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October 28, 1976

Mr. Dennis L. Ziemann, Chief
Operating Reactors - Branch 2
Division of Operating Reactors
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: Dresden Station Units 2 and 3
Quad-Cities Station Units 1 and 2
Mark I Containment Plant Unique Analyses
NRC Docket Nos. 50-237/249 and 50-254/265

Dear Mr. Ziemann:

On October 6, 1976, Mr. C. Hofmayer of your staff requested answers to questions concerning Dresden Units 2 and 3 and Quad-Cities Units 1 and 2 Mark I containment plant unique analyses. Some items were resolved at that time; however, the remaining questions were reviewed more carefully and a complete response made by Nutech to Mr. Hofmayer in a telephone conversation of October 14, 1976.

It was agreed during the October 14 telephone conversation that a written response was appropriate for these questions.

Of the three questions requiring written responses, one question applies to the Dresden analysis only, one applies to the Quad-Cities analysis only, and one question applies to both Dresden and Quad-Cities. A restatement of the questions requiring documentation and the required written responses are provided in the attachments to this letter.

One (1) signed original and 59 copies of this letter are submitted for your use.

Very truly yours,

G. A. Abrell
Nuclear Licensing Administrator
Boiling Water Reactors

Enclosure (1): Response to NRC Questions on Mark I Short Term Program Plant Unique Analysis.

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RESPONSE TO NRC QUESTIONS ON MARK I SHORT TERM PROGRAM PLANT UNIQUE ANALYSIS - DRESDEN UNITS 2 AND 3

(NRC DOCKET NUMBERS 50-237 AND 50-249)

REFERENCES:

1) Report titled, "Dresden Nuclear Generating Plant Units 2 and 3 Short Term Program Plant Unique Torus Support and Attached Piping Analysis", Revision 1, August 1976, NUTECH Report Number COM-01-040.

2) Addendum No. 1 to Reference 1 above, dated August 1976.

3) Supplement to Reference 1 above, dated September 1976, NUTECH Report Number COM-01-051.

QUESTION NO. 1

In the calculation of the ultimate capacity of the pin connection at the base of the torus support columns, the capacity of the pin in bending is based on an elastic analysis with a 2 Sy limit on the extreme fiber stress. Since twice the yield stress of the pin material is greater than the ultimate tensile stress (Su), the method of calculation of the ultimate bending capacity of the pin should be re-evaluated.

RESPONSE:

The ultimate capacity of the pin in bending has been evaluated by an alternate method which more accurately represents the pin bending failure mode. The plastic bending moment capacity of the pin has been used in this alternate method in lieu of the original elastic analysis. For the plastic moment capacity calculation the shape factor of 1.70 for a solid circular cross section has been used. No credit has been taken for increased yield strength due to dynamic loading.

The appropriate changes to the numerical values for the pin connection capacities and the strength ratios given in Reference 3 are provided below. Reference 3 reports "worse case" loads and strength ratios for the structural components of the torus support system, pin connections included, for various submergences within Dresden's operating water level band. Therefore, loads and strength ratios reported in References 1 and 2 are superseded by the values reported in Reference 3.

It can be seen from the information presented below that the Short Term Program acceptance criteria is satisfied for all loading conditions for the pin connections at the base of the torus support columns.

- Reference 3, Table 3, page 13; Column 4 of that Table - Revise inside pin connection ultimate capacity to "3015(1) kips"; revise the outside pin connection ultimate capacity to 1973 kips". Column 5 of the table - revise the inside pin connection ultimate capacity strength ratio to "0.26"; revise the outside column ultimate capacity strength ratio to "0.50".

- Reference 3, Table 5, page 15; Column 4 of that Table - Revise inside pin connection ultimate capacity to "3015(1) kips"; revise outside pin connection ultimate capacity to "1973 kips". Column 5 of the table - revise strength ratio for inside pin connection to "0.23"; revise strength ratio for outside pin connection to "0.35".

- Reference 3, Table 10, page 20; Column 4 of that Table - Revise inside pin connection ultimate capacity to "3015(1) kips"; revise outside pin connection capacity to "1973 kips". Column 5 of the table - revise inside pin connection ultimate capacity strength ratio to "0.26"; revise outside pin connection ultimate capacity strength ratio to "0.50".

(1) The value currently reported in Table 3 is 2513 kips and is based on the plastic moment capacity of the pin and a yield strength of the pin material of 40 ksi. The revised value of 3015 kips is also based on plastic moment capacity, however, a yield strength of 48 ksi is used in accordance with Reference 2 (page III-5).

QUESTION NO. 2

Reference 1 - Tables 6.1.1-7 (page 6.17) and 6.1.2-2 (page 6.26) and write-up on page 3.35: The notation "P_L" and "Q" used in the tables does not appear to be consistent with the write-up on page 3.35. Also, the use of "Q" (secondary; membrane plus bending) is questioned in the table column headings for ring stresses.

RESPONSE:

The write-up on page 3.35 should be revised as follows: in the third and fourth lines, delete the words "plus primary bending".

Tables 6.1.1-7 and 6.1.2-2 are to be replaced with the revised tables given below.

Replacement for Table 6.1.1-7

	MAXIMUM COMPUTED STRESS INTENSITY			CODE ALLOWABLE STRESS INTENSITY			STRENGTH RATIO
SHELL	P_L	P_M	$P_L + Q$	P_L	P_M	$P_L + Q$	Not applicable since code allowables are met
	16.0	16.0*	23.6	28.9	19.3	57.8	

	MAXIMUM COMPUTED STRESS INTENSITY	CODE ALLOWABLE (Code allowable for general membrane (1.0 Sm) assumed to apply for all ring stresses)	STRENGTH RATIO
RING	11.5	19.3	Not applicable since code allowables are met

* Conservatively assumed to be same as maximum computed value of P_L.

Replacement for Table 6.1.2-2

	MAXIMUM COMPUTED STRESS INTENSITY			CODE ALLOWABLE STRESS INTENSITY			STRENGTH RATIO
SHELL	P_L	P_M	$P_L + Q$	P_L	P_M	$P_L + Q$	Not applicable since code allowables are met
	7.03	7.03*	11.0	28.9	19.3	57.8	

	MAXIMUM COMPUTED STRESS INTENSITY	CODE ALLOWABLE (Code allowable for general membrane (1.0 Sm) assumed to apply for all ring stresses)	STRENGTH RATIO
RING	5.05	19.3	Not applicable since code allowables are met

* Conservatively assumed to be same as maximum computed value of P_L.

RESPONSE TO NRC QUESTIONS ON MARK I SHORT TERM PROGRAM PLANT UNIQUE ANALYSIS - QUAD CITIES UNITS 1 AND 2

(NRC DOCKET NUMBERS 50-254 AND 50-265)

REFERENCES:

1) Report titled, "Quad Cities Station Units 1 and 2 Short Term Program Plant Unique Torus Support and Attached Piping Analysis", Revision 0, August 1976, NUTECH Report Number COM-01-040.

2) Supplement to Reference 1 above, dated September 1976, NUTECH Report Number COM-01-050.

QUESTION NO. 1

The strut between the base of the torus support columns and the lower portion of the torus shell has not been addressed in structural component evaluation reported in Reference 1 and 2 above. These components should be evaluated as part of the torus support structure.

RESPONSE:

Figure 1 below provides a general description of the subject strut and its connection to the torus support column and suppression chamber shell.

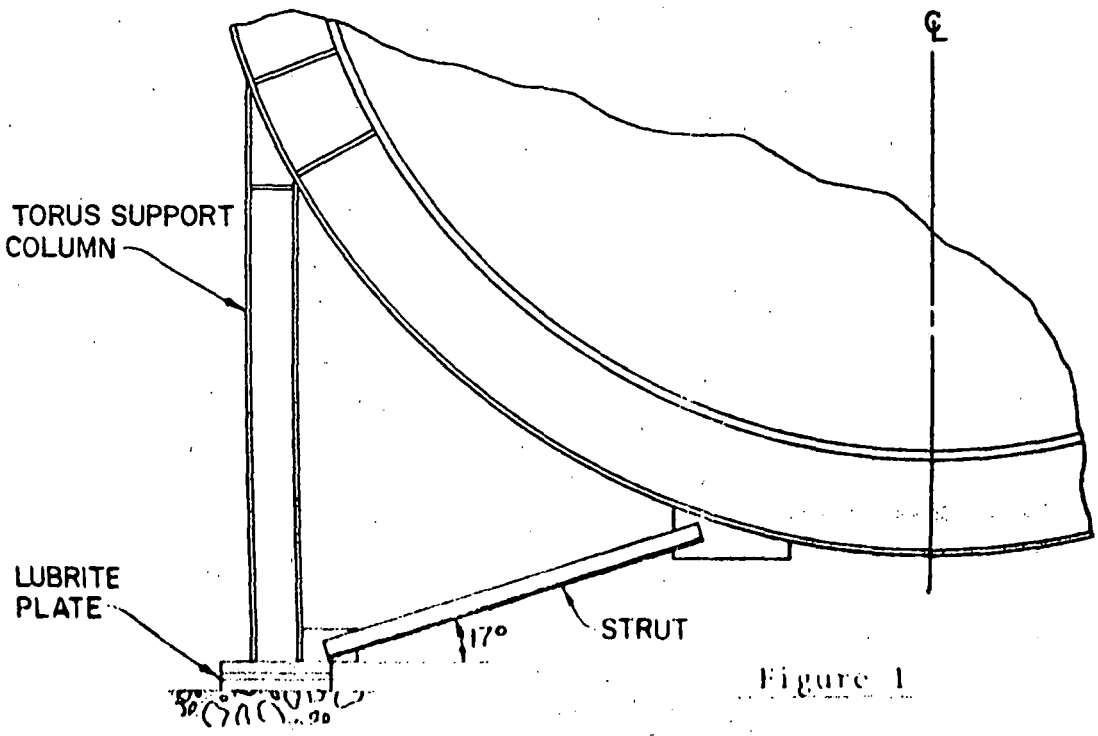

Figure 1

The 3-D model of the torus shell and its support system did not model the struts. However enough information is available from the 3-D model results to allow a separate evaluation of the struts. Such an evaluation has been conducted and the results are reported below.

For the evaluation of the strut and the effect that the strut has on the torus support column, the following three questions have been addressed.

1) Assuming the strut did not exist, are the horizontal reaction forces at the base of the torus support column, during the pool swell loading transient, of such a magnitude that they exceed the horizontal friction force resistance at the base of the column?

2) Are the axial tension and compression forces that are developed in the strut and its connections, during the pool swell loading transient, of such a magnitude that Code allowables or STP criteria are exceeded? This is an important consideration since the primary purpose of the struts is to provide lateral restraint for the column base during the post-LOCA long term thermal heat-up of the torus. For this design condition, the base of the columns slide on the lubrite plates in a radially outward direction (on the order of 1 inch). Since the struts have an important function for this design condition, it is necessary to establish that a buckling or tension failure of the strut does not occur during the pool swell loading.

3) Are the forces in the strut during pool swell loading of such a magnitude that the strut will impose displacement of the base of the column? If such displacements do occur, what is the resulting bending moment in the column and its effect on the axial load carrying capacity of the column?

Relative to Item 1 above, it can be concluded that during the pool swell load transient the lateral restraint of the base of the torus support columns can be provided entirely by the horizontal friction between the column base and bearing plate anchored in the concrete. This conclusion is reached by the fact that the horizontal reaction forces at the base of the columns from the 3-D analysis (column base was "pinned" in the 3-D model) are on the order of 2 kips, whereas the horizontal friction resistance is equal to or greater than 16 kips. The 16 kip value is calculated by multiplying the axial load in the column by a conservatively low value for the lubrite plate

coefficient of friction of 0.03. The recommended design value for the lubrite plate coefficient of friction is 0.10. Also a conservatively low value of 521 kips for the column axial load has been used in this calculation. This is the axial load in the inside column for the minimum submergence case with both horizontal and vertical seismic forces assumed to act in a direction so as to reduce the net column load. The conclusion that can be drawn from the above is that if the strut did not exist, the base of the torus support column would be stable from a lateral displacement consideration.

For the evaluation of Item 2 above, it can be observed from Figure 1 that the maximum axial force that can exist in the strut is that which would cause a lateral displacement of the base of the torus support column. To determine an upper bound value for this axial load in the strut a conservatively high value for the coefficient of friction and the column load has been used. Using a value of 0.10 for the coefficient of friction and 845 kips for the column load (outside column, maximum submergence, horizontal and vertical seismic acting in compressive direction), the horizontal friction resistance is 85 kips. The upper bound value for strut axial load is therefore 85/cos 17° = 89 kips.

The "code allowable" axial compression and tension force and the "ultimate capacity" axial compression and tension force for the strut have been calculated for comparison with the 89 kips computed above. The code allowable axial compressive load is 110 kips and is controlled by strut buckling considerations. The code allowable axial tension load in the strut is 111 kips and is controlled by the connection of the strut to the base of the torus support column. The ultimate capacity value for the tension and compression force in the strut is 251 kips and is controlled by the connection of the strut to the column base. Based on the above it can be concluded the axial forces in the strut will not exceed code allowables during the pool swell transient.

To answer the question posed in Item 3 above, the horizontal and vertical displacements at the point that the strut attaches to the shell have been extracted from the 3-D analysis results. It has been determined that these displacements are such that the strut will impose a lateral displacement at the base of the column. The maximum displacement of the base of the column occurs for the inside torus support column. This horizontal displacement of 0.067 inches results from a downward displacement of the strut-to-shell attachment point of 0.114 inches with a corresponding horizontal displacement (radially inward toward the centerline of the drywell) of 0.032 inches. The 0.067 inch lateral displacement results in a maximum bending moment in the torus support column of 420 in-kips. As reported

in Reference 1 the code allowable axial load and ultimate capacity of the torus support column is controlled by evaluation of the interaction equations for "primary" stresses. By using the diagrams on pages 3.14 through 3.21 of Reference 1, it has been determined that the additional 420 in-kips of secondary moment in the column does not result in secondary stresses controlling the code allowable or ultimate capacity of the column. However the 0.067 inch displacement at the base of the column does introduce some primary bending moments in the column due to increased column curvature. Using the column interaction diagrams of Reference 1 it has been determined that the reduction in the column axial load carrying capacity (both Code and ultimate) is less than 1% due to this increase in primary moment in the column. Therefore, the 0.067 inch lateral displacement will result in an insignificant change in the strength ratios reported in References 1 and 2.

In summary; the strut has been evaluated for the effects of pool swell loads on the torus shell and it has been determined that forces in the strut and its connections will remain within code allowables. The effects of strut forces on the torus support columns are insignificant. Therefore the strength ratios and conclusions reported in References 1 and 2 remain unchanged.

QUESTION NO. 2

Reference 1 - Table 6.1.1-7 (page 6.17) and write-up on page 3.24: The notation "P_L" and "Q" used in the table does not appear to be consistent with the write-up on page 3.24. Also, the use of "Q" (secondary; membrane plus bending) is questioned in the table column headings for ring stresses.

RESPONSE:

The write-up on page 3.24 should be revised as follows: in the third and fourth lines, delete the words "plus primary bending".

Table 6.1.1-7 is to be replaced with the revised tables given below.

	MAXIMUM COMPUTED STRESS INTENSITY			CODE ALLOWABLE STRESS INTENSITY			STRENGTH RATIO
SHELL	P_L	P_M	$P_L + Q$	P_L	P_M	$P_L + Q$	
	19.9	19.9*	25.1	28.9	19.3	57.8	0.26

	MAXIMUM COMPUTED STRESS INTENSITY	CODE ALLOWABLE (Code allowable for general membrane (1.0 Sm) assumed to apply for all ring stresses)	STRENGTH RATIO
RING	13.9	19.3	Not applicable since code allowables are met

* Conservatively assumed to be same as maximum computed value of P_L.