Question #8 (Request Number 14)

NRC Question:

For the original structural integrity test, were there any strain gauges in the construction opening area or near it?

CR3 Response:

Section 5.3.2 of the Dome Repair report included with Letter 3F1276-10 outlines where the strain gauges were attached. This section also refers to Figure 5-1 for locations.

In addition to the final report, Attachment 1 to Supplement number 2 (transmitted via letter 3F1076-05) contained a detailed listing of strain gages for the SIT. The construction opening is centered on azimuth 150° (between buttresses 3 and 4) from Elevations 180' to 210'. The listing in Letter 3F1076-05, Attachment 1, does not show any gages in this area. The closest would be at azimuths 90° and 200° at Elevation 204' (gages 13, and 15).

The SIT report (GAI Report 1930, dated 12/7/76) contains radial displacements for these gages (See Appendix B, Page B-5 of the GAI report).

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5.3.2 Instrumentation

During detensioning and retensioning, the vertical movement of the apex of the dome at the top of the cap and the top of the lower concrete would be observed. These observations would be correlated with readings of the position of the bottom of the dome at the apex. In addition, the movements at eight other locations on two orthogonal radii, 70° and 345° orientation, would be monitored from the outside of the dome (see Figure 5-1 for locations).

The meridional and circumferential strains at three existing gages would also be monitored as well as the existing temperature gages.

In addition to the existing gages, eight sets of gages would be set in the existing concrete and on the steel liner to monitor strains. The gages would be located at 15, 30, and 45 feet from the dome apex as near the 75° and 345° lines as the physical conditions allow.

Gages would also be located on a series of the radial anchors along the axis to measure any changes in bolt strain.

The remainder of the instrumentation originally intended for the Structural Integrity Test (SIT) would also be monitored.

5.3.3 Dome Detensioning and Retensioning

Unbalanced loads due to unsymmetrical tensioning of the structure existed during the initial tensioning of the dome tendons, see Section 3.3.9. It would be difficult to perform such operations without minor imbalances. Although the lack of symmetry is not assumed to have been a major contributor to the delamination, a more balanced system of detensioning and tensioning has been developed. In reviewing concerns that a variation from the original tensioning sequence might produce undesirable effects, it should be noted that the detensioning is being done for a different structure, (a delaminated dome), than the one that was initially tensioned. The sequence results in the same tendons being stressed as in the original sequence at the 25, 50, 75 percent points.

To maintain an even balance of prestress forces during detensioning a triangular pattern of three tendons, concentric with the dome center would be jacked either simultaneously, or one after the other beginning with the bottom tendon in a group of three. This would help ensure a balance of both membrane forces and forces normal to the dome from the tendon system.

The detailed detensioning sequence is given in Table 5-1 and the pattern of tendons remaining tensioned at various stages during the detensioning operation are shown on Figures 5-2 through 5-8. At the stage when only 30 tendons remain stressed, sequence 32 through 41 are repeated twice, detensioning to approximately 50% of the existing tension in each tendon in the first stage and completion of detensioning of each tendon for the second and final stage.

Florida Power
CORPORATION

DOCKET COPY

MICROFILMED

October 14, 1976

3F1076-05
Mr. John Stolz
Branch Chief
Light Water Reactors Branch I
Division of Project Management
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

REACTOR BUILDING DOME DELAMINATION REPORT, SUPPLEMENT 2

Subject: Crystal River Unit #3
Docket No. 50-302

Dear Mr. Stolz:

We are today filing Supplement No. 2 to our interim report, "Reactor Building Dome Delamination - June 11, 1976 (40 copies plus 1 original)

This Supplement No. 2 provides you with our response to Structural Engineering Branch (NRC) request for additional information transmitted to us by telecopier on September 30, 1976.

Please insert this Supplement into your copies of the report document for completeness.

Very truly yours,

J. T. Rodgers

J. T. Rodgers
Asst. Vice President

JTR/iw
Attachment.

cc: Mr. Norman C. Moseley
Director, Region II I&E
Atlanta, GA

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. <u>GENERAL COMMENTS</u>.

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.

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:

356

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.

354

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.

349

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 6130 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 6600 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 atresses 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.

ATTACHMENT 1

Florida
Power
CORPORATION

October 8, 1976

Mr. John Stolz
Branch Chief
Light Water Reactors Branch I
Division of Project Management
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: Crystal River Unit #3
Docket No. 50-302

Dear Mr. Stolz:

Attached are our Instrument Location, pages 1 and 2, and our Displacement Acceptance Criteria, pages 1 and 2. These are for your information and use in the evaluation of our Crystal River Unit 3 Structural Integrity Test results.

Inasmuch as you did not specify any particular format for this input, we have chosen this one as representative of our understanding of the staff request. Also, this submittal is being made at least two weeks prior to the conduct of SIT.

Please advise if unacceptable, or if you need any further information in this regard.

Very truly yours,

J. T. Rodgers
J. T. Rodgers
Asst. Vice President

JTR/iw
Attachments

bc: BLC
ECS
[illegible]

General Office 3201 Thirty-fourth Street South • P.O. Box 14042, St. Petersburg, Florida 33733 • 813 866-5151 363

INSTRUMENT LOCATION

The location of displacement measurements are as follows:

Cylinder Wall and Dome Junction Radial Displacements

LP Gage Loc.	Elevation	Azimuth	Notes
1, 2, 3	98'-0"	90°, 200°, 333°-55'	Radial Displacement
4, 5, 6	108'-0"	90°, 200°, 333°-55'	Radial Displacement
7, 8, 9	140'-0"	90°, 200°, 333°-55'	Radial Displacement
10, 11, 12	172'-0"	90°, 200°, 333°-55'	Radial Displacement
13, 14, 15	204'-0"	90°, 200°, 333°-55'	Radial Displacement
16, 17, 18	236'-0"	90°, 200°, 333°-55'	Radial Displacement
19, 20, 21	246'-0"	90°, 200°, 333°-55'	Radial Displacement
22, 23, 24	253'-0"	90°, 200°, 333°-55'	Radial Displacement
25, 26, 27	267'-0"	90°, 200°, 333°-55'	Radial Displacement
128, 129, 130	270'-8"	90°, 200°, 333°-55'	Radial Displacement

Ring Girder Vertical Displacement - LP

LP Gage Loc.	Elevation	Azimuth	Notes
28, 29, 30	267'-6"	90°, 200°, 333°-55'	Vertical Displacement

Dome Vertical Displacement - LP

LP Gage Loc.	Elevation	Location	Notes
34	282'-4 1/8"	Dome Apex	Vertical Displacement
128, 129, 130	49'-3" radius	90°, 200°, 333°-55'	Vertical Displacement
164, 165, 166	28'-8" radius	90°, 200°, 333°-55'	Vertical Displacement

Equipment Access Opening Displacement - LP

LVDT Gage Loc.	Elevation	Notes
35, 37, 38, 39 40, 41, 42	132'-0"	Radial Displacement
35, 37, 38, 39, 40, 41, 42	132'-0"	Vertical Displacement

Equipment Access Opening Displacement - LP (Cont.)

LVDT Gage Loc.	Elevation	Notes
36	120'-0"	Radial Displacement
36	120'-0"	Vertical Displacement
43	144'-0"	Radial Displacement
43	144'-0"	Vertical Displacement
44	147'-3"	Radial Displacement
44	147'-3"	Vertical Displacement
45	151'-6"	Radial Displacement
45	151'-6"	Vertical Displacement
46	155'-6"	Radial Displacement
46	155'-6"	Vertical Displacement
47	159'-6"	Radial Displacement
47	159'-6"	Vertical Displacement
48	163'-6"	Radial Displacement
48	163'-6"	Vertical Displacement

365

Displacement Acceptance Criteria

Gage ID	Measurement	Theoretical Displacement (inches)	Limiting Displacement (inches)	Tolerance (inches)
1, 2, 3	Radial	0.010	0.020	0.010
4, 5, 6	Radial	0.090	0.115	0.025
7, 8, 9	Radial	0.205	0.260	0.055
10, 11, 12	Radial	0.200	0.250	0.050
13, 14, 15	Radial	0.205	0.260	0.055
16, 17, 18	Radial	0.160	0.205	0.045
19, 20, 21	Radial	0.060	0.080	0.020
22, 23, 24	Radial	-0.025	-0.040	0.015
25, 26, 27	Radial	-0.055	-0.070	0.015
28, 29, 30	Vertical	0.215	0.275	0.060
34	Vertical	0.905	1.135	0.230
35	Radial	0.10	0.130	0.030
35	Vertical	0.10	0.130	0.030
36	Radial	0.08	0.105	0.025
36	Vertical	0.10	0.130	0.030
37	Radial	0.12	0.155	0.035
37	Vertical	0.10	0.130	0.030
38	Radial	0.115	0.150	0.035
38	Vertical	0.10	0.130	0.030
39	Radial	0.115	0.150	0.035
39	Vertical	0.10	0.130	0.030

Displacement Acceptance Criteria (continued)

Gage ID	Measurement	Theoretical Displacement (inches)	Limiting Displacement (inches)	Tolerance (inches)
40	Radial	0.11	0.140	0.030
40	Vertical	0.10	0.130	0.030
41	Radial	0.11	0.140	0.030
41	Vertical	0.10	0.130	0.030
42	Radial	0.11	0.140	0.030
42	Vertical	0.10	0.130	0.030
43	Radial	0.090	0.120	0.030
43	Vertical	0.105	0.135	0.030
44	Radial	0.090	0.120	0.030
44	Vertical	0.105	0.135	0.030
45	Radial	0.095	0.125	0.030
45	Vertical	0.110	0.140	0.030
46	Radial	0.100	0.130	0.030
46	Vertical	0.11	0.140	0.030
47	Radial	0.105	0.135	0.030
47	Vertical	0.115	0.150	0.035
48	Radial	0.110	0.140	0.030
48	Vertical	0.120	0.155	0.035
128, 129, 130	Radial	0.015	0.025	0.010
128, 129, 130	Vertical	0.365	0.460	0.095
164, 165, 166	Vertical	0.900	1.130	0.230

ATTACHMENT 2

JOHN C. KING, P.E.
CONSULTING ENGINEER

101 East 252 Street / Cleveland, Ohio 44132
(216) 731-0177

September 23, 1976

Dr. F. L. Moreadith
Gilbert Associates, Inc.
P.O. Box 1498
Reading, Pa. 19603

F. L. Moreadith
SEP 27 1976

Subject: CR3 Dome Repair
Epoxy Grouting

Dear Dr. Moreadith:

In my opinion the epoxy grouting of delamination and other cracks, if any, in the CR3 Dome has been a complete success. I estimate that between 90 and 100% of the cracks are filled with very strong epoxy bonded to both surfaces of the cracks.

The above conclusion is based on (1) my observation of the entire operation, (2) very few of quite a number of holes actually missed or presumably missed (for lack of logs showing that holes had been grouted) took any grout, and (3) a random selection of 10 holes that had been grouted and redrilled accepted no grout whatever at 200 psi.

Details of my observations are given in Field Notes on Epoxy Grouting of Cracks, copies of which have been sent to you by Mr. John Herr.

Sincerely,

John C. King

John C. King