SMUD
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AN ELECTRIC SYSTEM SERVING THE HEART OF CALIFORNIA

MPC&D 00-036

February 24, 2000

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

Docket No. 72-11
Rancho Seco Independent Spent Fuel Storage Installation
Additional Information on the Rancho Seco ISFSI Safety Analysis Report (TAC No. L10017)

Attention: Randy Hall

In a letter dated July 21, 1999, the NRC submitted a Request for Additional Information (RAI) to support the review of the Rancho Seco Independent Spent Fuel Storage Installation (ISFSI) Safety Analysis Report (SAR). We responded to that request in SMUD Letter MPC&D 99-120, dated September 9, 1999.

Based on subsequent telephone calls between SMUD and NRC staff, on or about December 1, 1999, January 20, 2000, and February 1, 2000, we are providing additional information to support the NRC's review of the Rancho Seco ISFSI SAR. In addition to our response to NRC questions, we are including the following documents:

1. Duke Engineering Services Calculation 00079.02.0002.ST02 "SSI Effect on ISFSI Slab Acceleration," Revision 1.

2. SMUD Calculation Z-DRY-C1024 "ISFSI Concrete Slab Design," Revisions 3.

3. Geotechnical study for the Rancho Seco ISFSI, dated June 1, 1993.

If you, or members of your staff, have questions requiring additional information or clarification, please contact Bob Jones at (916) 732-4843.

Sincerely,

DE Jardin for S.R.

Steve Redeker
Manager, Plant Closure & Decommissioning

NMSS01 Public

1. Revise SAR section 3.4 to address design criteria for other Structures, Systems, and Components (SSCs) subject to NRC approval to include the basemat foundation. Describe how the soil characteristics are used to design the mat, including the method of analysis, variation of soil properties and the potential differential settlement, identified in section 6.3 of the "Geotechnical study" (Reference 2.8 in SAR Vol. 1). State the maximum soil bearing pressure and compare to the allowable soil bearing pressure.

   In accordance with the requirements of 10CFR72.102(d), site-specific investigations and analyses must show that soil conditions are adequate for the foundation loading.

   Response:

   SMUD analyzed the HSM and apron slabs in accordance with the Uniform Building Code (1991).

   The soil characteristics such as allowable bearing pressure and vertical subgrade modulus are used by the calculation in various ways. The bearing pressure is used as a maximum value of pressure the soil is allowed to take from the structure due to vertical loads or overturning loads. A calculated bearing pressure comes from a finite element model analysis and is compared to the allowable bearing pressure. The allowable bearing pressure is given for load applications that are not wind or seismic related; however, a one third stress increase is allowed for those particular cases per the soils report. The vertical modulus of subgrade reaction is used to establish a spring constant representing the soil in the finite element model.

   The slab analysis was performed using two finite element models; one for the HSM slab and one for the apron slab. The HSM slab supports only the HSMs and is not subject to transporter loads or crane loads. The design of the HSM slab considered the dead load and seismic loads associated with all HSMs being in place as well as the potential case where only a few of the modules were in place. The intent of these loading cases being to identify the maximum moments reasonably possible in the slab. The design of the apron slab took into account the movements of the transporter load over various parts of the apron slab, the cask load, and the crane load associated with movement of the cask.

   Soil properties were modeled as springs and variations of properties were not considered other than those embedded in the allowable bearing pressure and vertical modulus of subgrade reaction. This is consistent with relatively simple designs using the Uniform Building Code as its design basis.

   The soils report indicated that settlement could be expected. The total settlement was given at 1.5" and differential settlement was given as ½ to $^1/_3$ of the total settlement. The differential settlement, therefore, would amount to ¾" to 1" over the length or width of the slabs. The smallest dimension of a slab is the HSM slab with a width of 38 feet. A maximum differential settlement of 1" over this width is acceptable and will not adversely affect the design.

The calculated bearing pressures are compared to the allowable bearing pressures in various parts of the calculation. The following presents an overview of the bearing pressure comparisons:

Load Case	Calculated Bearing Pressure	Allowable Bearing Pressure
HSM:		
DL+LL	2.8 ksf	4.0 ksf
D+E (long.)	3.11 ksf	5.3 ksf
D+E (trans)	3.94 ksf	5.3 ksf

Bearing pressures for the apron slab were determined to be less than the HSM slab and so did not control bearing pressure evaluations.

2. The geotechnical report states that a vertical subgrade modulus of 300 psf can be used for the design of the basemat. It appears that the units of the recommended modulus should be "pounds per cubic inches", and not "psf". What is the actual magnitude of the soil modulus used in to verify the structural adequacy of the basemat design.

Response:

We discovered an error in the soils report regarding its quantification of the vertical modulus of subgrade reaction. The geotechnical company was contacted about this and indicated the corrected value should be 300 kips/ft^2/ft. This value corresponds to 174 lbs./in^2/in, which is reasonable for the soil type encountered at this site. The 300 kips/ft^2/ft is the value used in the analysis of the slabs.

3. The potential for liquefaction of soil is addressed in section 3.4 (page 6). However, the statement is qualitative without any substantive basis and/or engineering rationale. Please provide more explicit discussion of how this report (or other previous site analyses or information) forms the basis for meeting 10 CFR 72.102 (c) and (d). Also discuss the potential seismic amplification due to soil-structure interaction.

Response:

Liquefaction

The response provided by the geotechnical firm is consistent with the level of detail that is usually provided by these firms for simple non-nuclear applications, especially if a site is not prone to liquefaction. Liquefaction occurs most often where groundwater is within 30 feet of the surface, but it may occur in areas where ground water is up to 50 feet beneath the surface. High pore pressures that build up in sediments during repeated seismic vibrations cause the soil to behave as a liquid. The excess pore pressures are often pushed upward through fissures and soil cracks causing a water-slurry to bubble onto the ground surface. The resulting features are called sand boils, sand blows, or "sand volcanoes." The reduction in soil volume due to densification or extrusion causes settlement, which may result in failure of structural foundations.

For liquefaction to occur, three primary conditions must occur:

1. A moderate to strong earthquake that generates strong ground shaking;
2. Shallow groundwater, within 50 feet of the ground surface;
3. Laterally extensive layers of loose, fine to medium-grained sandy soils within the saturated zone

For the Rancho Seco site, a moderate to strong earthquake that generates strong ground shaking is possible; however, the other two attributes do not exist.

The geotechnical study estimated the groundwater table to be approximately 150 feet below the surface well. This exceeds the 50 feet usually attributed to liquefaction potential.

The soil at the site varies at different levels with sand, silt, clay, gravel, and sand all present. The soils report indicates that for the clay and silt layers the consistencies are typically hard while for the sand and gravel they are usually very dense with one location being medium dense. These soils do not meet the requirement of a loose (unconsolidated) soil for liquefaction to occur.

Based on the above it is highly unlikely that liquefaction could occur at the Rancho Seco ISFSI site. This conclusion is consistent with the Updated Safety Analysis Report (USAR) which does not identify liquefaction as a hazard at this site.

Soil Amplification due to Soil-Structure Interaction

Duke Engineering Services Calculation 00079.02.0002.ST02 "SSI Effect on ISFSI Slab Acceleration," Revision 1 discusses the issue of soil amplification due to soil-structure interaction (SSI) at the ISFSI. As shown in the calculation, the amplification of the acceleration response of the ISFSI slab and HSMs due to SSI is negligibly small when compared to the existing design margins of the slab and HSMs.

4. Verify the magnitudes in Table 8-15 of Volume II of the Rancho Seco SAR for the following comments and explain the reasons for the wide variations (or the identical values).

   a. Results (moments and shears) for cases 3 and 4 are identical. The case 3 load combination includes effects of 1.275W in addition to the loads included in case 4.

   b. Moments for cases 3 through 7 are significantly higher than the results for the same cases for the Standardized NUHOMS®-24P SAR Table 8.2-18 (e.g., for cases 3 and 4, the standardized NUHOMS® SAR shows the moments as 154.1 in-kips/ft, while the Rancho Seco SAR show the magnitude the magnitude of 800.2 in-kips/ft). Loads for the Rancho Seco HSM and NUHOMS® HSM are similar.

   c. The moment capacity of 910 in-kips/ft for cases 1 through 6 is significantly higher than the moment capacity of 229 in-kips/ft shown in Table 8.2-18 of the NUHOMS®-24P SAR.

Response to Question 4a

The calculated moments and shears from load combination 3 provide bounding results for load cases 3 and 4. Therefore, to reduce the volume of calculations, the calculated moments and shears for load combination 3 are reported in Table 8-15 for both load combinations. This results in the moments and shears for cases 3 and 4 being identical. This method was used for both the standardized NUHOMS®-24P design and SMUD calculation.

Response to Question 4b

The results reported in both the standardized NUHOMS®-24P SAR and the Rancho Seco SAR are based on the HSM element with the lowest margins (highest ratio of calculated value to capacity). For the standardized NUHOMS®-24P design the maximum moment results reported in NUHOMS®-24P SAR Table 8.2-18 for load combinations 3 and 4 occur in the floor. The maximum computed moment in the floor elements is 154.1 in-kip/ft and maximum moment capacity of the floor is 229 in-kip/ft. The maximum ratio of computed moment to moment capacity is 0.673.

The maximum moment results reported in the Rancho Seco SAR for load combinations 3 and 4 occur in the front wall. The maximum computed moment is 800.2 in-kip/ft and maximum moment capacity of the front wall is 910 in-kip/ft. The maximum ratio of computed moment to moment capacity is 0.88.

This change is due to the increase in the normal handling load for the Rancho Seco design from the 20 kips used in the standardized NUHOMS®-24P SAR to 60 kips. For the Rancho Seco HSM concrete design reported in the SAR the affect of the 60 kip normal handling load was conservatively ratioed up from the 20 kip standardized NUHOMS®-24P results. This results in a conservative computed moment in the front wall for the Rancho Seco design which is significantly higher than the computed moment in the front wall of the standardized NUHOMS®-24P design. Therefore, the maximum moment results reported in Rancho Seco SAR for load combination 3 and 4 occur in the front wall, which has a moment capacity of 910 kip-in/ft. The results reported in Revision 4A of the standardized NUHOMS®-24P SAR occur in the floor, which has a moment capacity of 229 kip-in/ft. The standardized NUHOMS®-24P has been evaluated for this change in load and the results will be reported in the next update of the standardized NUHOMS®-24P SAR.

Response to Question 4c

As discussed in response to Question 4b the Rancho Seco SAR maximum stress ratio's for load combinations 1 through 4 and 6 occur in the front wall, which has a moment capacity of 910 kip-in/ft. For the standardized NUHOMS®-24P SAR the maximum stress ratios for load combinations 3 through 6 occur in the floor which has a moment capacity of 229 kip-in/ft. The difference in stress ratios is the result of increasing the normal handling load for DSC transfer from 20 to 60 kips. A similar change has been evaluated for the standardized NUHOMS®-24P and will be shown in the next updated SAR.

Response to NRC Question Regarding HSM Floor Temperature

For the Standardized NUHOMS system, conservative estimates of the floor and roof temperatures were made using initial HSM concrete conditions with a DSC heat load of 24 kW and 125°F ambient temperature for steady state conditions at the start of the blocked vent transient. The methodology similar to the 24 kW case was used to estimate the temperatures at the end of 40 hours blocked vent transient at the Rancho Seco ISFSI with 13.5 kW per DSC. With 13.5 kW decay heat, at the end of 40 hours of blocked vent transient, the floor and roof temperatures are estimated to be 300°F and 289°F respectively. Note that with 13.5 kW decay heat and 117°F maximum off-normal ambient temperature, the floor and roof are below the 350°F limit at 40 hours into the blocked vent transient.

We will revise ISFSI SAR, Volume II, Table 8-4 as follows:

Table 8-4

HSM Thermal Analysis Results Summary

HSM Vent Inlet Air Temperature (°F)	Maximum DSC Outer Surface Temperature (°F)			Maximum HSM Concrete Temperature (°F)			
				Roof			
	Bottom	Side	Top	Inside	Outside	Side Wall	Floor
70	224	279	323	164	114	137	139
117	274	334	382	241	186	203	199
N/A(1) (All vents plugged with outside air at 117°F)	438	556	614	<350(2)	N/A	323	<350(2)

Notes:

1. The maximum concrete temperatures are based on 24 kW decay heat per DSC, 125°F ambient, and 40 hours of blocked vent condition.

2. The maximum roof and floor temperatures with 13.5 kW are expected to be 289°F and 300°F respectively at the end of 40 hours of blocked vent transient.