Florida Power
CORPORATION

August 10, 1976

United States Nuclear Regulatory Commission
Washington, D.C. 20555

Attention: Mr. John F. Stolz, Chief
Light Water Reactor Branch No. 1
Division of Project Management

Re: Florida Power Corporation
Crystal River Unit #3
Docket No. 50-302

Gentlemen:

In accordance with your letter of July 30, 1976 relative to the request for information concerning our interim report, "Reactor Building Dome Delamination," June 11, 1976, we submit the attached response as supplement number 1. Forty (40) copies are included.

We have prepared this response for submittal at this time per your request as a supplement to the interim report and in the next two weeks we will forward the agreed upon correction/addition pages to amend the report to reflect the repair procedures being followed subsequent to July 27, 1976.

It is anticipated that the format being followed will allow a single report to suffice in furnishing the necessary information for you and your staff. Additional supplemental pages will be submitted as required by your review.

We will discuss this matter with Mr. Engle today after our presentation on the repair status.

Very truly yours,

J T Rodgers

J. T. Rodgers
Assistant Vice President

JTR:cd

cc: Mr. Norman Moseley
Atlanta, Georgia
Region II Inspection & Enforcement

8003120 795

SUPPLEMENT 1

NRC COMMENTS TO THE CRYSTAL RIVER #3

REACTOR BUILDING DOME DELAMINATION REPORT

DATED JUNE 11, 1976

DOCKET NO. 50-302

GENERAL

1. For easy reference, provide a list of tables and figures in the Table of Contents.

Answer: The material suggested has been incorporated into the Table of Contents.

SECTION 1.2

1. The staff considers the establishment of the causes of the dome delamination to be important in assessing the adequacy of the repair program and in providing assurance that another crack will not occur again during the life of the structure. The potential contributing factors should, therefore, be identified indicating the magnitude of radial tensile stresses created in the concrete.

Answer: The material has been incorporated into the report in Section 1.2.

2. The use of radial anchors will enhance the capability of the dome to resist radial tension. However, they will not eliminate tension in concrete, and therefore small cracks may still exist. Provide an analysis to indicate that such cracks will not jeopardize the required structural integrity of the dome to resist all combinations of loadings for which it is designed.

Answer: These cracks exist primarily in regions where membrane behavior dominates, i.e., negligible shear stress across the cracks.

Despite the presence of the cracks, the membrane compression capacity of the concrete is adequate. Under LOCA or SIT, there is radial compression across the cracks.

It is of interest to note that under 15% detensioning (admittedly a nominal stress charge) deflections were less than predicted. If the cracks were contributing any significant effect to the response, larger rather than smaller deflections would be expected.

In addition to the above considerations, secondary cracks will be epoxy grouted and the radial reinforcing will cross virtually all these cracks.

SECTION 2.3 AND TABLE 2-2

1. Clarify the definition of tensile capacity of concrete. Explain how principal tension is related to shear and diagonal tension as indicated in Section 2.3.1, and what is the difference between the shear discussed in this section and that in the next section (2.3.2).

Answer: The material in sections 2.3.1 and 2.3.2 have been rearranged in the report, section 2.3.1 title is "Flexural and Membrane Tensile Stresses" and section 2.3.2 is entitled "Shear".

2. Provide and describe with examples of actual design, the conditions under which each of the criteria (a) and (b) in Section 2.3.1 is applied.

Answer: The material requested now appears in section 2.3.2. Attachment #1 is a design example.

3. Since the stress/strain distribution is tri-axial, the limits of 3 $\sqrt{f'_c}$ and 6 $\sqrt{f'_c}$ may not be directly applicable to this problem and their use should be justified.

Answer: The state of stress in the dome may be regarded as being biaxial since the stress in the radial direction is very small in comparison with the membrane stresses. The interactions for tension-tension and tension-compression are not significant at least until the compression exceeds about 60% of the compressive strength of the concrete (Kupfer, Hilsdorf and Rusch, ACI Journal Aug. 1969) (Ref. 8 of the Report). Thus the limits of 3 $\sqrt{f'_c}$ and 6 $\sqrt{f'_c}$ are justified.

4. If $0.85f'_c$ as extreme compression in ultimate strength design is used, it may not be directly applicable for the same reason as in the above comment and should be justified.

Answer: Although criteria indicate that under factored load concrete stresses would be allowed to reach $0.85f'_c$ they do not. The actual stresses are much lower and do not appear critical since the dominant stress is bi-axial compression the strength should be higher.

5. The shear strength of concrete is influenced by stresses orthogonal to the axis of the element; therefore, this effect should be considered.

Answer: Hoop tension stresses should have little or no effect on radial shear strength, since sufficient bonded hoop rebar has been provided to preclude hoop tensile "Failure".

NRC Question 2.3.2 - Shear Design Examples

Case a. Membrane tension, or membrane comp. < 100 psi

Example | LOCA 36" Dome (Fig 2-30 & 2-32)
Sta. 157.5°

From shell analysis
$V = 9.4^{k/1}$ $M_\phi = 172 \frac{1\text{-}k}{1}$ (Ten OF) $\bar{\sigma}_\phi = 98$ psi (comp) membrane

Since $\bar{\sigma}_\phi = 98$ psi (comp) < 100 psi (comp), use 318-63, ch 17.

Calculate conc. capacity using (17-2) with (17-3).

$\frac{\phi A_v d f_y}{s} = 0$ since no shear rein. avail. at 157.5°.

From (17-3),

$$M' = M - N\left(\frac{4t-d}{8}\right) = 172 - 42.3\left(\frac{4\times36-31.8}{8\times12}\right) = 122.5\ \frac{1\text{-}k}{1}$$

$N = 98\text{ psi} \times 12'' \times 36'' = 42.3^{k/1}$
$t = 36''$ $d = 31.8''$ (dist. from IF to OF rebar)

From (17-2),

$$v_c = \phi\left(1.9\sqrt{f_c'} + 2500\rho_w \frac{Vd}{M'}\right) \qquad \left(\rho_w = \frac{A_s}{bd} = \frac{1.20}{12\times31.8} = 0.003\right)$$

$$v_c = 0.85\left[1.9\sqrt{5000} + 2500(.003)(9.4)(31.8)/122.5\right]$$
$$v_c = 129.8\text{ psi } (< 3.5\phi\sqrt{f_c'} @ 210\text{ psi})$$

$$V_c = bd\,v_c = \frac{(12)(31.8)(129.8)}{1000} = \boxed{49.5^{k/1}}$$ Fig 2-32

Ultimate applied shear, V_u, $= V = 9.4^{k/1}$

$\boxed{49.5^{k/1} \text{ capacity} > 9.4^{k/1} \therefore \text{section ok}}$ Fig 2-32

NRC Question 2.3-2 — Shear Design Examples

Case b. Membrane Compression > 100 psi

Example | Norm. Winter Oper. Cond. 36" Dome (Figs 2-18 & 2-20)
Sta. 138.3°

From shell analysis

$V = 119.3^{k}/1$ $M_\phi = 277 \frac{1\text{-}k}{1}$ (Ten OF) $\sigma_\phi = 1384$ psi (comp)

Membrane stress = 1384 psi comp > 100 psi comp

∴ use 318-63, Ch 26 (P/s Conc)

Try to show OK using minimum concrete capacity, ϕV_c, plus shear rein., $\frac{\phi A_v d f_y}{s}$.

If

$$\phi V_{c\,min} + \frac{\phi A_v d f_y}{s} \geq V_u \text{ , OK ;}$$

if not, calc V_c from (26-12) with FSAR exceptions.

$\phi V_{c\,min} = \phi (1.7 b' d) \sqrt{f_c'}$, $d = 18.4$" (dist from extreme comp. fiber to cg tendons)

$= (0.85)(1.7)(12)(18.4)\sqrt{5000}$

$\phi V_{c\,min} = 22{,}560\ \#/1 = \boxed{22.6^{k}/1}$ concrete

$\frac{\phi A_v d f_y}{s} = \frac{(0.85)(0.79)(31.8)(40 ksi)}{4.5} = \boxed{189.8^{k}/1}$ shear rein.

d = dist. from extreme comp fiber to tension rein.
s = 4½" meridional spacing of #8 shear bars (12" hoop spa)

∴ Conc + Rein : $\boxed{212.4^{k}/1 \text{ Capacity}}$

Applied (factored), $V_u = 1.5V = 1.5(119.3^{k}/1) = \boxed{179.0^{k}/1}$

$212.4^{k}/1$ capacity > $179.0^{k}/1$ applied (factored), ∴ section is OK based on minimum conc shear capacity.

NRC Question 2.3-2 — Shear Design Examples

Case 6.-1 Membrane Compression > 100 psi

Example | Norm. Winter Oper. Cond. 36" Dome (Figs 2-13 & 2-20)
Sta 152.0°

From shell analysis

$V = 45.5^{k}/_{1}$ $\quad M_{\phi} = 307 \frac{1\text{-}k}{1}$ (Ten OF) $\quad \sigma_{\phi} = 1488$ psi (comp)

$V_u = 1.5V = 1.5(45.5) = 68.3^{k}/_{1}$

Membrane stress = 1488 psi comp > 100 psi comp
∴ use 318-63, Ch 26 (P/S Conc)

Try min V_c + shear rein (#8@18" V&H)

$\phi V_{c\,min} = 22.6^{k}/_{1}$ (obtained previously)

$$\frac{\phi A_v d f_y}{s} = \frac{(0.85)(.79/1.5)(31.8)(40)}{18"} = \boxed{31.8^{k}/_{1}}$$

total cap. = $22.6 + 31.8 = 54.4^{k}/_{1} < 68.3^{k}/_{1}$
∴ calc actual V_c from (26-12) w/FSAR exceptions

$$V_{ci} = K_{\Delta v} b'd\sqrt{f'_c} + \frac{M_{cr}}{\frac{M}{V} - \frac{d}{2}} + V_d$$

$$K_{\Delta v} = 1.75 - \frac{0.036}{np} + 4np$$

$$M_{cr} = \frac{I}{y}\left(6\sqrt{f'_c} + f_{pe} - f_d\right)$$

$f_{pe} - f_d$ = compressive stress in concrete due to prestress and dead load after all losses, at the extreme fiber of the section at which tension stresses are caused by applied loads

M and V are due to all loads
d = 18.4" (dist. from extreme comp. fiber to cg tendons)

Case 6.-1 (con't)

Since M_ϕ causes tension on OF, $\rho = \rho_{OF}$.

$\rho = \rho_{OF} = A_s/bd = 1.60/(12)(31.3) = 0.0042$

$n = 29/4 = 7.25$

$K_{\Delta V} = 1.75 - \dfrac{0.036}{(7.25)(.0042)} + 4(7.25)(.0022) = 0.692 > 0.60$ use 0.60

$I = \frac{1}{12}(12)(36)^3 = 46656\,in^4$, $y = 36/2 = 18''$
$6\sqrt{f_c'} = 6\sqrt{5000} = 424\,psi$

$f_{pe} - f_d = -910$ psi comp (D+F: -910 OF, -2300 IF)

$M_{cr} = \dfrac{46656}{18}(6\sqrt{5000} + 910) \times \dfrac{1}{12000} = 288\ \frac{1\text{-}k}{1}$

$V_{ci} = \dfrac{.6(12)(18.4)(70.7)}{1000} + \dfrac{288}{\frac{307}{45.5} - \frac{18.4}{2 \times 12}} =$

$V_{ci} = 9.4 + 48.2 = 57.6\ ^k/_1$

From (26-13), diag. cr. shear (V_{cw}),

$V_{cw} = b'd(3.5\sqrt{f_c'} + 0.3 f_{pc})$

$f_{pc} = \bar{\sigma}_\phi = 1488$ psi (comp), $d = 0.80 \times 36'' = 28.8'$ ($> 18.4''$)

$V_{cw} = 12(28.8)(3.5\sqrt{5000} + 0.3 \times 1488)$
$V_{cw} = 240\ ^k/_1 > V_{ci}$

$\therefore \boxed{V_c = V_{ci} = 57.6\ ^k/_1}$

Total Capacity of Conc + Rein $= 57.6 + 31.8 = 89.4\ ^k/_1$

$\boxed{\text{Capacity @ } 89.4\ ^k/_1 > V_u \text{ @ } 68.3\ ^k/_1 \ \therefore \text{section OK}}$ Fig 2-20

SECTION 2.4

1. In the paragraph in the middle of Page 2-4, you indicated that for structural integrity test and accident condition load combinations, stresses for sustained loads cannot be combined with those due to rapidly applied loads internally in the program and are combined externally. Provide an example of actual design to show how the stresses are combined externally and illustrate the combination on a stress-strain diagram.

Answer: See Attachment 2.

2. On Page 2-5 under Item b Creep, it is indicated that as a result of concrete creep there is a reduction in concrete stress and an increase in liner stress. Since the liner is relatively thin and may buckle under prestress, the liner should not be considered to contribute any strength to the containment vessel. However, in the design of the steel liner, strain due to creep of concrete should be considered to check its leaktightness integrity. Revise the concrete stresses in the report if they have been reduced.

Answer: A reduced modulus of elasticity of concrete has been used in the analysis and thus the effect of creep on concrete and liner stresses has been accounted for. Our analysis indicates that for the load combinations D+F and D+F + T_o the concrete stress is increased if the liner is removed in the analytical model. From the standpoint of concrete stress behavior for the SIT and LOCA load combinations, to remove the liner from the analytical model is not conservative.

The figures in the report have been modified to provide a comparison of both results at selected points.

3. Provide the procedure which you used in the design of the steel liner. In Table 2-2, you stated that no criteria on liner strains were used in the original design. Indicate the criteria you used for the steel liner design.

Answer: Table 2.2 has been modified to reflect liner design criteria.

4. Discuss in detail the effects of creep, including the following consideration:

> Because of the different level of prestress in the wall in the vertical direction, the wall in the hoop direction, in the ring girder and in the dome the E_c' is different in all these directions and this effect should be considered in the analysis. The wall acts as an orthotropic element. The different parts of the structure have simultaneously different E_c' due to different specific creep.

Answer: The effect of creep has been accounted for by the use of reduced modulus. Although the different parts of the structure have different prestress, the specific creep (creep due to unit psi) should be the same for the same material. Thus the reduced modulus should be about the same for the various parts of the structure. A calculation is attached to demonstrate this (See Attachment 3).

5. In Table 2-3 add load combination equation for repairs. This equation should include the seismic load term.

Answer: The FSAR and the current ASME Code load combinations do not include earthquake effects in combination with construction loads.

ATTACHMENT 2 FOR ANSWER TO QUESTION 2.4(1)

EXAMPLE OF SIT CALCULATION

24" DOME SIT

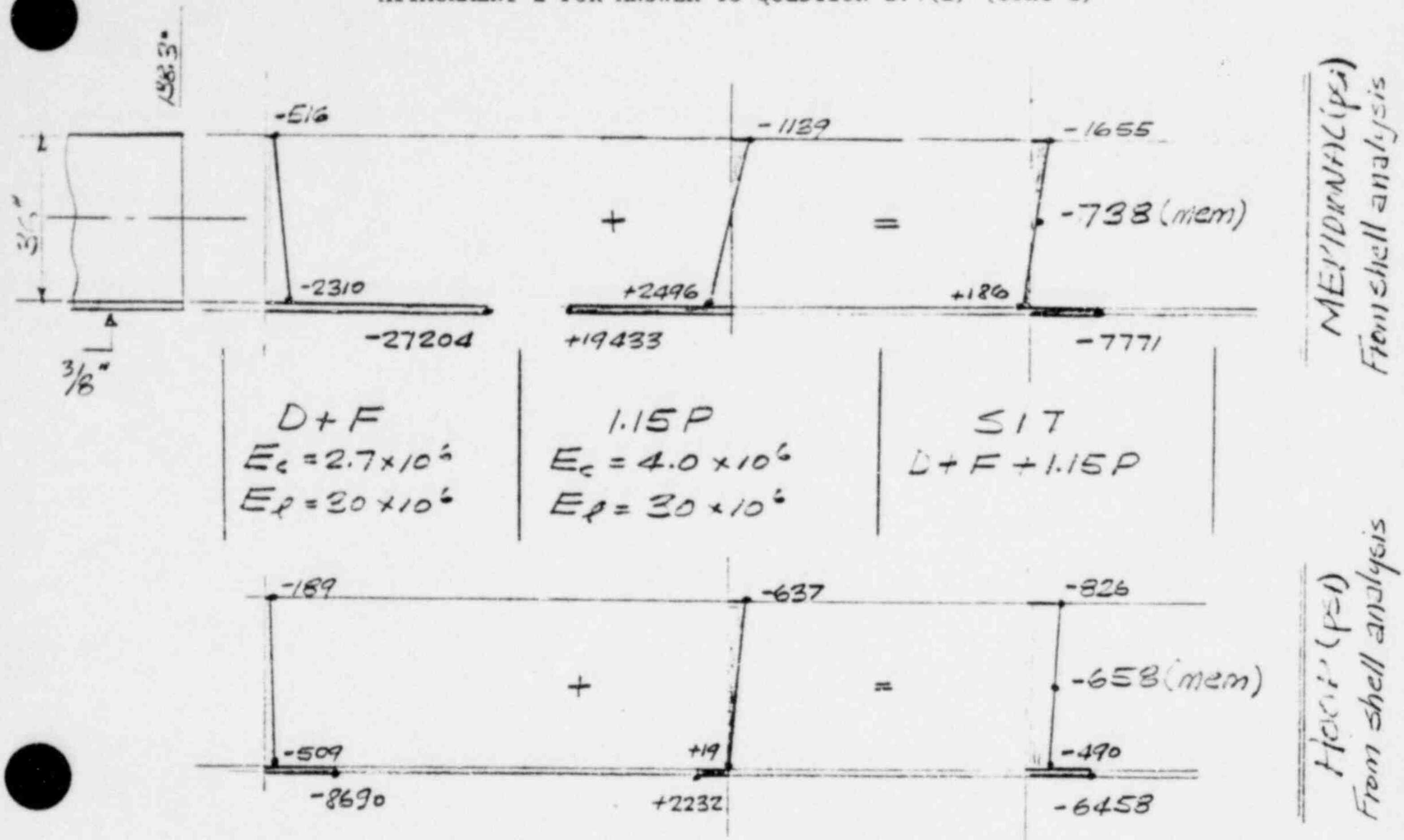
Ref: Figs 4-14 thru 4-17

$E_c = 2.7 \times 10^6$, $\nu_c = 0.20$ D+F

$E_c = 4.0 \times 10^6$, $\nu_c = 0.20$ 1.15P

$E_l = 30 \times 10^6$, $\nu_l = 0.30$

ATTACHMENT 2 FOR ANSWER TO QUESTION 2.4(1) (Cont'd)



Liner Strains

$$\epsilon_\varphi = \frac{-7771-0.3(-6458)}{30\times10^6} = -0.000194\,''/''$$

$$\epsilon_\theta = \frac{-6458-0.3(-7771)}{30\times10^6} = -0.000138\,''/''$$

IF Conc. Strains

$$\epsilon_\varphi = \frac{-2310-0.20(-509)}{2.7\times10^6} + \frac{+2496-0.2(19)}{4\times10^6} = -0.000195\,''/''$$

$$\epsilon_\theta = \frac{-509-0.2(-2310)}{2.7\times10^6} + \frac{19-0.2(2496)}{4\times10^6} = -0.000137\,''/''$$

(Total SIT Conc. Strains cannot be calc. from total conc stresses)

ATTACHMENT 3 FOR ANSWER TO QUESTION 2.4(4) - CREEP

Creep of the concrete under sustained loads D, F_V, F_H, and F_D has several effects on tendon forces and containment stresses. The most obvious is to decrease the tendon forces with time. This effect is taken into account in the prestress loss calculations.

Another effect is to decrease concrete stresses and to increase liner stresses and strains, which are compressive over most of the containment structure. The decrease in concrete stress is due to the additive effects of the decrease in tendon force plus the creep straining of the concrete acting with a non-creeping liner, which tends to shed compressive stresses from the concrete to the liner. This latter effect is taken into account in the analysis through the use of the effective Young's Modulus, E_c', appearing on page 4-3 of the report.

Using this approach, less concrete compression is calculated to be available to resist SIT or LOCA conditions than would be calculated by considering the reduced tendon force alone.

With respect to liner stresses and strains, the structural analyses show that the E_c' effect ($E_c' = 2.7 \times 10^6$ @ present and $E_c' = 1.8 \times 10^6$ @ 40 yr versus $E_c = 4 \times 10^6$ - "instantaneous) is much greater than that of the reduced tendon force. The net result is liner stresses and strains which have compressive values much greater than those which occur at initial prestress. The liner strains in the report include this.

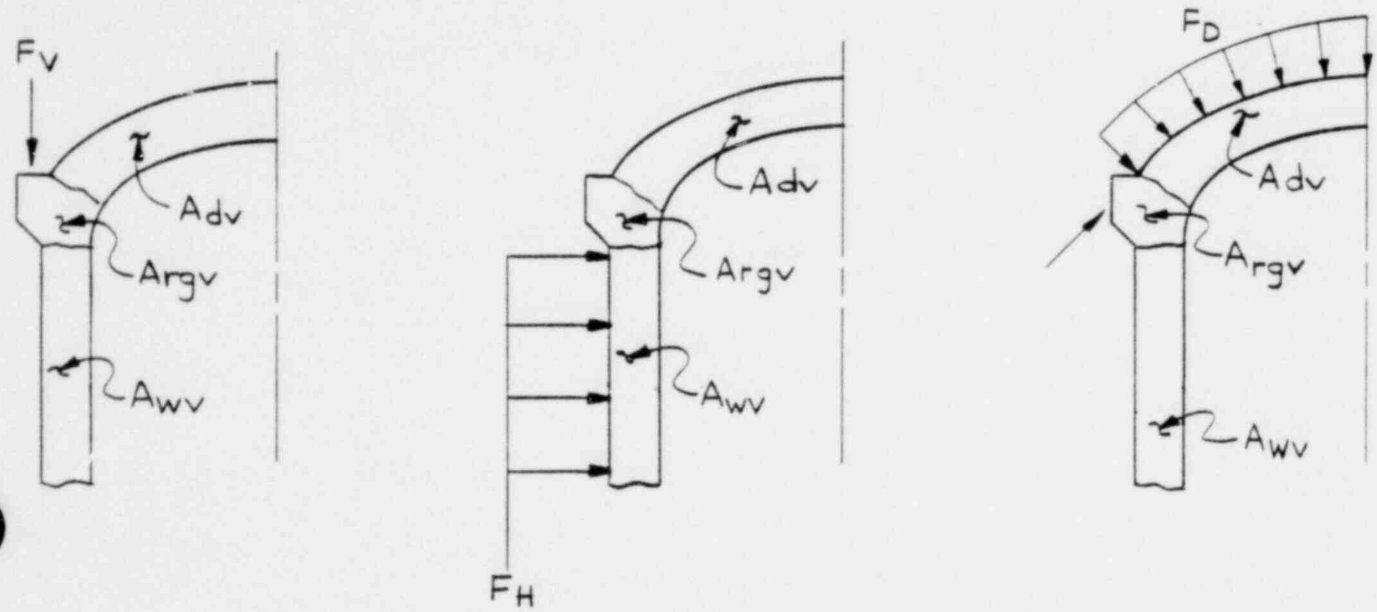
A third effect of concrete creep is to produce creep induced stresses which result only when the E_c' is not uniform over the containment structure. If E_c' is uniform, the stresses at any time are equal to those at initial prestress less tendon losses, in the case where the liner is not part of the model.

The E_c' values used in the structural analyses correspond to specific creep values, sc, which were calculated based on the 1) average age of the dome concrete at application of the dome prestress (average) and 2) the duration of this prestress to "present" and to "40 yrs". The resulting E_c' values were applied to the entire structure in the analyses for D + F. It was recognized that a different E_c' is associated with vertical (F_V), hoop (F_H), and dome (F_D) prestress loading conditions. This is so only because the concrete age at application of each prestress load is different and the duration of each type is different. However, E_c' values were based on the dome concrete age and dome prestress since it is that part of the containment structure which is most effected by the delamination. Also, it was felt that these E_c' values would be an average for the wall since, chronologically, F_D was applied between F_V and F_H. Nevertheless, a more accurate determination of the creep effects due to the separate application and duration of the prestress is discussed below.

As pointed out previously the determination of E_c' depends on 1) age of concrete at loading and 2) duration of load. E_c' is independent of the level of stress in the concrete, which is reflected in sc (μ in/in per 1 psi of stress).

ATTACHMENT 3 FOR ANSWER TO QUESTION 2.4(4) - CREEP (Cont'd)

Therefore, creep induced stress under either F_V, F_H, or F_D will be reflected only in differences in E_c' for the various elements in the containment for each of these prestress loads. The total results would be obtained from the sum of the analyses shown below.



Awv = age of wall at time F_V is applied.

Argv = age of ring girder at time F_V is applied.

Adv = age of dome at time F_V is applied.

Similar for AwH, ArgH, AdH and AwD, ArgD, AdD.

Dv = duration of F_V from time of application to "present" or "40 yr" times.

similar for D_H and D_D

Knowing the values of A and D permit calculation of E_c' for the three elements.

ATTACHMENT 3 FOR ANSWER TO QUESTION 2.4(4) - CREEP (Cont'd)

Values for E_c' were obtained based on average pour dates for the wall, ring girder, and dome and average stressing dates for the three tendon systems. This is presented below.

	Wall	Ring Girder	Dome
Average Pour Dates:	6-19-72	9-1-73	5-15-74

	Vertical	Hoop	Dome
Average Stressing Dates:	11-15-74	2-15-75	12-1-74

				"Present" Time	E_c' (psi) x 10^6			"40 yr" Time	E_c' (psi) x 10^6		
P/S System	Aw (da)	Arg (da)	Ad (da)	D (da)	Wall	Ring Gir.	Dome	D (yrs)	Wall	Ring Gir.	Dome
Vertical	880	425	180	545	3.13	2.89	2.63	41.5	2.08	1.96	1.75
Hoop	970	515	270	455	3.17	3.03	2.86	41.2	2.13	2.04	1.89
Dome	880	455	210	515	3.13	3.03	2.70*	41.4	2.08	2.04	1.80*

* used in structural analysis of containment.

The differences between E_c' values for the wall, ring girder, and dome under a specific prestress condition is not enough to produce stresses significantly different from those reported.

SECTION 3.1

1. Discuss the reliability of direct tensile tests performed on cores. Since in the structure the radial tensile stress occurs simultaneously with two orthogonal compressions or with two orthogonal tensions, a more thorough investigation is required.

Answer: The direct tensile test was designed to identify the tensile capacity of the concrete in the structure in relation to its compressive strength. It was not intended to define the property of the concrete in a state of triaxial stresses, since the actual state of stress at points of stress concentration in the delaminated dome cannot be accurately defined.

The effect of the tensile stress in combination with two orthogonal compressions is discussed in Section 3.3.3 of the report. No further investigations are planned.

SECTION 3.3

1. In the list of factors which may have contributed to the delamination problem, add: creep and stress concentrations (at tendons) inherent in this type of structure.

Answer: Creep in the membrane direction would not increase the radial stress. The effect of stress concentrations is discussed in section 3.3.2.

2. In Section 3.3.2 it is indicated that by using SAP IV computer program and the model shown in Fig. 3-16, the effects of material properties on radial tension stresses are evaluated. Identify in the model:

   (1) the steel elements, such as reinforcing steel, and tendon conduits,

Answer: There is no element representing reinforcing steel or tendon conduit. The effect of reinforcing steel is calculated as transformed concrete area and represented by effective Young's Modulus. Modeling of the tendon conduit is described in Section 3.3.2.

   (2) the manner in which the prestressing force is applied, indicating if the prestressing force component tangent to the dome curvature is considered.

Answer: Prestressing force is applied on three middle layers of the model in both the radial and the tangential directions of the dome.

3. Provide the hand calculation which you made to obtain the radial tension.

Answer: These calculations are included in attachment.

4. In Section 3.3.4, transient thermal gradients may generate shear stresses, and should be considered in the analysis. Similar effect exists for localized thermal gradients.

Answer: Since thermal restraint produces normal strain, but no shear strain, the thermal gradient causes shear stress; but only in the areas which are reinforced for shear (Chapter 14 of Reference 11).

5. The solution for stress concentrations as shown in Fig. 3-17 & 3-18 is incomplete. It should be noted that compression exists also in the direction parallel to the conduit (σ_1). This stress generates additional stress concentration in the plane (σ_2; σ_3) orthogonal to the tendon, which should be added to the stresses shown in Fig. 3-18.

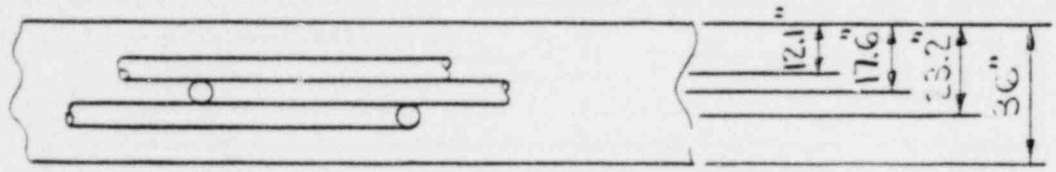
Answer: Assuming this question addresses the effect of Poisson's ratio, this effect was considered and is discussed in Section 3.3.2.

6. When the effect of tendon conduits is analyzed, it should be noted that this effect is different when evaluated in the direction parallel to the tendon and orthogonal to the tendon. In the direction parallel to the tendon a 1/4" thick pipe (5"Ø) approximately replaced the removed concrete. But in the direction perpendicular to the tendon, the pipe introduces a flexible link which modifies the average properties of the concrete section.

Answer: We have reviewed the effect of the conduit on stresses following a path 90° from the stress profile shown in Figure 3-18. The distribution shown in attachment indicates that the effect will not be significant. However, the effect on conduits are conservatively represented by a concrete layer with equivalent Young's modulus calculated by the ratio of net concrete volume to gross concrete volume.

ATTACHMENT FOR ANSWER TO QUESTION 3.3(3)

The radial tension are hand-calculated as follows:



Top Tendon R_1 = 1343.9"

Middle Tendon R_2 = 1338.4"

Bottom Tendon R_3 = 1332.8"

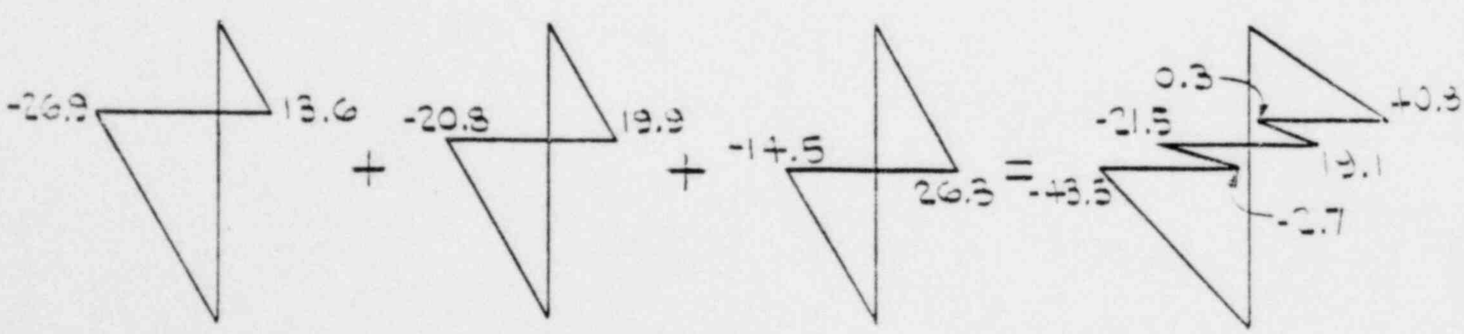
Tendon force at 0.7 ultimate = 1633^k tendon spacing 30"

Top Tendon $\sigma_1 = \frac{1633 \times 1000}{30 \times 1343.9} \times \frac{12.1}{36} = 40.5 \times \frac{12.1}{36} = 13.6$ psi

Middle Tendon $\sigma_2 = \frac{1633 \times 1000}{30 \times 1338.4} \times \frac{17.6}{36} = 40.7 \times \frac{17.6}{36} = 19.9$ psi

Bottom Tendon $\sigma_3 = \frac{1633 \times 1000}{30 \times 1332.8} \times \frac{23.2}{36} = 40.8 \times \frac{23.2}{36} = 26.3$ psi

The radial tension due to all three layers of tendon are superimposed as follows

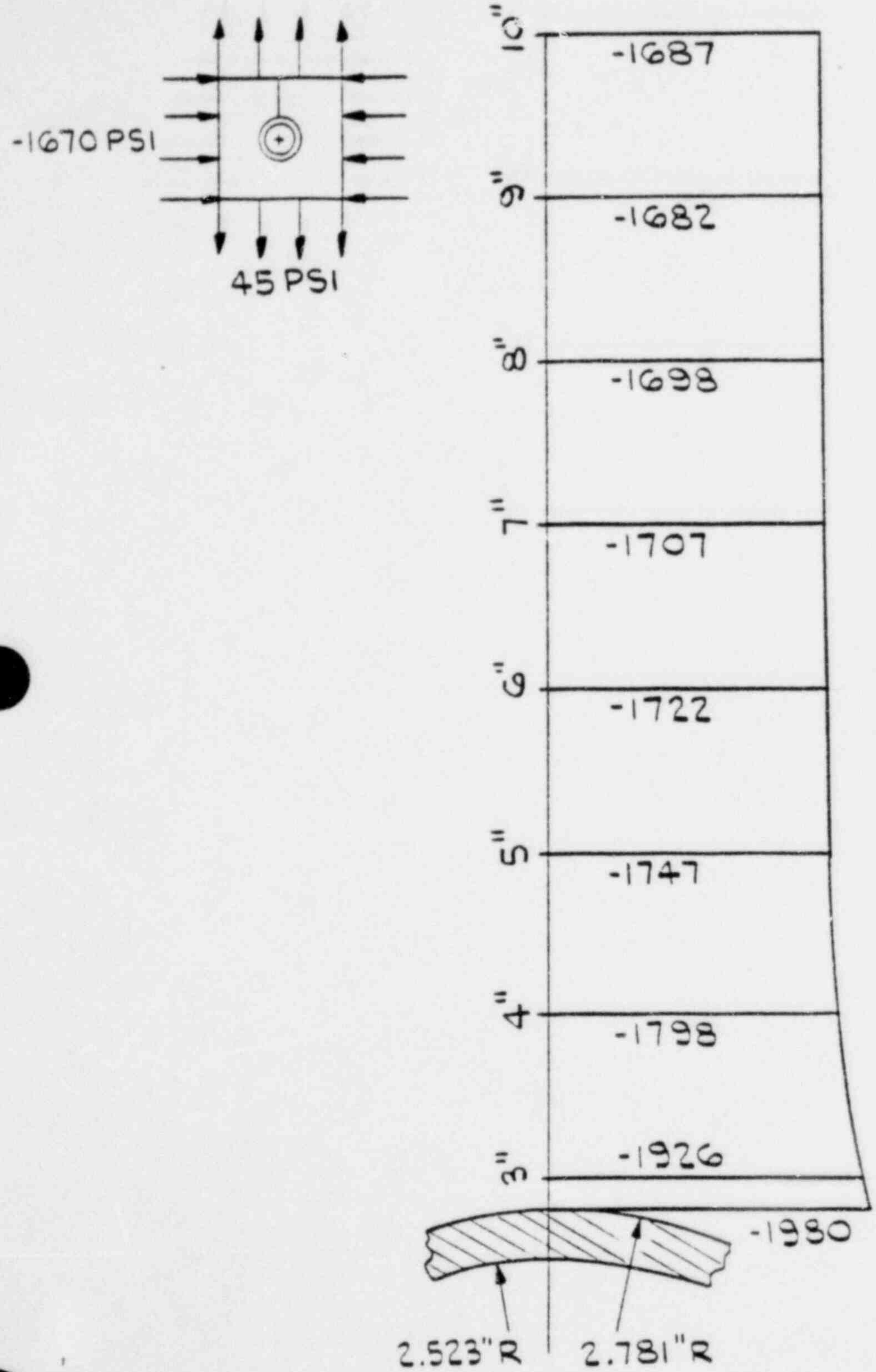


SECTION 4.4

1. In Sections 4.4.1 and 4.4.2 you indicated that in order to consider the containment structure serviceable for the two loading conditions the shear capacity of the tendon conduit would have to be considered. Such consideration may not be possible, unless the bond stress between the conduit and concrete can be justified to be adequate.

Answer: The tendon conduit is not required and has not been considered as contributing to the shear capacity.

ATTACHMENT FOR ANSWER TO QUESTION 3.3(6)



STRESS PROFILE 90° AWAY FROM FIG. 3-13

SECTION 5.3

1. In releasing the prestressing force as a result of tendon detensioning, strain recovery will occur. However, most likely the strain recovery in concrete will be resisted by the steel reinforcing bars and steel liner, because of creep effects, and tension may result in the concrete. Provide an analysis to show that the resulting cracking in dome concrete will not jeopardize the structural integrity of the dome particularly in the region of the liner anchors.

Answer: Not applicable under new repair sequence.

2. The behavior of the detensioned dome is strongly influenced by the creep of the prestressed structure which has taken place after prestressing and up to this date. The detensioning of the dome will not return the structure to a previously unprestressed state, whatever the sequence of operations. It is therefore imperative to analyze the detensioned dome for the influence of creep. Present such an analysis and demonstrate that the integrity of the detensioned dome will not be impaired. The analysis should include the ring girder and the top of the cylindrical wall.

Answer: Not applicable under new repair sequence.

3. The figures 5-11 to 5-14 do not include a study on shears. Provide a detailed analysis of shear stresses in the detensioned dome and demonstrate that these shear stresses, acting simultaneously with normal stresses, do not endanger the stability of the dome. Special attention should be given to radial shears.

Answers: Not applicable under new repair sequence.

4. Either justify in detail the use of 24" for the dome thickness in the present analysis, or present a parametric study for different thicknesses; for instance 24"; 18"; 15".

Answer: The response of the structure to detensioning, and the parametric studies of section 3.0 indicate that the structure is responding as a 24" structure. The addition of epoxy grout, radial anchors and new reinforcing on the cap will assure its continued performance. Also see response to question 1.2.2

5. Demonstrate that the detensioned dome and the steel liner can take the load applied during the repair operations.

Answer: Not applicable under new repair sequence.

6. Present a detailed discussion of the provision made to monitor the behavior of the dome, the ring girder, and the top part of the cylindrical wall during repair operations. Indicate:

   a. The acceptance criteria for safety in such operations, and

Answer: This information has been added to the report as Section 5.0 Corrective Action.

   b. the provisions made to safely stop the repair procedures if the acceptance criteria for safety are not met.

Answer: All activities on the dome will be temporarily suspended and no personnel, except inspectors, will be allowed on the dome after a stop work signal until approval to proceed is obtained from the Engineer and the Owner.

The acceptance criteria shall be in accordance with the requirements noted for each measurement in Table 1. Work shall stop immediately when readings are outside the limits noted in Table 1 for displacements and liner strains and the Engineer shall be notified.

An unsatisfactory set of readings requiring immediate notification of the Engineer during detensioning shall be when one concrete strain or reinforcing bar gage reading exceeds the value specified in Table 1.

The top surface of the dome shall be visually inspected for cracks before commencement of detensioning and any findings recorded. During detensioning and retensioning operations, the inspection for cracking shall be made on a daily basis as a minimum. Observations shall be reported to the Engineer.

7. Describe in detail the methods, acceptance criteria and methods of inspection for the grouting of the cap on the dome, the radial anchors to be installed and the grouting of these anchors. Present the planned testing of these anchors.

Answer: Grouting of cap of dome is no longer part of the repair sequence. The procedure for sizing radial anchors is described in Section 5 of the report.

A test program is being conducted to choose the best set of anchoring devices among the following; a cone and expansion shell anchor system grouted with cement grout, a thread rod with nut bearing grouted with cement grout, a threaded anchor with nut bearing grouted with epoxy grout, and a deformed rod grouted with epoxy grout.

Three (3) types of anchors manufactured by Williams Form Engineering Company have been selected for testing.

1. Williams long cone and long expansion shell (LCS-200).

2. Williams standard cone and standard expansion shell (SCS-200).

3. Williams deformed anchor with and without an end nut. In addition a non-deformed anchor with nut bearing assembly will be tested.

Three (3) different grouts are being tested:

1. Masterflow 814 cement grout.

2. Masterflow 713 cement grout.

3. Sikadur Hi-Mod 370 epoxy.

Following series of tests are conducted to verify the anchor strength.

1. A :allow hole, 2" in diameter and 7 inches deep.

   a. To verify that torquing of bolt will not cause damage or rupture to the nearby concrete.

   b. To establish the failure mode of concrete for available minimum depth.

   c. To establish the design load capacity of the anchor at the minimum available embedment depth.

2. A 2" diameter hole 10 inches deep.

   a. To verify that torquing of bolt will not cause damage or rupture to the nearby concrete.

   b. To establish the failure mode of concrete for this embedment.

   c. To establish the load capacity of anchor for this embedment.

3. A 31 inch deep hole.

   a. To develop and maintain the design preload in the bolt.

   b. Upper and lower bound torque values requirements to develop the design preload.

c. To verify strength of the anchor with respect to concrete capacity.

4. A 31 inch deep hole (epoxy grouted test block).

a. To investigate anchors capacity in epoxy grouted concrete.

b. To establish the failure mode of concrete.

c. To compare the anchor capacity with solid concrete block.

The most suitable anchor type will be established after testing is complete. Final anchor configuration and design basis for the anchor will be submitted as an addenda to the report.

8. Provide a commitment that sufficient strain instrumentation will be installed at the top and bottom of the dome to assure that during retensioning of tendons the upper portion of the dome (above the crack) will be participating in developing compressive stress at the same rate as the lower portion.

Answer: The instrumentation is described in Section 5.0. The gages which exist in the cap will be replaced with strain gages on embedded reinforcing bars and the radial anchors. Observation of this instrumentation during the retensioning and SIT should assure that the structure is responding as designed.

9. Indicate in more detail the planned method of waterproofing of the repaired dome and its protection against detrimental environmental conditions.

Answer: A detailed description will be provided later.

10. Describe the acceptance testing of the repaired dome and the inservice monitoring of the structure.

Answer: Acceptance of the repaired dome will be based on satisfactory completion of the SIT. After the SIT the currently accepted inservice inspection requirements will be performed.

11. Investigate the influence of possible cracking in the hoop direction on the dome tendon conduits.

Answer: Not applicable under new repair sequence.

TABLE 1 FOR ANSWER TO QUESTION 5.3(6b)

PREDICTED STRAINS AND DISPLACEMENTS
15% PRESTRESS

Gage No.	Type of Measurement	Predicted Measurement	Range
E22	Liner Rad. Displace.	0.017 in	±0.004 in
E23	Liner Rad. Displace.	0.017 in	±0.004 in
E24	Liner Rad. Displace.	0.017 in	±0.004 in
E25	Liner Rad. Displace.	0.008 in	±0.002 in
E26	Liner Rad. Displace.	0.008 in	±0.002 in
E27	Liner Rad. Displace.	0.008 in	±0.002 in
E28AV	Liner Vert. Displace.	0.041 in	±0.01 in
E28BV	Liner Vert. Displace.	0.016 in	±0.004 in
E29AV	Liner Vert. Displace.	0.041 in	±0.01 in
E29BV	Liner Vert. Displace.	0.016 in	±0.004 in
E30AV	Liner Vert. Displace.	0.041 in	±0.01 in
E30BV	Liner Vert. Displace.	0.016 in	±0.004 in
	Apex Vert. Displace.	0.129 in	±0.032 in
	15' Radius Vert. Displace.	0.129 in	±0.032 in
	30' Radius Vert. Displace.	0.120 in	±0.03 in
	45' Radius Vert. Displace.	0.069 in	±0.017 in
R118M	Liner Merid. Strain	65 μ in/in	± 33 μ in/in
R118D	Liner Diag. Strain	-	-
R118H	Liner Hoop Strain	33 μ in/in	± 16 μ in/in
R119M	Liner Merid. Strain	65 μ in/in	± 32 μ in/in
R119D	Liner Diag. Strain	-	-
R119H	Liner Hoop Strain	67 μ in/in	± 33 μ in/in
R120M	Liner Merid. Strain	65 μ in/in	± 33 μ in/in
R120D	Liner Diag. Strain	-	-
R120H	Liner Hoop Strain	33 μ in/in	± 16 μ in/in
R121M	Liner Merid. Strain	65 μ in/in	± 32 μ in/in
R121D	Liner Diag. Strain	-	-
R121H	Liner Hoop Strain	67 μ in/in	± 33 μ in/in
R122M	Liner Merid. Strain	73 μ in/in	± 37 μ in/in
R122D	Liner Diag. Strain	-	-
R122H	Liner Hoop Strain	74 μ in/in	± 37 μ in/in
R123M	Liner Merid. Strain	65 μ in/in	± 33 μ in/in
R123D	Liner Diag. Strain	-	-
R123H	Liner Hoop Strain	33 μ in/in	± 16 μ in/in
R124M	Liner Merid. Strain	73 μ in/in	± 37 μ in/in
R124D	Liner Diag. Strain	-	-
R124H	Liner Hoop Strain	74 μ in/in	± 37 μ in/in
R125M	Liner Merid. Strain	65 μ in/in	± 33 μ in/in
R125D	Liner Diag. Strain	-	-
R125H	Liner Hoop Strain	33 μ in/in	± 16 μ in/in