

SUPPLEMENT 2

RESPONSES TO
STRUCTURAL ENGINEERING BRANCH
COMMENTS AND REQUEST
FOR

ADDITIONAL INFORMATION ON CRYSTAL RIVER UNIT NO. 3
REACTOR BUILDING DOME DELAMINATION
INTERIM REPORT AND SUPPLEMENT NO. 1

1. GENERAL COMMENTS.

In the report the applicant discussed all possible factors which could have caused the delamination of the dome. No single or overriding mechanism has been positively identified as the cause of the delamination. However, the following facts are significant.

1. The indication of a tension failure along the delaminated surface.
2. The complete fracture of the coarse aggregate on the delaminated surface.
3. Large variations in the strength values obtained from the direct tensile tests of the concrete.
4. The presence of cracks of various sizes and extents in the concrete below the delamination as indicated by core borings.

On the basis of these facts, the sequence of events that led to delamination could be surmised:

From the evidence indicated above, one could conclude that; (1) the characteristics of the dome concrete are such that it is crack-prone, and localized cracks may have existed even before the prestressing force was applied, and (2) the coarse aggregates are fragile, thus, instead of acting as crack arresters, they became the path of cracks.

With the existence of precracks and the presence of fragile coarse aggregates, the radial tension accumulated from all sources was so large that it overcame the very limited tensile strength of the concrete, resulting in the separation of the dome concrete.

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It has been found by various investigators that cracking of concrete under compression is slight for loads below 30 to 50 percent of the ultimate. This is basically the reason why the allowable concrete compressive stress is limited to 45% of the ultimate. The cracks, if any, which initially may have developed in the dome concrete as a result of prestressing are unstable. They increase in length and width until either they eventually stabilize or ultimate failure occurs. The slow crack growth in concrete under sustained loading is most likely associated with creep.

The postulation of the delamination mechanism and the understanding of concrete crack initiation and propagation are essential for the establishment of the dome repair procedure and its evaluation. The following repair procedure is being pursued by the applicant:

1. Holes will be core-drilled into the lower concrete;
2. Top delaminated concrete will be removed;
3. Final inspection of 24" structure will be performed;
4. Lower level cracks will be grouted with epoxy;
5. Radial anchors will be set and the holes grouted;
6. New reinforcement and concrete will be added;
7. 18 tendons will be retensioned;
8. Structural Integrity Test will be performed.

The 18 tendons will be partially retensioned as described in Section 5.2.9, Page 5-6, September 22, 1976 revision to the report, "Reactor Building Dome Delamination."

On the basis of the postulation of the delamination mechanisms and understanding of concrete crack initiation and propagation as discussed above, the staff has reviewed and evaluated the repair procedure. However, before the staff can finalize its evaluation, the applicant should respond to the staff's concerns as indicated below:

Answer: The variation of direct tensile test results is discussed in Appendix C of the report, "Reactor Building Dome Delamination." The Table on Page C-15 (Attachment D) presents direct tensile strength test results and describes in the remarks column the relative "hardness" of the coarse aggregate. A review of that table indicates tensile strength is related to "hardness" of coarse aggregate. Also, see Page C-6 for a discussion of direct tensile tests by Mr. Joseph F. Artuso.

With regard to the tensile load capability of the concrete, two types of tests were performed to measure the tensile capability of the "in-place" concrete; i.e., split tensile and direct tensile tests. Attachment B of Appendix C of the Dome Delamination Report indicated that the average value for split tensile test of the "in-place" concrete was 710 psi, with a minimum of 625 psi. Attachment C of Appendix C in the Report indicates that the average value for direct tensile tests of the "in-place" concrete was 420 psi, with two test values lower than the average; i.e., 360 and 230 psi.

As indicated on Table 2-2 of the Dome Delamination Report, in the original design criteria the allowable membrane tension stress for "factored" loads was 212 psi and even lower for service loads. As indicated above, the lowest individual value for tensile strength obtained from either the split tensile or direct tensile tests was greater than the original design values for membrane tension for even the factored load condition. Therefore, it is logical to conclude that "poor quality of the aggregate" was not the cause of the delamination particularly since the structure has only seen service loads to date. The pattern of cracking as determined from inspection of a large number of bore holes was oriented in planes parallel to the dome surface. If due to generally poor properties of concrete, a more random orientation would have been expected. There is no indication that cracking existed prior to dome prestressing.

A review of the dome delamination problem indicates that local tensile stresses at the tendon conduits existed at each layer of tendons in the original 36-inch dome after tensioning of the dome tendons. This review indicates local tensile stresses in the vicinity of the conduits were in excess of even the split tensile strength of the concrete stated above, and thus large enough to cause local cracking of the concrete. These local tensile stresses were oriented in a direction nearly perpendicular to the dome surface. It is concluded that cracking initiated by these high local stresses, in the absence of radial ties, propagated into the delaminations observed at Crystal River Unit 3. Radial ties have been incorporated into the dome as part of the corrective action (see Section 5.2.7).

II. DOME REPAIR.

1. An analysis of the repaired dome should be made for the following conditions:

- (a) Before the hardening of the cap concrete.

Answer: Analysis of the repaired dome before the hardening of the cap concrete has been performed. The controlling stresses and deformations are reported in Appendix G. "COMPARISON OF DESIGNS," Pages G-7 through G-9, September 22, 1976 revision to Dome Delamination report, refer to column headed "Dead Load Plus Prestress at Early Plant Life."

- (b) After the hardening of the cap concrete, including all the loading conditions as described in the FSAR.

Answer: Controlling analytical results for the repaired structure with the new cap in place are summarized in Appendix C, "COMPARISON OF DESIGNS," Pages G-7 through G-9. Other FSAR load combinations have not been presented since they do not control any of the final dome design.

Indicate the stresses and strains in the mainly reinforced concrete cap portion and in the prestressed concrete lower portion.

Answer: Appendix G, "COMPARISON OF DESIGNS," includes the requested information.

2. Provide a description of the final design of the radial anchors and indicate how the combined action of the cap concrete and the lower dome concrete is ensured.

Answer: The final design of the radial reinforcement and the combined action of the cap with lower dome concrete are presented in Section 5.2.7 (Page 5-5, September 22, 1976 Revision). Specific reference is also made to figures 5-22 and 5-23, as well as Appendix I.

3. It was indicated that two layers of reinforcing steel will be provided in the cap. For the meridional reinforcing steel, if only one layer can be spliced to the existing meridional steel near the ring girder, indicate how the other layer can effectively carry the load if it is not spliced to the existing steel, noting that under internal pressure, dome concrete may crack in tension.

Answer: The #8 lower layer meridional reinforcement is provided for crack control only. Figure 5-20 illustrates meridional steel provided versus that required and does not include consideration of the #8 lower layer meridional steel shown in Figure 5-19. The lower layer of the meridional steel therefore is not assumed to "...effectively carry the load...". The top layer of meridional and both layers of hoop reinforcement in the new cap are considered to provide strength.

4. Since the repaired dome becomes a unique structural element of the containment structure, indicate any special considerations to meet the requirements of Regulatory Guide 1.18 in executing the structural integrity test of the containment.

Answer: Regulatory Guide 1.18 requires that displacement be measured at the apex and spring line of a containment dome. The instrumentation for the Crystal River Unit 3 Reactor Building has been considerably enhanced with regard to the dome. Refer to Section 5.7.1.c (page 5-3 of the September 22, 1976 revision) for detail on the dome instrumentation for the SIT. The additional measurements of dome displacement will be included in the SIT acceptance requirements. The predicted response data was supplied by letter of October 8, 1976 (Attachment 1).

5. The original dome design concrete strength, f'_c is based on 5000 psi; now a concrete strength of 6000 psi is used for evaluating the repaired dome. The basis for using 6000 psi is that the actual strength of the existing structure possesses that strength. It is a well-known fact that concrete strength increases with age beyond 28 days and stabilizes after a certain time. Generally, designers of concrete structures do not take such increases into consideration mainly to offset "ignorance factors" in areas of design and construction.

Provide a justification that such additional margins of safety are not required in the case of a concrete containment, noting that there is a reduction in dome concrete area due to the presence of cracks, sheathing ducts and other possible voids, and if such reduction of concrete area is disregarded in the stress computation, the computed membrane compressive stress may be less than the actual.

Answer: The in-place concrete strength is usually not taken into account in design of structural concrete. The reason for this practice is that the in-place strength is not known at the time the design is performed. However, it is also current practice to use a design strength (f'_c) based on an age closer to the time of first service loads rather than based on an arbitrary age (e.g., 28 days).

For the Crystal River Unit 3 Reactor Building Dome, the in-place strength has been evaluated in accordance with the accepted practice of Chapter 4, Section 4.3.3 of ACI 318-71 and the compressive strength has been determined to be 5130 psi (See Table 3-2, Page 3-15, Dome Delamination Report.). Another calculation, using ACI 216 (Midcell Method) and ACI 318, Section 4.3.5.1, had given a compression strength of 5600 psi (See Page C-5, Dome Delamination Report.).

Therefore, there is sound technical basis for using a design in-place compressive strength of 6000 psi.

With regard to "...presence of cracks, sheathing ducts and other possible voids...":

1. The lower level cracks are parallel to the membrane and do not constitute a reduction in the concrete area available to carry membrane forces. They have been successfully grouted (see Attachment I). Section 5.2.6.
2. "Sheathing ducts" are 5" diameter Schedule 40 pipe and replace the displaced concrete. See Supplement 1, August 10, 1976 revision, page 5-8, Question 6 for additional detail.
3. We are not aware of "other possible voids." Considering the number of cores taken in the Crystal River Unit 3 dome (in excess of 2000), it is unlikely that any voids exist in the dome.

Considering the above 3 factors and the actual response of the structure to 15% detensioning program, "computed membrane compressive stress" should be quite close to actual stress seen by the structure under any load combination.

6. The cracks in the dome concrete as discussed in the general comments have reached stability. The Structural Integrity Test (SIT) will affect such stability. Provide an evaluation of SIT on the lower level cracks of concrete which may not be grouted with epoxy. Provide the data on the effectiveness of epoxy grout in controlling concrete cracks.

Answer: The current through-thickness stresses in the dome are compressive (see Figure 5-22, September 22, 1976 revision). The pressurization of the Reactor Building for the SIT will increase the existing radial compression through the entire thickness of the repaired dome. The added radial compression will vary from 63.3 psi on the inside surface to zero (0) on the outside surface. Since the through-thickness stresses will still be compressive, they will not disturb the stability of the lower level cracks. Although not essential to the structural behavior during the SIT, the epoxy grouting of lower level cracks has been accomplished (see response to Item II.5) and should enhance through-thickness stability.

III. CAUSES OF DELAMINATION.

1. On Page C-3 in Appendix C under the subsection on "~~Direct Tensile Test Results~~" the applicant indicates that the range of direct tensile tests on 6 core samples was 230 psi to 505 psi with an average value of 420 psi. In view of these low results, the allowable membrane tensile stresses indicated in table 2-2 appear high. Discuss the cause of these low tensile ultimate stresses, the reason for the wide scattering of the test results and the possibility that the delamination phenomena was caused by the poor quality of the aggregate, and the propagation of local cracks along the whole surface of the dome as surmised in the general comments above.

Answer: The variation as well as the cause of the "low tensile ultimate stress" is discussed in Appendix C of the report. Reactor Building Dome Delamination. The table on page C-15 (Attachment D) presents direct tensile strengths, test results and describes in the remarks column the relative "hardness" of the coarse aggregate. A review of that table indicates tensile strength is related to "hardness" of coarse aggregate. Also see Page C-6 for a discussion of direct tensile tests by Mr. Joseph F. Artuso.

The "...poor quality of the aggregate, and the propagation of the local cracks along the whole surface of the dome..." has been discussed in Section 3.3 and Appendix F as being a major contributor to the delamination.

2. The applicant presented in Fig. 3-22 the plane strain finite element model used to evaluate some stress concentrations at the tendon ducts.
 - a. Present a detailed description of boundary conditions (especially at the duct) and initial conditions introduced in the computer analysis for all cases of stress concentration.

Answer: The model shown in Fig. 3-22 was used to calculate stresses in the concrete due to shrinkage effects. At the interface of concrete and duct, perfect bond was assumed because of compressive interface pressure. The outside boundary was assumed to be free. Rollers on the boundaries were used to simulate symmetry. The model was assumed to be stress-free prior to application of the shrinkage effects. The geometry and material behavior was assumed to be linear.

- b. Justify the use of plane strain to analyze what is essentially a three-dimensional problem.

Answer: The plane strain model is not intended to accurately describe the real situation (for example, 3 layers of conduit, double curvature and loads induced by the tendon in the conduit). It was, however, considered adequate to examine the replacement effect of the 5" Schedule 40 pipe.