdot p \cdot t_w \cdot [2 \cdot c]$

$A = 12.495$ sq.in. of weld area per leg

$FY := A \cdot t_a$ $\qquad FY = 3.726 \cdot 10^5$

lbs.

$MY := \left[A \cdot \left[2 \cdot \frac{c}{p}\right]\right] \cdot t_a$ $\qquad MY = 1.067 \cdot 10^6$ in.lb.

We check the welds for critical cases. The case to check is run di12x14a.rf8 (table 6.7.2) which has the critical value of R6 for the upper support locations.

From table 6.7.30 of Licensing Report referenced above

$R_6 := .484$ the values for the individual load factors are

$R_1 := .287$ on the same foot

Then we estimate the total bending load factor as (Rb is bending effect on circular section.)

$R_b := R_6 - R_1$ $\qquad R_b = 0.197$

Therefore, the actual stresses (based on support area and inertia, not weld area, inertia) are

$s_b := 15000$ psi $=.6{*}Sa$

$s_0 := s_b \cdot R_1$ $\qquad s_0 = 4.305 \cdot 10^3$

$s_1 := s_b \cdot R_b$ $\qquad s_1 = 2.955 \cdot 10^3$

Knowing the support area AS and support inertia IS input into the analysis runs, we can back figure the actual direct load and bending moments as follows: (get data from sec 6 of this report dealing with input to predyna)

$AS := 45.859$ sq.in. $\qquad IS := 369.04$ in**4

$F1 := s_0 \cdot AS$ $\qquad F1 = 1.974 \cdot 10^5$

$M1 := \frac{IS}{c} \cdot s_1$

$M1 = 2.423 \cdot 10^5$

$I := \frac{F1}{FY} + \frac{M1}{MY}$

$I = 0.757 \quad < \quad 1$ O.K

Note that this neglects the added inertia and area of the gussets!!!

ANALYSIS OF SPOT WELDS

Ref. Holtec drawings 852,853, we can locate the spot welds. each weld is considered as having effective diameter .5 inch. There are two welds at any level. Therefore, the weld area available for shear transfer is

$p := 3.14159$ $\qquad d_w := .5$ $\qquad t_a = 2.982 \cdot 10^4$ psi

$a_w := 2 \cdot p \cdot \frac{d_w^{\,2}}{4}$ $\qquad a_w = 0.393$ sq. in.

The capacity of the welds (2 at any level) is: $P_c := a_w \cdot t_a$ $\qquad P_c = 1.171 \cdot 10^4$ lbs.

We compare the weld capacity at any level by the load that need be transferred by any impact. For a weld analysis, assume that adjacent boxes are not moving and that the impact load is being transferred from the box being impacted by the fuel to an adjacent box
Assume that each weld set transfers impacts at two locations (simultaneously) in the box

$P_a := 2 \cdot P$ $\qquad P_a = 599$ lbs.

Another shear check can be made at the bottom of the rack where we can take the shear loading at the worst location and see if the available weld spots can transfer the load. We make the worst case assuption that the adjacent boxes are fixed.. From Table 6.7.4 of the licensing report, the limit value of R2 is

$R2 := .041$

Therefore, the maximum elastic shear stress is $S_{max} := 1.5 \cdot R2 \cdot S_y$

$S_{max} = 1.538 \cdot 10^3$ psi $\qquad P_{max} := S_{max} \cdot a_w$ $\qquad P_{max} = 603.774$ lb.

Pmax is less than Pc for both cases considered. Also, note that at the bottom of the rack, there are two closely spaced set of spot welds so that the actual capacity is doubled.

ARCHIVE CALCULATIONS FOR TVA SEQUOYAH REGULAR FUEL

MCAD file \mcad\tvargrpt.mcd August. 28,1991

IMPACT LOAD BETWEEN FUEL ASSEMBLY AND CELL WALL

Design calculations are made using Section 1 of HI-89330.
NC= number of loaded cells, LOAD=total load. From Table 6.7.2 of Licensing report,HI-91670, the highest rack to fuel impact load is for run di13x14c.rf8. Also see, table 6.7.12

$NC := 182.$ $\qquad LOAD := 54509$

Therefore, the impact load per loaded cell is

$$P := \frac{LOAD}{NC} \qquad P = 299.5 \quad \text{pounds}$$

We use equ. 1.1 and 1.2 of Part III of the general seismic report to compute the cell capacity. We assume an impact over a length L. Data below comes from Holtec fuelrack data sheet attached to this calculation.

w= cell width, a=fuel width $\qquad w := 8.75 \qquad a := 8.426$

$s_y := 25000.$ $\qquad L := 10$

$t := .060 \qquad c := \frac{w-a}{2} \qquad c = 0.162$

$$Q_L := s_y \cdot \frac{L}{c} \cdot t^2 \cdot [.5] \qquad Q_L = 2.778 \cdot 10^3$$

per cell including a FURTHER safety factor of 2.0
The shear load limit is

$$Q_{Limit} := s_y \cdot \frac{t}{2} \cdot (a + L) \qquad Q_{Limit} = 1.382 \cdot 10^4$$

There will be no damage to the fuel assembly due to this load.
The fuel assembly manufacturer can attest that this load is less than that required to fail the assembly.

ATTACHMENT FOR RESPONSE TO NRC QUESTION NO. 6

WEIGHT OF SPENT FUEL POOL (in Kips)

	PROPOSED	CURRENT
CONCRETE	18694.4	18694.4
WATER	3918.6	3918.6
RACKS	330.8	198.0
SPENT FUEL	3589.8	2148.3
TOTAL	26533.6	24959.3

$$\% = \frac{26533.6 - 24959.3}{24959.3} \times 100$$

$= 6.3\%$