15 pages

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

CALCULATION TITLE PAGE

Client: Progress Energy	Page 1 of 12 plus Attachment
Project: CR3 Containment Calculations	Task No. 0102-0906-0135
Title: Concrete Modulus of Elasticity and Specified Compressive Strength	Calculation No. 0102-0135-02

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
J. L. Hibbard J. L. Hibbard 1-16-2010	CWBagley Chris Bagley 1-16-2010	Patrick Butler P. Butler 1-16-2010	0

QUALITY ASSURANCE DOCUMENT

This document has been prepared, checked, and reviewed/approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B, as specified in the MPR Quality Assurance Manual.

P/79

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0102-0135-02	J. L. Hibbard	C W Bagley	

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

Table of Contents

1.0 PURPOSE

This calculation determines the concrete elastic modulus and the concrete specified compressive strength for original concrete and for new concrete for the Steam Generator Replacement construction opening plug and containment repair for Crystal River Unit 3.

2.0 SUMMARY

The elastic modulus and the specified concrete compressive strength for the new and existing concrete for maintenance conditions, design basis return to service conditions, and design basis end of life conditions are summarized in Table T_s.

$$T_s = \begin{bmatrix} \text{"Concrete"} & \text{"Applicable"} & \text{"Elastic"} & \text{"Specified Comp."} \\ \text{""} & \text{"Conditions"} & \text{"Modulus"} & \text{"Strength for"} \\ \text{""} & \text{""} & \text{""} & \text{"Allowable"} \\ \text{""} & \text{""} & \text{"psi * E06"} & \text{"psi"} \\ \text{"Original"} & \text{"Maint. / Repair"} & 4.03 & 6720 \\ \text{"Original"} & \begin{pmatrix} \text{"Design Basis Return to Service"} \\ \text{"Design Basis End of Life"} \end{pmatrix} & 4.03 & 5000 \\ \text{"New"} & \text{"Maint. / Repair"} & 5.12 & 6000 \\ \text{"New"} & \begin{pmatrix} \text{"Design Basis Return to Service"} \\ \text{"Design Basis End of Life"} \end{pmatrix} & 5.12 & 5000 \end{bmatrix}$$

Notes:

1. 6000 psi is the 5-day specified compressive strength of the new concrete.
2. 5000 psi is the specified compressive strength of the containment concrete in the FSAR. 7000 psi is the 28 day specified compressive strength of the new concrete. 7000 psi can be used instead of 5000 psi for new concrete if the FSAR is revised.
3. This note applies to the column titled, "Elastic Modulus." The elastic modulus is for analytical use. The concrete compressive strength (psi) used for the calculation of the elastic modulus is:

$$n_3 = \begin{pmatrix} \text{"Original"} & 5000 \\ \text{"New"} & 7000 \end{pmatrix} .$$

3.0 BACKGROUND

A project is underway at Progress Energy's Crystal River Unit 3 site to replace the steam generators. As part of that project, an opening has been cut into the concrete containment above the equipment hatch. As this opening was being cut, cracking in the concrete containment wall was identified. The crack is around the full periphery of the opening and is in the plane of the wall. The cracking is located at the radius of the circumferential tensioning tendons, and is indicative of a delaminated condition.

4.0 ASSUMPTIONS

4.1 Unverified Assumptions

None.

4.2 Other Assumptions

None.

5.0 APPROACH

The concrete modulus of elasticity is calculated with the correlation provided in ACI 318-63 (Reference 1.1, Sections 301 and 1102). ACI 318-63 is the design code for the Crystal River Unit 3 containment (Reference 13, Section 5.2.3.1).

$$E_c = 33 \cdot \rho_c^{1.5} \cdot \sqrt{f_c}$$

where	E_c	=	static modulus of elasticity of concrete, psi
	ρ_c	=	density of concrete, lb/ft^3
	f_c	=	specified compressive strength of concrete, psi

The source of the correlation in ACI 318-63 is a paper by Pauw (Reference 5, p. 686 and Reference 1.2, Section 8.5). The correlation is based on a best fit to experimental data as shown in the following figure from Pauw's paper, Reference 5, Figure 2.

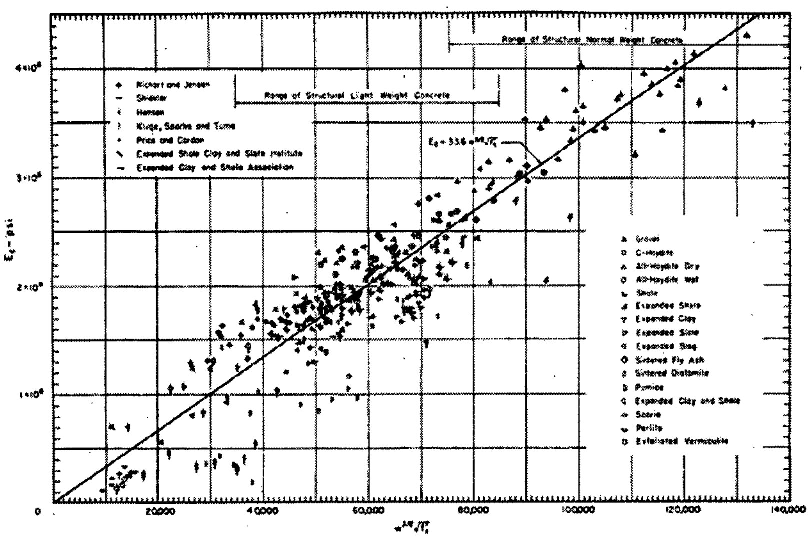


Fig. 2—Correlation of test data

The Pauw correlation was based on lower strength concretes than are used today. The suitability of the ACI 318-63/Pauw correlation for high strength concretes is established in Reference 9, Conclusions Section, Reference 10, Figure 1, Reference 11, Figure 1 and Table 9, and Reference 12, Conclusion 3.

The concrete strength parameter in ACI 318-63 is fc`, the specified compressive strength (Reference 1.1, Sections 1102 and 301). The concrete strength parameter in the Pauw correlation is the concrete strength at the time of the test (Reference 5, p. 681). The effect of this difference in definition of concrete strength on the calculated modulus of elasticity is evaluated in Section 6.2.

6.0 CALCULATION

6.1 Design Inputs

$$f_{c'.orig} := \begin{pmatrix} 5000 \\ 6720 \end{pmatrix} \cdot psi \qquad case1 := \begin{pmatrix} \text{"Specified \& Design Basis"} \\ \text{"5-year"} \end{pmatrix}$$

Original concrete compressive strength.
-Ref. 2, p. 2
-Ref. 3, Results Summary, Class 5000 concrete

$$f_{c'.new} \equiv \begin{pmatrix} 5000 \\ 6000 \\ 7000 \end{pmatrix} \cdot psi \qquad case2 := \begin{pmatrix} \text{"Design Basis"} \\ \text{"5-day"} \\ \text{"Specified"} \end{pmatrix}$$

New concrete compressive strength.
-Ref. 2, p. 2
-Ref. 6 and Ref. 7, Table 1
-Ref. 6 and Ref. 7, Table 1

$$\rho_c \equiv \begin{pmatrix} 144 \\ 151 \end{pmatrix} \cdot \frac{lb}{ft^3} \qquad case3 := \begin{pmatrix} \text{"Original"} \\ \text{"New"} \end{pmatrix}$$

Concrete density
-Ref. 4
-Ref. 6 and Ref. 8, p. 6; Ref. 8 provides the theoretical density and measured density for two mixes, Options 1A and 2A. A density of 151 lb/ft^3 is representative of the theoretical and measured densities of the two mixes.

core :=

"core 16-1"	$3.75 \cdot 10^6$
"core 16-2"	$4.05 \cdot 10^6$
"core 40-1"	$3.15 \cdot 10^6$
"core 40-3"	$2.95 \cdot 10^6$
"core 65-2"	$2.7 \cdot 10^6$
"core 66-2"	$3.1 \cdot 10^6$
"core 63-2"	$3.3 \cdot 10^6$
"core 59"	$3.35 \cdot 10^6$

Measured modulus of elasticity from CR3 concrete cores
-Ref. 14 for all cores but Core 59
-Ref. 15 for Core 59

$$E_{c.meas} := core^{\langle 2 \rangle} \cdot psi$$

6.2 Modulus of Elasticity

Original Concrete

The modulus of elasticity for the original concrete is determined based on the core measurements, and is also calculated for the specified compressive strength ($f_{c'.orig_1} = 5000\,psi$) and 5-year compressive strength ($f_{c'.orig_2} = 6720\,psi$). A comparison of the results and selection of the concrete modulus is at the end of the section.

The average modulus of elasticity for the original concrete from measurements of cores taken from the CR3 containment is:

$$E_{c.avg.m} := mean(E_{c.meas}) \qquad E_{c.avg.m} = 3.29 \times 10^{6}\,psi$$

where $$E_{c.meas} = \begin{pmatrix} 3.75 \\ 4.05 \\ 3.15 \\ 2.95 \\ 2.7 \\ 3.1 \\ 3.3 \\ 3.35 \end{pmatrix} \cdot 10^{6} \cdot psi$$

The calculated modulus of elasticities for the specified compressive strength and the 5-year compressive strength are:

$$E_{c.orig_i} := 33 \cdot psi \cdot \left(\frac{\rho_{c_1}}{lb \div ft^3} \right)^{1.5} \cdot \sqrt{\frac{f_{c'.orig_i}}{psi}}$$

$$E_{c.orig} = \begin{pmatrix} 4.03 \times 10^{6} \\ 4.67 \times 10^{6} \end{pmatrix} psi \qquad case1 = \begin{pmatrix} \text{"Specified \& Design Basis"} \\ \text{"5-year"} \end{pmatrix}$$

where $$\rho_{c_1} = 144 \frac{lb}{ft^3} \qquad f_{c'.orig} = \begin{pmatrix} 5000 \\ 6720 \end{pmatrix} psi \qquad case1 = \begin{pmatrix} \text{"Specified \& Design Basis"} \\ \text{"5-year"} \end{pmatrix}$$

The above results show that the elastic modulus ranges from a low of $E_{c.avg.m} = 3.29 \times 10^{6}\,psi$ to a high of $E_{c.orig_2} = 4.67 \times 10^{6}\,psi$ based on the 5-year compressive strength. It is concluded that the modulus of elasticity based on the specified compressive strength best represents this range. This calculated modulus is consistent with ACI 318-63, the design basis for the CR3 containment. The elastic modulus for the original concrete is:

$$E_{c.orig_1} = 4.03 \times 10^{6}\,psi$$

This elastic modulus is for the original concrete from the current time to the end of plant life.

New Concrete

The concrete modulus of elasticity is calculated with the ACI 318-63 correlation in Reference 1.1, Section 1102.

$$E_{c.new} := 33 \cdot psi \cdot \left(\frac{\rho_{c_2}}{lb \div ft^3} \right)^{1.5} \cdot \sqrt{\frac{f_{c'.new_3}}{psi}}$$

$$E_{c.new} = 5.12 \times 10^{6}\,psi$$

where $\rho_{c_2} = 151 \frac{lb}{ft^3}$ $\quad f_{c'.new_3} = 7000\,psi$

This elastic modulus is for the new concrete from the time the concrete reaches at least its 5-day strength of 6000 psi to the end of plant life. Use of a single modulus for this time period is justified based on the scatter in results for the elastic modulus correlation shown in the figure in Section 5.0.

7.0 REFERENCES

1. American Concrete Institute, "Building Code Requirements for Reinforced Concrete."

1.1 ACI 318-63
1.2 ACI 318-05

2. Progress Energy, "Design Basis Document for the Containment," Revision 6.

3. Florida Power Corporation Document Identification No. S-00-0047, As-built Concrete Strength for Class 1 Structures, Revision 0.

4. Email from Mr. J. Holliday (PE) to Mr. K. Gantz (MPR), 12-30-2009, 10:35 AM, Subject: Concrete Density.

5. A. Pauw, "Static Modulus of Elasticity of Concrete as Affected by Density," Journal of the American Concrete Institute, Vol. 57, 1960, pp. 679–687.

6. Email from Mr. J. Holliday (PE) to Mr. J. Hibbard (MPR), 1-7-2010, 3:42 PM, Subject: Comments Calculation 0102-0135-02.

7. Progress Energy Specification CR3-C-0003, "Specification for Concrete Work for Restoration of the SGR Opening in the Containment Shell," Revision 0.

8. S&ME Phase II Test Report Trial Mixture Testing for Crystal River Unit 3 Steam Generator Replacement Project," S&ME Project No. 1439-08-208, January 13, 2009.

9. F. Oluokun, E. Burdette, and J. Deatherage, "Elastic Modulus, Poisson's Ratio and Compressive Strength Relationships at Early Ages," ACI Materials Journal, Jan.-Feb. 1991, pp. 3-10.

10. T. Shih, G. Lee, K. Chang, "On Static Modulus of Elasticity of Normal-weight Concrete," Journal of Structural Engineering, Vol. 115, No. 10, October 1989, pp. 2579-2587.

11. P. Gardoni, D. Trejo, M. Vannucci, and C. Bhattacharjee, "Probabilistic Models for Modulus of Elasticity of Self-Consolidated Concrete: Bayesian Approach," Journal of Engineering Mechanics, April 2009, pp. 295-306.

12. G. Washa, J. Saemann, and S. Cramer, "Fifty-year Properties of Concrete made in 1937," ACI Materials Journal, July-August, 1989, pp. 367-371.

13. Progress Energy Final Safety Analysis Report (FSAR), Containment System & Other Special Structures, Chapter 5, Revision 31.3.

14. S&ME Document Transmittal No. 09-208-03, S&ME Project No. 1439-08-208, November 16, 2009.

15. S&ME Document Transmittal No. 09-208-05, S&ME Project No. 1439-08-208, November 24, 2009.

Attachment

The attachments are:

- Email from Mr. J. Holliday (PE) to Mr. K. Gantz (MPR), 12-30-2009, 10:35 AM, Subject: Concrete Density.
- Email from Mr. J. Holliday (PE) to Mr. J. Hibbard (MPR), 1-7-2010, 3:42 PM, Subject: Comments Calculation 0102-0135-02.

Hibbard, Jim

From: Holliday, John [John.Holliday@pgnmail.com]
Sent: Wednesday, December 30, 2009 10:35 AM
To: Gantz, Kevin; Knott, Ronald
Cc: Hibbard, Jim; Dyksterhouse, Don
Subject: RE: Concrete Density

Kevin,
The reference will be EC 75218, RB Delamination Repair Phase 2- Detensioning

The unit weight is 144 lbs cu ft.

From: Gantz, Kevin [mailto:kgantz@mpr.com]
Sent: Wednesday, December 30, 2009 10:01 AM
To: Knott, Ronald; Holliday, John
Cc: Hibbard, Jim
Subject: RE: Concrete Density

John and Ron,

I don't think there was ever a follow-up sent to this email. Could you provide us with the reference. I did not see it in S00-0047.

Kevin

-----Original Message-----
From: Knott, Ronald [mailto:Ronald.Knott@pgnmail.com]
Sent: Wednesday, December 16, 2009 10:15 AM
To: Holliday, John
Cc: Gantz, Kevin
Subject: FW: Concrete Density

John,
Can you direct Kevin to the density reference. I don't know where the original data came from for density. I was only quoting what I heard in the meeting. I assumed it was in the S00-0047 attachments.

From: Gantz, Kevin [mailto:kgantz@mpr.com]
Sent: Tuesday, December 15, 2009 6:22 PM
To: Knott, Ronald
Cc: Dyksterhouse, Don; Holliday, John; Bird, Edward; Butler, Patrick
Subject: Concrete Density

Ron,

During our previous meeting you received some original information on the concrete density. I remember you saying later that the concrete density was 144 or 145 pcf. Do you have a reference or an actual number so that I can make sure I have the correct modulus calculated?

Thanks,

Kevin

Hibbard, Jim

From: Holliday, John [John.Holliday@pgnmail.com]
Sent: Thursday, January 07, 2010 3:42 PM
To: Hibbard, Jim
Cc: Dyksterhouse, Don; Knott, Ronald
Subject: RE: comments calculation 0102-0135-02
Attachments: Z25R5 Concrete spec CR3-C-0003.pdf; Z43R3 Phase II Test Plan.pdf; Z44R3 Phase II Test Report.pdf

Jim,
The following inputs are approved by Progress Energy as being acceptable for use by MPR:

The 5 and 28 day minimum concrete compressive strengths for the new concrete for the SGR access opening and repair of the delamination are 6000 and 7000 psi respectively. This requirement for the new concrete is contained in Attachment 1 of specification CR3-C-0003 and in S&MEs phase II Test Plan. Additionally, the theoretical unit weight of the new concrete is 151 pcf as reported in the S&ME Phase II Test Report.

Regards,

John Holliday

From: Hibbard, Jim [mailto:jhibbard@mpr.com]
Sent: Thursday, January 07, 2010 2:47 PM
To: Holliday, John
Subject: comments

John,

Could you give me a call to discuss your comments on the -02 calc? At present I do not have your number, although I may get it from Ed or Patrick.

Jim

51 pages

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

CALCULATION TITLE PAGE

Client: Progress Energy	Page 1 of 47 (+ Att. A)
Project: CR3 Containment Calculations	Task No. 0102-0906-0135
Title: Tendon Tension Calculation	Calculation No. 0102-0135-03

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
Kevin Gantz 1/18/2010 Kevin Gantz	A. Trif 1/18/2010 Adrian Trif	J L Hibbard 1/18/2010 Jim Hibbard	0

QUALITY ASSURANCE DOCUMENT

This document has been prepared, checked, and reviewed/approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B, as specified in the MPR Quality Assurance Manual.

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0102-0135-03	Kevin Gtz	[illegible]	

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

Table of Contents

1.0 PURPOSE

This calculation determines the dome, vertical, and horizontal tendon tension immediately following the Steam Generator Replacement (SGR) Project completion (33 years) and at end of plant life (60 years) in the Crystal River Unit 3 containment. The values of tendon tension calculated herein will be used in structural analyses of the containment for ages 33 and 60 years after the Structural Integrity Test (SIT).

2.0 SUMMARY

Average dome, vertical, and horizontal tendon losses from the following four mechanisms were calculated:

- Elastic Shortening
- Concrete Shrinkage
- Tendon Steel Relaxation
- Concrete Creep

The above mechanisms are described in Reference 22. Tendon losses were calculated individually for different groups. For the dome tendons, the tension in all tendons is not modified during the SGR project. For the vertical tendons, some of the tendons are detensioned and subsequently retensioned, and some of the tendons are not modified at all. Losses are calculated separately for these two groups. For the horizontal tendons, several tendons are detensioned and subsequently retensioned and other tendons are not modified at all. For the detensioned and retensioned tendons, several tendons pass through replacement concrete that fills the SGR opening plug and replaces the delaminated concrete, and others do not pass through the replacement concrete. Tendon losses are calculated individually for these two groups of detensioned and retensioned tendons as well as the tendons that are not modified during the SGR project.

Concrete shrinkage and concrete creep are dependent on the material properties of the concrete that the tendons pass through. By calculating tendon tension losses separately depending on the tendon location (as explained above), the effects of local concrete material differences are accounted for. However, for tendons that are detensioned and subsequently retensioned that pass through or near the repaired SGR opening, the tendon losses are calculated as if the tendon passes directly through the repaired SGR opening. Tendon steel relaxation losses are not dependent on the tendon location, and they are treated the same for all tendons. Elastic shortening losses are unique to each tendon based on the sequence with which the tendons are tensioned. An average elastic shortening loss is calculated based on tendon orientation (dome, vertical, or horizontal) so that every tendon does not have to be tensioned individually in the containment structural analyses.

The tension in each group of tendons is reported as the average tension along the tendon length.

The tendon tension at the end of the SGR Project (33 years):

Dome Tendons:

All Dome Tendons: $Tension_{d.33} = 1376\,kip$

Vertical Tendons:

Detensioned and Retensioned Tendons: $Tension_{v.33.mod} = 1603\,kip$

Unadjusted Tendons: $Tension_{v.33.unmod} = 1474\,kip$

Horizontal Tendons:

Detensioned and Retensioned Tendons Passing through SGR Opening Bay: $Tension_{h.33.mod.SGR} = 1573\,kip$

Detensioned and Retensioned Tendons not Passing through SGR Opening Bay: $Tension_{h.33.mod} = 1573\,kip$

Unadjusted Tendons: $Tension_{h.33.unmod} = 1398\,kip$

The tendon tension at the 60 year end of life:

Dome Tendons:

All Dome Tendons: $Tension_{d.60} = 1353\,kip$

Vertical Tendons:

Detensioned and Retensioned Tendons: $Tension_{v.60.mod} = 1539\,kip$

Unadjusted Tendons: $Tension_{v.60.unmod} = 1464\,kip$

Horizontal Tendons:

Detensioned and Retensioned Tendons Passing through SGR Opening Bay: $Tension_{h.60.mod.SGR} = 1498\,kip$

Detensioned and Retensioned Tendons not Passing through SGR Opening Bay: $Tension_{h.60.mod} = 1508\,kip$

Unadjusted Tendons: $Tension_{h.60.unmod} = 1380\,kip$

3.0 ASSUMPTIONS

3.1 Unverified Assumptions

There are no unverified assumptions.

3.2 Verified Assumptions

1. The thickness of the concrete replacing the delaminated concrete is approximately 10 inches, the width spans the entire span between buttresses 3 and 4, and the height spans between the top of the equipment hatch to approximately 10 feet below the bottom of the ring girder. These dimensions are consistent with the measured extents of the delamination with only the tendons that pass through the Steam Generator Replacement (SGR) opening detensioned (see Figure 1).

2. The end of plant life is assumed to be 60 years after the containment Structural Integrity Test (SIT) in November 1976 (Reference 7, page 10). This assumption has been confirmed by Progress Energy (see Lead Reviewer comments to this calculation).

3. The replaced concrete in the patch and the outer portion of the delamination will not be prestressed until 5 days after pouring. This assumption has been confirmed by Progress Energy (see Lead Reviewer comments to this calculation).

4. The concrete that is used to plug the SGR opening and replace the outer portion of the delamination will have improved shrinkage properties (less shrinkage) compared to the existing concrete when it was first placed. This assumption has been confirmed by Progress Energy (see Lead Reviewer comments to this calculation).

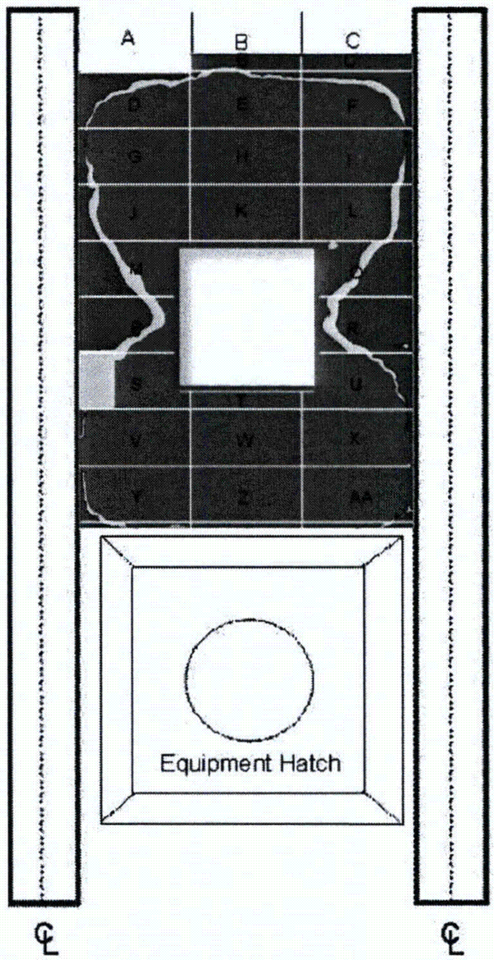


Figure 1. Delamination Boundary (Delamination shown in red)

4.0 Methodology

The dome, vertical, and horizontal tendon losses are determined by considering losses from four different mechanisms:

- Elastic Shortening - Shortening of concrete as prestress is applied
- Concrete Shrinkage - Decrease in concrete volume
- Steel Relaxation - Stress relaxation in the prestressing steel
- Concrete Creep - Strain of the concrete over time due to sustained loads

Each loss has been determined at 40 years after the Structural Integrity Test (SIT) in various Crystal River Unit 3 calculations (References 2, 3, and 7). These losses are used as a basis for determining the losses at the end of steam generator replacement and at 60 years after SIT. The methodology for this calculation is similar to that of Progress Calculation S08-0008 (Reference 14).

Calculation of the increase in tendon tension during an accident which increases containment pressure is not included in this calculation.

5.0 CALCULATION

5.1 Data

$A_t := 9.723in^2$		Total cross section area of 163 wires in a single tendon; Ref. 1, page 6.
$E_c := 4.03 \times 10^6 psi$		Elastic modulus of existing concrete; Ref. 16, page 4.
$E_s := 29 \times 10^6 psi$		Elastic modulus of steel; Ref. 4, Table 38.
$h_{open} := 27ft$		Height of SGR opening; Ref. 5.
$El_{top.SGR} := 210ft$		Elevation of the top of the SGR opening; Ref. 5.
$w_{open} := 25ft$		Width of SGR opening; Ref. 5.
$t_{delam} := 10in$		Approximate thickness of the delaminated concrete; Assumption 3.2.1.
$El_{top.eq.hatch} := 157ft + 10in$	$El_{top.eq.hatch} = 157.83ft$	Elevation of the concrete at the transition to 3'-6" wall thickness above the equipment hatch; Ref. 9.
$El_{bot.ring.girder} := 250ft$		Elevation of the bottom of the ring girder; Ref. 9.
$El_{top.ring.girder} := 267.5ft$		Elevation of the top of the ring girder; Ref. 9.
$L_{delam.ring.girder} := 10ft$		Approximate distance from the bottom of the ring girder to the top of the delamination boundary; Assumption 3.2.1.
$El_{bot.containment} := 80.5ft$		Elevation of the bottom of containment; Ref. 15.
$El_{top.basemat} := 93.0ft$		Elevation of the top of the containment basemat; Ref. 9.
$\alpha_{buttress} := 60deg$		Radial angle between adjacent buttresses; Ref. 9.
$w_{buttress} := 12ft + 4.125in$	$w_{buttress} = 12.34ft$	Average buttress width; Ref. 9.
$t_{buttress} := 2ft + 4.5in$		Buttress thickness increase beyond containment wall thickness; Ref. 9.
$R_{liner} := 65ft$		Radial distance to liner inside surface; Ref. 9

$t_{liner} := 0.375in$	Nominal liner thickness throughout most of the containment; Ref. 9.
$t_{wall} := 3.5ft$	Wall thickness between buttresses (undelaminated); Ref. 9.
$Stress_{d.creep.40} := 13.85ksi$	Loss in dome tendon stress due to creep at 40 years life; Ref. 2, page 4.
$Stress_{d.shrink.40} := 2.90ksi$	Loss in dome tendon stress due to concrete shrinkage at 40 years life; Ref. 2, page 4.
$Stress_{d.eshort.40} := 5.50ksi$	Loss in dome tendon stress due to elastic shortening at 40 years life; Ref. 2, page 4.
$Force_{d.relax.40} := 48.5kip$	Loss in dome tendon force due to steel relaxation at 40 years life; Ref. 7, Att. F, page F2.
$Force_{d.relax.35} := 48.2kip$	Loss in dome tendon force due to steel relaxation at 35 years life; Ref. 7, Att. F, page F2.
$\sigma_{d.axial} := 1530psi$	Average concrete compressive prestress in dome, in direction of tendon length; Ref. 3, page 49. As a check of this value from Ref. 3 a scoping comparison was made to finite element analysis results for the CR3 containment. It was concluded that this is an appropriate stress for this calculation.
$Creep_{d.basic.60} := 0.35 \times 10^{-6} \cdot \frac{1}{psi}$	Basic creep for dome tendon loading beginning 180 days after pouring, at 60 years life; Ref. 7, Att. G, page G5.
$Creep_{v.basic.60} := 0.25 \times 10^{-6} \cdot \frac{1}{psi}$	Basic creep for vertical tendon loading beginning 834 days after pouring, at 60 years life; Ref. 7, Att. G, page G5.
$Creep_{h.basic.60} := 0.24 \times 10^{-6} \cdot \frac{1}{psi}$	Basic creep for horizontal tendon loading beginning 964 days after pouring, at 60 years life; Ref. 7, Att. G, page G5.
$Creep_{d.basic.33} := 0.30 \times 10^{-6} \cdot \frac{1}{psi}$	Basic creep for dome tendon loading beginning 180 days after pouring, at 33 years life; Ref. 7, Att. G, page G5.
$Creep_{v.basic.33} := 0.215 \times 10^{-6} \cdot \frac{1}{psi}$	Basic creep for vertical tendon loading beginning 834 days after pouring, at 33 years life; Ref. 7, Att. G, page G5.

$Creep_{h.basic.33} := 0.205 \times 10^{-6} \cdot \frac{1}{psi}$		Basic creep for horizontal tendon loading beginning 964 days after pouring, at 33 years life; Ref. 7, Att. G, page G5.
$Stress_{v.shrink.40} := 2.90ksi$		Loss in vertical tendon stress due to concrete shrinkage at 40 years life; Ref. 2, page 2.
$Force_{v.relax.40} := 48.5kip$		Loss in vertical tendon force due to steel relaxation at 40 years life; Ref. 7, Att. F, page F2..
$Force_{v.relax.35} := 48.2kip$		Loss in vertical tendon force due to steel relaxation at 35 years life; Ref. 7, Att. F, page F2..
$v_{u.patch} := 1.14$		Ultimate creep coefficient for concrete in plug; Ref. 6.
$Stress_{h.shrink.40} := 2.90ksi$		Loss in horizontal tendon stress due to concrete shrinkage at 40 years life; Ref. 2, page 3.
$Force_{h.relax.40} := 48.2kip$		Loss in horizontal tendon force due to steel relaxation at 40 years life; Ref. 7, Att. F, page F2..
$Force_{h.relax.35} := 47.9kip$		Loss in horizontal tendon force due to steel relaxation at 35 years life; Ref. 7, Att. F, page F2..
$GUTS_{70} := 1635kip$		Tendon lock off tension, equal to 70% of the Guaranteed Ultimate Tensile Strength (GUTS) per tendon; Ref. 1, page 14.
$n_{v.tendon} := 144$		Total number of vertical tendons; Ref. 1, page 14.
$Age_{outage} := 33yr$	$Age_{outage} = 12053\,day$	Age of original concrete at SGR outage, starting from the date of containment Structural Integrity Test; Ref. 7, page 10 and Ref. 8, page 7.
$Age_{eol} := 60yr$	$Age_{eol} = 21915\,day$	Age of original concrete at end of plant life, starting from date of containment Structural Integrity Test; Assumption 3.2.2
$\lambda := 75$		Relative humidity for the containment outside environment, in percent; Reference 17.

$Force_{h.relax.30} := 47.6kip$	Horizontal tendon steel relaxation load at 30 years; Ref. 7, Att. F, page F2.
$Force_{v.relax.30} := 47.8kip$	Vertical tendon steel relaxation load at 30 years; Ref. 7, Att. F, page F2.
$Force_{d.relax.33} := 48.0kip$	Dome tendon steel relaxation load at 33 years; logarithmically interpolated from Ref. 7, Att. F, page F2.
$Force_{h.relax.33.unmod} := 47.8kip$	Horizontal tendon steel relaxation load at 33 years; logarithmically interpolated from Ref. 7, Att. F, page F2.
$Force_{v.relax.33.unmod} := 48.0kip$	Vertical tendon steel relaxation load at 30 years; logarithmically interpolated from Ref. 7, Att. F, page F2.
$El_{avg.tend.space.bot} := 183ft + 10.75in$	Bottom horizontal tendon elevation used to calculate average horizontal tendon spacing; Ref. 20.
$El_{avg.tend.space.top} := 212ft + 8.25in$	Top horizontal tendon elevation used to calculate average horizontal tendon spacing; Ref. 20.
$n_{avg.tend.space} := 19$	Number of tendons spanning between $El_{avg.tend.space.bot}$ and $El_{avg.tend.space.top}$, inclusive; Refs. 20 and 21.
$r_{h.tendon} := 67ft + 8.625in$	Horizontal tendon placement radius; Ref. 24.
$n_{buttress} := 6$	Number of buttresses in the containment; Ref. 9.

5.2 Dome Tendons - 60 Years After Initial SIT

The dome tendons will not be detensioned or retensioned during the Steam Generator Replacement (SGR) outage. The tendon tension at 60 years after the Structural Integrity Test (SIT) of November 1976 (Reference 7, page 10) is determined by scaling the predicted tension at 40 years after SIT. The individual losses in the dome tendons at 40 years after SIT from creep, steel stress relaxation, elastic shortening, and concrete shrinkage are as follows (see Section 5.1 for references):

$Stress_{d.creep.40} = 13850\,psi$

$Stress_{d.eshort.40} = 5500\,psi$

$Stress_{d.shrink.40} = 2900\,psi$

$Stress_{d.relax.40} := \dfrac{Force_{d.relax.40}}{A_t}$ $\qquad Stress_{d.relax.40} = 4988\,psi$

where

$Force_{d.relax.40} = 48.5\,kip$ $\qquad A_t = 9.723\,in^2$

Elastic Shortening

The dome tension losses due to elastic shortening do not change over time. The elastic shortening losses at 60 years after SIT are:

$Stress_{d.eshort.60} := Stress_{d.eshort.40}$ $\qquad Stress_{d.eshort.60} = 5500\,psi$

Concrete Shrinkage

Industry experience shows that the majority of concrete shrinkage occurs in the early life of the containment. Since the containment was constructed over 30 years ago, nearly all of the shrinkage has already taken place. At this point, shrinkage is essentially time-independent, and the concrete shrinkage at 60 years will be approximately equal to the concrete shrinkage predicted at 40 years.

$Stress_{d.shrink.60} := Stress_{d.shrink.40}$ $\qquad Stress_{d.shrink.60} = 2900\,psi$

Steel Relaxation

The steel relaxation losses at 40 years have been calculated previously (Reference 7, Att. F, Page F2). Based on Figure 5-26 of Reference 10, steel relaxation is linear with time on a logarithmic scale. The losses calculated at 40 years will be extrapolated to 60 years based on the last two data points from Reference 7, Att. F, page F2.

$$Force_{d.relax.60} := \frac{Force_{d.relax.40} - Force_{d.relax.35}}{\log(40) - \log(35)} \cdot (\log(60) - \log(35)) + Force_{d.relax.35}$$

$Force_{d.relax.60} = 49.4\,kip$

$Stress_{d.relax.60} := \dfrac{Force_{d.relax.60}}{A_t}$ $\qquad$ $Stress_{d.relax.60} = 5082\,psi$

where

$Force_{d.relax.40} = 48.5\,kip$ $\qquad$ $Force_{d.relax.35} = 48.2\,kip$ $\qquad$ $A_t = 9.723\,in^2$

Creep

The basic creep determined from testing extrapolated to 60 years is (see Section 5.1 for reference):

$Creep_{d.basic.60} = 3.5 \times 10^{-7} \dfrac{1}{psi}$

The average prestress in the dome in the axial direction of the tendons is (see Section 5.1 for reference):

$\sigma_{d.axial} = 1530\,psi$

The reasonableness of this value has been confirmed using finite element analysis.

The tendon prestress lost due to creep is calculated based on Page 4 of Reference 2:

$Stress_{d.creep.60} := \sigma_{d.axial} \cdot Creep_{d.basic.60} \cdot E_s$ $\qquad$ $Stress_{d.creep.60} = 15529\,psi$

where E_s is the steel elastic modulus and is equal to:

$E_s = 2.9 \times 10^{7}\,psi$

Total 60 Year Loss

The total tendon stress loss after 60 years is:

$Stress_{d.total.60} := Stress_{d.eshort.60} + Stress_{d.shrink.60} + Stress_{d.relax.60} + Stress_{d.creep.60}$

$Stress_{d.total.60} = 29011\,psi$

Converting the stress lost into a force per tendon that is lost:

$Force_{d.total.60} := Stress_{d.total.60} \cdot A_t$ $\qquad Force_{d.total.60} = 282.1\,kip$

where

$A_t = 9.723\,in^2$

The design tension per tendon, excluding losses, is (see Section 5.1 for reference):

$Force_{design} := GUTS_{70}$ $\qquad Force_{design} = 1635\,kip$

The remaining tension in the dome tendons at 60 years is:

$Tension_{d.60} := Force_{design} - Force_{d.total.60}$ $\qquad Tension_{d.60} = 1352.9\,kip$

5.3 Vertical Tendons - 60 Years After Initial SIT

Some of the vertical tendons near the SGR opening will be detensioned and retensioned during SGR while the tension in some other tendons will not change at all. The tendon tension at 60 years will be calculated for each of the two sets of tendons individually. When calculating tendon losses, all of the vertical tendons that are detensioned and retensioned will be considered to pass directly through the SGR opening since cutting and repairing the opening will affect the region both inside and around the SGR opening. The tendon tension at 60 years after the Structural Integrity Test (SIT) of November 1976 (Reference 7, page 10) is determined by scaling the predicted tension at 40 years after SIT. The individual losses in the vertical tendons at 40 years after SIT from steel stress relaxation and concrete shrinkage are as follows (see Section 5.1 for references):

$Stress_{v.shrink.40} = 2900\,psi$

$Stress_{v.relax.40} := \dfrac{Force_{v.relax.40}}{A_t}$ $\qquad Stress_{v.relax.40} = 4988\,psi$

where

$A_t = 9.723\,in^2$ $\qquad Force_{v.relax.40} = 48.5\,kip$

Elastic Shortening

The total vertical force in the containment due to the vertical tendons tensioned to lock off load is:

$Force_{v.axial} := n_{v.tendon} \cdot GUTS_{70}$ $\qquad Force_{v.axial} = 235440\,kip$

where

$n_{v.tendon} = 144$ $\qquad GUTS_{70} = 1635\,kip$

The horizontal cross sectional area of concrete at approximately the mid height of the containment is:

$$A_{v.contain} := \pi \cdot \left[\left(R_{liner} + t_{liner} + t_{wall}\right)^2 - \left(R_{liner} + t_{liner}\right)^2 \right] + n_{buttress} \cdot w_{buttress} \cdot t_{buttress}$$

$A_{v.contain} = 236807.3\,in^2$

where

$R_{liner} = 65\,ft$ $\qquad t_{liner} = 0.375\,in$ $\qquad t_{wall} = 3.5\,ft$

$w_{buttress} = 12.34\,ft$ $\qquad t_{buttress} = 2.38\,ft$ $\qquad n_{buttress} = 6$

The horizontal cross sectional area of the liner at approximately the mid height of the containment is:

$$A_{v.liner} := \pi \cdot \left[\left(R_{liner} + t_{liner}\right)^2 - R_{liner}^{\ 2}\right]$$

$$A_{v.liner} = 1838.3\,in^2$$

The average elastic shortening losses for the vertical tendons are calculated based on the equations found in Section 2.1 of Reference 22. The vertical tension losses due to elastic shortening do not change over time. Note that the proportion of load in the tendon conduit is conservatively neglected from the calculation.

$$Force_{v.eshort.60.unmod} := \frac{1}{2} \cdot \frac{GUTS_{70}}{\left(A_{v.contain} - n_{v.tendon} \cdot A_t\right) \cdot E_c + A_{v.liner} \cdot E_s + n_{v.tendon} \cdot A_t \cdot E_s} \cdot n_{v.tendon} \cdot E_s \cdot A_t$$

$$Force_{v.eshort.60.unmod} = 31.84\,kip$$

$$Stress_{v.eshort.60.unmod} := \frac{Force_{v.eshort.60.unmod}}{A_t}$$

$$Stress_{v.eshort.60.unmod} = 3274\,psi$$

where

$GUTS_{70} = 1635\,kip$ $\qquad E_c = 4.03 \times 10^6\,psi$ $\qquad E_s = 2.9 \times 10^7\,psi$

$A_t = 9.723\,in^2$ $\qquad n_{v.tendon} = 144$

This loss of stress applies to tendons that were not detensioned during the SGR.

For tendons that are adjusted during SGR, the elastic shortening stress losses will be affected by the material properties of the concrete used to replace the plug and the delaminated concrete. A diagram with the different areas of concrete represented as springs with different stiffnesses is presented in Figure 2. For a unit width along the circumference of the wall passing through the SGR opening, the equivalent spring stiffness would equal:

$$\frac{E_{eq} \cdot t_f}{L_{tot}} = \frac{1}{\dfrac{L_1}{E_e \cdot t_f} + \dfrac{1}{\dfrac{E_e \cdot t_e + E_d \cdot t_d}{L_2}} + \dfrac{L_3}{E_p \cdot t_f} + \dfrac{1}{\dfrac{E_e \cdot t_e + E_d \cdot t_d}{L_4}} + \dfrac{L_5}{E_e \cdot t_f}}$$

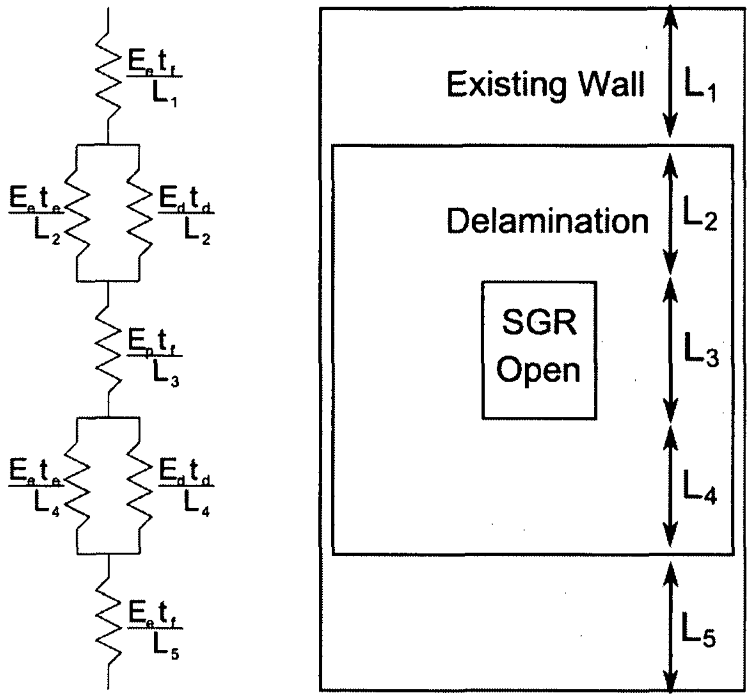


Figure 2. Spring Diagram for Vertical Stiffness of a Section of Unit Width Passing through the Reconstructed SGR Opening.

E_e	=	Existing concrete elastic modulus
E_d	=	Delamination concrete elastic modulus
E_p	=	Plug concrete elastic modulus
t_f	=	Full concrete wall thickness (between buttresses)
t_e	=	Existing concrete thickness in area of delamination, inner portion
t_d	=	Delaminated concrete thickness, outer portion
$L_{\#}$	=	Vertical length as defined in Figure 2

The value of each of the variables in the equation are defined below from inputs defined in Section 5.1:

$t_f := t_{wall}$ $\quad t_f = 3.5\,ft$

$L_1 := El_{top.ring.girder} - El_{bot.ring.girder} + L_{delam.ring.girder}$ $\quad L_1 = 27.5\,ft$

$L_2 := El_{top.ring.girder} - L_1 - El_{top.SGR}$ $\quad L_2 = 30\,ft$

$L_3 := h_{open}$ $\quad L_3 = 27\,ft$

$L_4 := El_{top.ring.girder} - L_1 - L_2 - L_3 - El_{top.eq.hatch}$ $\quad L_4 = 25.17\,ft$

$L_5 := El_{top.eq.hatch} - El_{bot.containment}$ $\quad L_5 = 77.33\,ft$

$t_d := t_{delam}$ $\quad t_d = 0.83\,ft$

$t_e := t_f - t_d$ $\quad t_e = 2.67\,ft$

Note that the wall thickness from the top of the ring girder to the bottom of the containment basemat (the entire span of the vertical tendons) is treated as a constant $t_{wall} = 3.5\,ft$ even though the wall is much thicker in the ring girder, basemat, and lower portion of the containment wall. By not accounting for the stiffness of the thicker walls, this calculation will be conservative.

The ratio of the equivalent elastic modulus of the containment in the vertical direction passing through the SGR opening compared to the modulus of the existing concrete is calculated. The calculation is based on a modulus of elasticity that is 25% higher in the plug and delamination compared to the existing concrete. This calculation will determine the relative significance of the plug material properties on the effective elastic modulus used for scaling the elastic shortening losses. The 25% increased modulus is not intended to be a definitive estimate of the new concrete properties but, rather, an estimate of the maximum difference in modulus of elasticity between new and old concrete. Based on Reference 16, 25% is a reasonable value for the difference in elastic moduli.

$E_e := 1$ $\quad$ Reference Factor

$E_d := 1.25 \cdot E_e$ $\quad E_d = 1.25$

$E_p := 1.25 \cdot E_e$ $\quad E_p = 1.25$

$$E_{eq} := \frac{L_1 + L_2 + L_3 + L_4 + L_5}{t_f\left(\frac{L_1}{E_e \cdot t_f} + \frac{1}{\frac{E_e \cdot t_e + E_d \cdot t_d}{L_2}} + \frac{L_3}{E_p \cdot t_f} + \frac{1}{\frac{E_e \cdot t_e + E_d \cdot t_d}{L_4}} + \frac{L_5}{E_e \cdot t_f}\right)}$$

$E_{eq} = 1.05$

If the modulus of elasticity for the patch and delamination replacement concrete were 25% greater than the existing concrete, the equivalent elastic modulus for the wall would be 5% greater than the modulus of the existing concrete. The same percentage decrease in the equivalent elastic modulus would be expected if the modulus of the patch and the delamination were 25% less than the existing concrete. This is a small increase in modulus. To determine the elastic shortening losses for the detensioned and retensioned tendons, the predicted loss for the existing concrete would be scaled by the same percentage. Since the exact properties of the replaced concrete are not known, the elastic shortening losses for the detensioned and retensioned tendons will be conservatively estimated to equal those of the unmodified tendons.

$Stress_{v.eshort.60.mod} := Stress_{v.eshort.60.unmod}$ $\qquad$ $Stress_{v.eshort.60.mod} = 3274\,psi$

Concrete Shrinkage

The majority of concrete shrinkage occurs in the early life of the containment. Since the containment was constructed over 30 years ago, nearly all of the shrinkage has already taken place. At this point, shrinkage is essentially time-independent, and the concrete shrinkage at 60 years will be approximately equal to the concrete shrinkage predicted at 40 years.

$Stress_{v.shrink.60.unmod} := Stress_{v.shrink.40}$ $\qquad$ $Stress_{v.shrink.60.unmod} = 2900\,psi$

This loss of stress applies to tendons that were not detensioned during the SGR.

The tendons that are detensioned and retensioned during SGR will only experience shrinkage in the concrete that replaces the SGR opening plug and that replaces the delamination. The replacement concrete is low-shrinkage concrete (Reference 11), but the shrinkage losses in this region will conservatively be set equal to the shrinkage losses of the original concrete at 40 years. However, the results will be scaled based on the ratio of the new concrete height to the entire height of the containment (The entire span of vertical tendons).

The total height of the containment is:

$h_{total} := El_{top.ring.girder} - El_{bot.containment}$ $\qquad h_{total} = 187\,ft$

where

$El_{top.ring.girder} = 267.5\,ft \qquad El_{bot.containment} = 80.5\,ft$

The height of the SGR opening is (see Section 5.1 for reference):

$h_{open} = 27\,ft$

The height of the delamination, excluding the height of the SGR opening, is:

$h_{delam} := El_{bot.ring.girder} - L_{delam.ring.girder} - El_{top.eq.hatch} - h_{open}$

$h_{delam} = 55.17\,ft$

where

$El_{bot.ring.girder} = 250\,ft \qquad L_{delam.ring.girder} = 10\,ft \qquad El_{top.eq.hatch} = 157.83\,ft$

The ratio of the delaminated thickness to the entire wall thickness is:

$Ratio_{t.delam} := \dfrac{t_{delam}}{t_{wall}}$ $\qquad Ratio_{t.delam} = 0.24$

where

$t_{delam} = 10\,in \qquad t_{wall} = 3.5\,ft$

The shrinkage loss for the tendons that are detensioned and retensioned around the SGR opening is equal to:

$$Stress_{v.shrink.60.mod} := \left(\frac{h_{open}}{h_{total}} + \frac{h_{delam}}{h_{total}} \cdot Ratio_{t.delam}\right) \cdot Stress_{v.shrink.40}$$

$Stress_{v.shrink.60.mod} = 622\,psi$

Steel Relaxation

The steel relaxation losses at 40 years have been calculated previously (Reference 7, Att. F, Page F2). Based on Figure 5-26 of Reference 10, steel relaxation is linear with time on a logarithmic scale. The losses calculated at 40 years will be extrapolated to 60 years based on the last two data points from Reference 7, Att. F, page F2.

$$Force_{v.relax.60} := \frac{Force_{v.relax.40} - Force_{v.relax.35}}{log(40) - log(35)} \cdot (log(60) - log(35)) + Force_{v.relax.35}$$

$$Force_{v.relax.60} = 49.4\,kip$$

$$Stress_{v.relax.60.unmod} := \frac{Force_{v.relax.60}}{A_t} \qquad Stress_{v.relax.60.unmod} = 5082\,psi$$

where

$$Force_{v.relax.40} = 48.5\,kip \qquad Force_{v.relax.35} = 48.2\,kip \qquad A_t = 9.723\,in^2$$

The detensioned and retensioned tendons will be active for the following number of years before the 60 year end of life is reached:

$$Age_{reten} := Age_{eol} - Age_{outage} \qquad Age_{reten} = 27\,yr$$

Conservatively using the tendon steel relaxation loss at 30 years from Reference 7, Attachment F, Page F2:

$$Force_{v.relax.60.mod} := Force_{v.relax.30} \qquad Force_{v.relax.60.mod} = 47.8\,kip$$

Converting the force loss to a prestress loss in the tendon:

$$Stress_{v.relax.60.mod} := \frac{Force_{v.relax.60.mod}}{A_t} \qquad Stress_{v.relax.60.mod} = 4916\,psi$$

Creep

The basic creep for the existing concrete determined from testing and extrapolated to 60 years is (see Section 5.1 for reference):

$$Creep_{v.basic.60} = 2.5 \times 10^{-7} \frac{1}{psi}$$

The ratio of the concrete stiffness to the total stiffness of the horizontal cross-section is calculated based on the equations in Section 2.1 of Reference 22.

$$Ratio_{v.conc.stiff} := \frac{A_{v.contain} \cdot E_c}{\left(A_{v.contain} - n_{v.tendon} \cdot A_t\right) \cdot E_c + A_{v.liner} \cdot E_s + n_{v.tendon} \cdot A_t \cdot E_s}$$

$$Ratio_{v.conc.stiff} = 0.92$$

where

$A_{v.contain} = 236807.3\, in^2$ $\quad E_c = 4.03 \times 10^6\, psi$ $\quad n_{v.tendon} = 144$

$A_t = 9.723\, in^2$ $\quad A_{v.liner} = 1838.3\, in^2$ $\quad E_s = 2.9 \times 10^7\, psi$

The average vertical prestress was calculated earlier in this section. For the stress contributing to creep, elastic shortening and shrinkage losses are subtracted because they occur early in the life of the containment.

$$\sigma_{v.axial.creep} := \frac{Force_{v.axial} - n_{v.tendon} \cdot \left(Stress_{v.eshort.60.unmod} + Stress_{v.shrink.40}\right) \cdot A_t}{A_{v.contain}} \cdot Ratio_{v.conc.stiff}$$

$$\sigma_{v.axial.creep} = 877\, psi$$

where

$Force_{v.axial} = 235440\, kip$ $\quad Stress_{v.shrink.40} = 2900\, psi$ $\quad Stress_{v.eshort.60.unmod} = 3274\, psi$

$A_t = 9.723\, in^2$ $\quad A_{v.contain} = 236807.3\, in^2$

The tendon prestress lost due to creep is calculated based on Page 4 of Reference 2. This value is applicable to tendons that were not detensioned during the SGR:

$Stress_{v.creep.60.unmod} := \sigma_{v.axial.creep} \cdot Creep_{v.basic.60} \cdot E_s$ $\qquad Stress_{v.creep.60.unmod} = 6356\,psi$

where E_s is the steel elastic modulus and is equal to:

$E_s = 2.9 \times 10^7\,psi$

Creep losses for the tendons that pass through the patch are calculated separately by taking into account the creep properties of the replacement concrete. The ultimate creep coefficient of the new concrete is (see Section 5.1 for reference):

$v_{u.patch} = 1.14$

The ultimate creep coefficient must be adjusted for non-standard environmental and geometrical properties in accordance with Reference 12, Section 2.5. There are also correction factors associated with concrete composition, but these have a smaller effect than geometrical and environmental properties and are neglected (Reference 12, Section 2.6).

The correction factor for the ultimate creep coefficient due to the relative humidity is expressed by (Reference 12, Section 2.5.4):

$\lambda = 75$ $\qquad$ Relative Humidity, (%)

$\gamma_\lambda := 1.27 - 0.0067\lambda$ $\qquad \gamma_\lambda = 0.767$

The vol[illegible] ratio of the plug and the delamination is calculated as follows. The wid[illegible] [illegible]lamination is

$$w_{delam} := [illegible] \left([illegible] + t_{liner} + t_{wall} - \frac{t_{delam}}{2} \right) - w_{buttress}$$

$[illegible] = 58.99[illegible]$

where

$\alpha_{buttress} = 60\,deg$ $\qquad R_{liner} = 65\,ft$ $\qquad t_{liner} = 0.38\,in$

$t_{wall} = 3.5\,ft$ $\qquad t_{delam} = 10\,in$ $\qquad w_{buttress} = 12.34\,ft$

The volume of the new concrete is:

$$V_{new} := w_{open} \cdot t_{wall} \cdot h_{open} + \left(w_{delam} \cdot h_{delam} - w_{open} \cdot h_{open}\right) \cdot t_{delam}$$

$$V_{new} = 4511.7\, ft^3$$

where

$w_{open} = 25\, ft$ $\quad$ $t_{wall} = 3.5\, ft$ $\quad$ $h_{open} = 27\, ft$

$w_{delam} = 58.99\, ft$ $\quad$ $h_{delam} = 55.17\, ft$ $\quad$ $t_{delam} = 10\, in$

The only surface exposed to the environment for the new concrete is the outside surface of the containment. This area is equal to:

$$S_{new} := w_{delam} \cdot h_{delam} \qquad S_{new} = 3254.04\, ft^2$$

where

$w_{delam} = 58.99\, ft$ $\quad$ $h_{delam} = 55.17\, ft$

The volume to surface area ratio is:

$$Ratio_{vs} := \frac{V_{new}}{S_{new}} \qquad Ratio_{vs} = 16.64\, in$$

The correction factor to the ultimate creep coefficient for the volume to surface ratio is (Reference 12, Section 2.5.5b):

$$\gamma_{vs} := \frac{2}{3} \cdot \left(1 + 1.13 \cdot e^{-0.54 \cdot Ratio_{vs} \div in}\right) \qquad \gamma_{vs} = 0.667$$

A correction factor must also be applied for operating temperature other than 70°F. Based on Reference 23, operating temperature correction will have a small effect on the concrete creep rate and is, therefore, neglected.

A correction factor is also to be applied when load is applied other than 7 days after concrete placement from Reference 12, Section 2.5.1. However, the ultimate creep coefficient was calculated based on a loading age of 5 days, and the load is assumed to be applied at 5 days in this calculation (see Assumption 3.2.3), so no correction for loading age is applied.

The resulting creep correction factor accounting for relative humidity, volume to surface ratio, and operating temperature effects is (see definition of γ in Reference 12):

$\gamma_{creep} := \gamma_{vs} \cdot \gamma_{\lambda}$ $\qquad \gamma_{creep} = 0.512$

The new concrete will be under load for $Age_{reten} = 27\,yr$. The creep coefficient at the end of this time is (Reference 12, Equation 2-8):

$$v_t := \frac{\left(Age_{reten} \div day\right)^{0.6}}{10 + \left(Age_{reten} \div day\right)^{0.6}} \cdot v_{u.patch} \cdot \gamma_{creep} \qquad v_t = 0.561$$

Note that this equation is applicable to Types I and III concrete. The concrete is Type I in accordance with Reference 6, page 3.

The tendon tension lost due to creep of the new concrete is scaled based on the tension lost due to elastic shortening. Elastic shortening is a short term loss and creep is a long term loss. The creep loss can be scaled from the elastic shortening loss by the effective short term and age-adjusted elastic moduli. The age-adjusted elastic modulus accounts for additional strain due to long term loads (Reference 12, Section 5.2). The short term losses (elastic shortening losses) can be scaled using the following equation (this equation was used in Reference 14, but was not derived in Reference 14. It is derived here for clarity.):

$$Loss_{creep} = Loss_{eshort} \cdot \frac{E_{eshort}}{E_{creep}} - Loss_{eshort} = Loss_{eshort} \cdot \left(\frac{E_{eshort}}{E_{creep}} - 1\right)$$

where E_{eshort} is the instantaneous modulus of elasticity (used for short term loads), E_{creep} is the effective modulus of elasticity for long term loads, and $Loss_{eshort}$ is the tendon elastic shortening loss.

The ratio of the effective modulus of elasticity for a short term load to a long term load minus one is determined by rearranging equation 5-1 of Reference 12.

$$\frac{E_{st}}{E_{lt}} - 1 = X v_t$$

where

E_{st} = Modulus of elasticity for short term loads
E_{lt} = Effective modulus of elasticity for long term loads
X = Aging coefficient
v_t = Creep coefficient

Looking at the aging coefficients in Table 5.1.1 of Reference 12, the maximum this value can be is 1 and the minimum value is 0.5. Conservatively assuming a value of 1 for X, the tendon loss due to creep in the new concrete can be calculated as follows based on combining the previous two equations:

$$Loss_{creep} = Loss_{eshort} \cdot v_t$$

The total loss in the new concrete is scaled based on the proportion of the height and cross-sectional ratio of the new concrete to the height and total thickness of the containment wall. The remaining concrete will creep following the same trend from the measured data in Reference 3, page 45. The creep experienced by the existing concrete up to the beginning of the SGR outage (33 years) is:

$$Stress_{v.creep.33.unmod} := \sigma_{v.axial.creep} \cdot Creep_{v.basic.33} \cdot E_s \qquad Stress_{v.creep.33.unmod} = 5466\,psi$$

The total creep loss in the vertical tendons at 60 years is:

$$Stress_{v.creep.60.mod} := \frac{h_{open}}{h_{total}} \cdot \left(Stress_{v.eshort.60.mod} \cdot v_t\right) + \frac{h_{delam}}{h_{total}} \cdot Ratio_{t.delam} \cdot \left(Stress_{v.eshort.60.mod} \cdot v_t\right) \dots$$
$$+ \left(\frac{h_{total} - h_{open}}{h_{total}} - \frac{h_{delam}}{h_{total}} \cdot Ratio_{t.delam}\right) \cdot \left(Stress_{v.creep.60.unmod} - Stress_{v.creep.33.unmod}\right)$$

$$Stress_{v.creep.60.mod} = 1093\,psi$$

where

$h_{open} = 27\,ft$ $\qquad h_{total} = 187\,ft$ $\qquad Stress_{v.eshort.60.mod} = 3274\,psi$

$v_t = 0.561$ $\qquad h_{delam} = 55.17\,ft$ $\qquad Ratio_{t.delam} = 0.24$

$Stress_{v.creep.60.unmod} = 6356\,psi$

Total 60 Year Loss

The total tendon stress loss after 60 years is calculated.

Unadjusted Tendons:

$$Stress_{v.total.60.unmod} := Stress_{v.eshort.60.unmod} + Stress_{v.shrink.60.unmod} + Stress_{v.relax.60.unmod} + Stress_{v.creep.60.unmod}$$

$$Stress_{v.total.60.unmod} = 17612\,psi$$

Detensioned and Retensioned Tendons:

$$Stress_{v.total.60.mod} := Stress_{v.eshort.60.mod} + Stress_{v.shrink.60.mod} + Stress_{v.relax.60.mod} + Stress_{v.creep.60.mod}$$

$$Stress_{v.total.60.mod} = 9906\,psi$$

Converting the stress lost into a force per tendon that is lost:

$$Force_{v.total.60.unmod} := Stress_{v.total.60.unmod} \cdot A_t \qquad Force_{v.total.60.unmod} = 171.2\,kip$$

$$Force_{v.total.60.mod} := Stress_{v.total.60.mod} \cdot A_t \qquad Force_{v.total.60.mod} = 96.3\,kip$$

where

$$A_t = 9.723\,in^2$$

The design tension per tendon is (see Section 5.2 for original calculation):

$$Force_{design} = 1635\,kip$$

The remaining tension in the vertical tendons at 60 years is:

$$Tension_{v.60.unmod} := Force_{design} - Force_{v.total.60.unmod} \qquad Tension_{v.60.unmod} = 1463.8\,kip$$

$$Tension_{v.60.mod} := Force_{design} - Force_{v.total.60.mod} \qquad Tension_{v.60.mod} = 1538.7\,kip$$

5.4 Horizontal Tendons - 60 Years After Initial SIT

Some of the horizontal tendons near and away from the SGR opening will be detensioned and retensioned during SGR while the tension in some other tendons will not be changed. The tendon tension at 60 years will be individually calculated for the detensioned and retensioned tendons that pass through the SGR opening bay, the detensioned and retensioned tendons that do not pass through the SGR opening bay, and the tendons that are not detensioned. The tension losses for the tendons that pass through the SGR opening bay will be calculated considering all of these tendons to pass directly through the SGR opening since cutting and repairing the opening will affect the region both inside and around the SGR opening. The tendon tension at 60 years after the Structural Integrity Test (SIT) of November 1976 (Reference 7, page 10) is determined by scaling the predicted tension at 40 years after SIT. The individual losses in the horizontal tendons at 40 years after SIT from steel stress relaxation and concrete shrinkage are as follows (see Section 5.1 for references):

$Stress_{h.shrink.40} = 2900\,psi$

$$Stress_{h.relax.40} := \frac{Force_{h.relax.40}}{A_t} \qquad Stress_{h.relax.40} = 4957\,psi$$

where

$A_t = 9.723\,in^2 \qquad Force_{h.relax.40} = 48.2\,kip$

Elastic Shortening

The total circumferential force from a single horizontal tendon tensioned to lock off load is:

$Force_{h.axial} := GUTS_{70} \qquad Force_{h.axial} = 1635\,kip$

where

$GUTS_{70} = 1635\,kip$

The average spacing between horizontal tendons near the containment mid-height is:

$$s_{h.avg} := \frac{El_{avg.tend.space.top} - El_{avg.tend.space.bot}}{n_{avg.tend.space} - 1} \qquad s_{h.avg} = 19.19\,in$$

where

$El_{avg.tend.space.top} = 212.69\,ft \qquad El_{avg.tend.space.bot} = 183.9\,ft \qquad n_{avg.tend.space} = 19$

The average elastic shortening losses for the horizontal tendons are calculated based on the equations found in Section 2.1 of Reference 22. The horizontal tension losses due to elastic shortening do not change over time. These losses are applicable for the tendons that are not detensioned during the SGR. Note that the proportion of load in the tendon conduit is conservatively neglected from the calculation.

$$Force_{h.eshort.60.unmod} := \frac{1}{2} \cdot \frac{Force_{h.axial}}{\left(s_{h.avg} \cdot t_{wall} - A_t\right) \cdot E_c + s_{h.avg} \cdot t_{liner} \cdot E_s + A_t \cdot E_s} \cdot A_t \cdot E_s$$

$$Force_{h.eshort.60.unmod} = 62.29\,kip$$

$$Stress_{h.eshort.60.unmod} := \frac{Force_{h.eshort.60.unmod}}{A_t}$$

$$Stress_{h.eshort.60.unmod} = 6407\,psi$$

This loss of stress applies to tendons that were not detensioned during the SGR.

For tendons that are adjusted during SGR, the elastic shortening stress losses will be affected by the material properties of the concrete used to replace the plug and to replace the outer portion of the delaminated concrete. As demonstrated in Section 5.3, the effect of the plug stiffness has a small effect on the elastic shortening losses, so the elastic shortening losses are estimated to be the same for all tendons.

$Stress_{h.eshort.60.mod} := Stress_{h.eshort.60.unmod}$ $\quad$ $Stress_{h.eshort.60.mod} = 6407\,psi$

$Stress_{h.eshort.60.mod.SGR} := Stress_{h.eshort.60.unmod}$ $\quad$ $Stress_{h.eshort.60.mod.SGR} = 6407\,psi$

Concrete Shrinkage

The majority of concrete shrinkage occurs in the early life of the containment. Since the containment was built over 30 years ago, nearly all of the shrinkage has already taken place. At this point, shrinkage is essentially time-independent, and the concrete shrinkage at 60 years will be approximately equal to the concrete shrinkage predicted at 40 years.

$Stress_{h.shrink.60.unmod} := Stress_{h.shrink.40}$ $\quad$ $Stress_{h.shrink.60.unmod} = 2900\,psi$

This loss of stress applies to tendons that were not detensioned during the SGR.

The tendons passing through the SGR opening bay that are detensioned and retensioned during SGR will only experience shrinkage in the concrete that is replaced in the plug and that replaces the delamination. The replacement concrete is low-shrinkage concrete (Assumption 3.2.4), but the shrinkage losses in this region will conservatively be set equal to the shrinkage losses of the original concrete at 40 years. However, the results will be scaled based on the proportion of the span of new concrete to the entire span of the containment wall.

The total circumferential length of a horizontal tendon is:

$w_{total} := 2\alpha_{buttress} \cdot r_{h.tendon} + w_{buttress}$ $\qquad$ $w_{total} = 154.17\,ft$

where

$\alpha_{buttress} = 1.05\,rad$ $\qquad$ $w_{buttress} = 12.34\,ft$ $\qquad$ $r_{h.tendon} = 67.72\,ft$

The span of the SGR opening is (see Section 5.1 for reference):

$w_{open} = 25\,ft$

The circumferential length of the repaired delamination, excluding the span of the SGR opening, is:

$w_{delam.sub.SGR} := \alpha_{buttress} \cdot r_{h.tendon} - w_{buttress} - w_{open}$

$w_{delam.sub.SGR} = 33.57\,ft$

where

$\alpha_{buttress} = 1.05\,rad$ $\qquad$ $w_{buttress} = 12.34\,ft$ $\qquad$ $w_{open} = 25\,ft$

$r_{h.tendon} = 67.72\,ft$

The ratio of the thickness of the repaired delamination to the entire wall thickness is (see Section 5.3 for original calculation):

$Ratio_{t.delam} = 0.24$

The shrinkage loss for the tendons that are detensioned and retensioned around the SGR opening is equal to:

$$Stress_{h.shrink.60.mod.SGR} := \left(\frac{w_{open}}{w_{total}} + \frac{w_{delam.sub.SGR}}{w_{total}} \cdot Ratio_{t.delam}\right) \cdot Stress_{h.shrink.40}$$

$Stress_{h.shrink.60.mod.SGR} = 621\,psi$

The detensioned and retensioned tendons that do not pass through the SGR opening bay will not experience any shrinkage since there is no new concrete in the span of these tendons.

$$Stress_{h.shrink.60.mod} := 0$$

Steel Relaxation

The steel relaxation losses at 40 years have been calculated previously (Reference 7, Att. F, Page F2). Based on Figure 5-26 of Reference 10, steel relaxation is linear with time on a logarithmic scale. The losses calculated at 40 years will be extrapolated to 60 years based on the last two data points from Reference 7, Att. F, page F2.

$$Force_{h.relax.60} := \frac{Force_{h.relax.40} - Force_{h.relax.35}}{log(40) - log(35)} \cdot (log(60) - log(35)) + Force_{h.relax.35}$$

$$Force_{h.relax.60} = 49.1\,kip$$

$$Stress_{h.relax.60.unmod} := \frac{Force_{h.relax.60}}{A_t} \qquad Stress_{h.relax.60.unmod} = 5051\,psi$$

where

$$Force_{h.relax.40} = 48.2\,kip \qquad Force_{h.relax.35} = 47.9\,kip \qquad A_t = 9.723\,in^2$$

The detensioned and retensioned tendons will be active for the following number of years before the 60 year end of life is reached (see Section 5.3 for original calculation):

$$Age_{reten} = 27\,yr$$

Conservatively using the tendon steel relaxation loss in the horizontal direction at 30 years from Reference 7, Attachment F, Page F2:

$$Force_{h.relax.60.mod.SGR} := Force_{h.relax.30} \qquad Force_{h.relax.60.mod.SGR} = 47.6\,kip$$

Converting the force loss to a prestress loss in the tendon:

$$Stress_{h.relax.60.mod.SGR} := \frac{Force_{h.relax.60.mod.SGR}}{A_t} \qquad Stress_{h.relax.60.mod.SGR} = 4896\,psi$$

This loss is also appropriate for the detensioned and retensioned tendons that do not pass through the SGR opening bay.

$Stress_{h.relax.60.mod} := Stress_{h.relax.60.mod.SGR}$ $\qquad Stress_{h.relax.60.mod} = 4896\,psi$

Creep

The basic creep for the existing concrete determined from testing and extrapolated to 60 years is (see Section 5.1 for reference):

$$Creep_{h.basic.60} = 2.4 \times 10^{-7} \frac{1}{psi}$$

The ratio of the concrete stiffness to the total stiffness through the cross-section of the containment wall is calculated based on the equations in Section 2.1 of Reference 22.

$$Ratio_{h.conc.stiff} := \frac{s_{h.avg} \cdot t_{wall} \cdot E_c}{\left(s_{h.avg} \cdot t_{wall} - A_t\right) \cdot E_c + s_{h.avg} \cdot t_{liner} \cdot E_s + A_t \cdot E_s}$$

$$Ratio_{h.conc.stiff} = 0.88$$

For the stress in the concrete contributing to creep, elastic shortening and shrinkage losses are subtracted because they occur early in the life of the containment.

$$\sigma_{h.axial.creep} := \frac{GUTS_{70} - \left(Stress_{h.eshort.60.unmod} + Stress_{h.shrink.40}\right) \cdot A_t}{s_{h.avg} \cdot t_{wall} - A_t} \cdot Ratio_{h.conc.stiff}$$

$$\sigma_{h.axial.creep} = 1703\,psi$$

where

$Stress_{h.eshort.60.unmod} = 6407\,psi$ $\qquad Stress_{h.shrink.40} = 2900\,psi$

$A_t = 9.723\,in^2$ $\qquad s_{h.avg} = 19.19\,in$ $\qquad t_{wall} = 3.5\,ft$

$GUTS_{70} = 1635\,kip$

The tendon prestress lost due to creep is calculated based on Page 4 of Reference 2. This value is applicable to tendons that are not detensioned during the SGR:

$$Stress_{h.creep.60.unmod} := \sigma_{h.axial.creep} \cdot Creep_{h.basic.60} \cdot E_s \qquad Stress_{h.creep.60.unmod} = 11850\,psi$$

where E_s is the steel elastic modulus and is equal to:

$$E_s = 2.9 \times 10^7\,psi$$

Creep losses for the tendons that pass through the patch are calculated separately by taking into account the creep properties of the replacement concrete. The creep properties have been calculated in Section 5.3 of this calculation. The creep coefficient for end of life is:

$$v_t = 0.56$$

The total loss in the new concrete is scaled based on the proportion of the width and cross-sectional ratio of the new concrete to the horizontal tendon lateral span and total thickness of the containment wall. The remaining concrete will creep following the same trend from the measured data in Reference 3, page 45. The methodology used here is duplicated from Section 5.3 of this calculation. The creep experienced by the existing concrete up to the beginning of the SGR outage (33 years) is:

$$Stress_{h.creep.33.unmod} := \sigma_{h.axial.creep} \cdot Creep_{h.basic.33} \cdot E_s \qquad Stress_{h.creep.33.unmod} = 10122\,psi$$

where

$$Creep_{h.basic.33} = 2.05 \times 10^{-7} \frac{1}{psi}$$

The total creep loss in the horizontal tendons at 60 years is:

$$Stress_{h.creep.60.mod.SGR} := \frac{w_{open}}{w_{total}} \cdot \left(Stress_{h.eshort.60.mod} \cdot v_t\right) + \frac{w_{delam.sub.SGR}}{w_{total}} \cdot Ratio_{t.delam} \cdot \left(Stress_{h.eshort.60.mod} \cdot v_t\right) \ldots$$
$$+ \left(\frac{w_{total} - w_{open}}{w_{total}} - \frac{w_{delam.sub.SGR}}{w_{total}} \cdot Ratio_{t.delam}\right) \cdot \left(Stress_{h.creep.60.unmod} - Stress_{h.creep.33.unmod}\right)$$

$Stress_{h.creep.60.mod.SGR} = 2127\,psi$

where

$w_{open} = 25\,ft$ $w_{total} = 154.17\,ft$ $Stress_{h.eshort.60.mod} = 6407\,psi$

$v_t = 0.561$ $w_{delam.sub.SGR} = 33.57\,ft$ $Ratio_{t.delam} = 0.24$

$Stress_{h.creep.60.unmod} = 11850\,psi$

The total creep loss for the horizontal tendons that do not pass through the SGR opening bay is:

$Stress_{h.creep.60.mod} := Stress_{h.creep.60.unmod} - Stress_{h.creep.33.unmod}$

$Stress_{h.creep.60.mod} = 1728\,psi$

Total 60 Year Loss

The total tendon stress loss after 60 years is calculated.

Unadjusted Tendons:

$Stress_{h.total.60.unmod} := Stress_{h.eshort.60.unmod} + Stress_{h.shrink.60.unmod} + Stress_{h.relax.60.unmod} + Stress_{h.creep.60.unmod}$

$Stress_{h.total.60.unmod} = 26208\,psi$

Detensioned and Retensioned Tendons that do not Pass through SGR Opening:

$Stress_{h.total.60.mod} := Stress_{h.eshort.60.mod} + Stress_{h.shrink.60.mod} + Stress_{h.relax.60.mod} + Stress_{h.creep.60.mod}$

$Stress_{h.total.60.mod} = 13031\,psi$

Detensioned and Retensioned Tendons that Pass through SGR Opening:

$Stress_{h.total.60.mod.SGR} := Stress_{h.eshort.60.mod.SGR} + Stress_{h.shrink.60.mod.SGR} + Stress_{h.relax.60.mod.SGR} + Stress_{h.creep.60.mod.SGR}$

$Stress_{h.total.60.mod.SGR} = 14050\,psi$

Converting the stress lost into a force per tendon that is lost:

$Force_{h.total.60.unmod} := Stress_{h.total.60.unmod} \cdot A_t$ $\quad Force_{h.total.60.unmod} = 254.8\,kip$

$Force_{h.total.60.mod} := Stress_{h.total.60.mod} \cdot A_t$ $\quad Force_{h.total.60.mod} = 126.7\,kip$

$Force_{h.total.60.mod.SGR} := Stress_{h.total.60.mod.SGR} \cdot A_t$ $\quad Force_{h.total.60.mod.SGR} = 136.6\,kip$

where

$A_t = 9.723\,in^2$

The design tension per tendon is (see Section 5.2 for original calculation):

$Force_{design} = 1635\,kip$

The remaining tension in the horizontal tendons at 60 years is:

$Tension_{h.60.unmod} := Force_{design} - Force_{h.total.60.unmod}$ $\quad Tension_{h.60.unmod} = 1380.2\,kip$

$Tension_{h.60.mod} := Force_{design} - Force_{h.total.60.mod}$ $\quad Tension_{h.60.mod} = 1508.3\,kip$

$Tension_{h.60.mod.SGR} := Force_{design} - Force_{h.total.60.mod.SGR}$ $\quad Tension_{h.60.mod.SGR} = 1498.4\,kip$

5.5 Dome Tendons - After SGR Completion

After the Steam Generator Replacement Project is completed, the containment will only have experienced 33 years of its 60 year life. The tendon losses are expected to be less at this time compared to the losses after 60 years. The total losses after SGR completion are calculated

Elastic Shortening

The elastic shortening losses are not time dependent. The elastic shortening losses after 33 years will be equal to the elastic shortening losses calculated for 60 years in Section 5.2.

$Stress_{d.eshort.33} := Stress_{d.eshort.60}$ $\qquad$ $Stress_{d.eshort.33} = 5500\,psi$

Concrete Shrinkage

The concrete shrinkage losses will be essentially independent of time after 33 years. Therefore, the concrete shrinkage losses calculated for the 60 year end of life calculated in Section 5.2 are appropriate for the 33 year losses.

$Stress_{d.shrink.33} := Stress_{d.shrink.60}$ $\qquad$ $Stress_{d.shrink.33} = 2900\,psi$

Wire Relaxation

The wire relaxation losses are interpolated from Reference 7, Appendix F, Page F2. The wire relaxation loss at 33 years is:

$Force_{d.relax.33} = 48\,kip$

Converting this load into a stress loss in the tendon:

$Stress_{d.relax.33} := \dfrac{Force_{d.relax.33}}{A_t}$ $\qquad$ $Stress_{d.relax.33} = 4937\,psi$

Creep

The tendon tension loss due to creep can be calculated using the basic creep at 33 years. The basic creep at 33 years is defined in Section 5.1:

$$Creep_{d.basic.33} = 3.00 \times 10^{-7} \frac{1}{psi}$$

The stress in the dome in the direction of the dome tendons is (see Section 5.1 for reference):

$$\sigma_{d.axial} = 1530\,psi$$

The creep loss at 33 years is:

$$Stress_{d.creep.33} := Creep_{d.basic.33} \cdot \sigma_{d.axial} \cdot E_s \qquad Stress_{d.creep.33} = 13311\,psi$$

where

$$E_s = 2.9 \times 10^{7}\,psi$$

Total Loss at 33 Years

The total tendon stress loss after 33 years is:

$$Stress_{d.total.33} := Stress_{d.eshort.33} + Stress_{d.shrink.33} + Stress_{d.relax.33} + Stress_{d.creep.33}$$

$$Stress_{d.total.33} = 26648\,psi$$

Converting the stress lost into a force per tendon that is lost:

$$Force_{d.total.33} := Stress_{d.total.33} \cdot A_t \qquad Force_{d.total.33} = 259.1\,kip$$

where

$$A_t = 9.723\,in^2$$

The design tension per tendon is (see Section 5.2 for original calculation):

$$Force_{design} = 1635\,kip$$

The remaining tension in the dome tendons at 33 years is:

$Tension_{d.33} := Force_{design} - Force_{d.total.33}$ $\qquad Tension_{d.33} = 1375.9\,kip$

5.6 Vertical Tendons - After SGR Completion

After the Steam Generator Replacement Project is completed, the containment will only have experienced 33 years of its 60 year life. The tendon losses are less at this time compared to the losses after 60 years. The total losses after SGR completion are calculated

Elastic Shortening

The elastic shortening losses are not time dependent. The elastic shortening losses after 33 years will be equal to the elastic shortening losses calculated for 60 years in Section 5.3. These losses are applicable to both tendons that are detensioned and retensioned and those that are not.

$Stress_{v.eshort.33.unmod} := Stress_{v.eshort.60.unmod}$ $\qquad Stress_{v.eshort.33.unmod} = 3274\,psi$

$Stress_{v.eshort.33.mod} := Stress_{v.eshort.60.mod}$ $\qquad Stress_{v.eshort.33.mod} = 3274\,psi$

Concrete Shrinkage

For the existing concrete, the concrete shrinkage losses will be essentially independent of time after 33 years. Therefore, the concrete shrinkage losses calculated for the 60 year end of life calculated in Section 5.3 are appropriate for the 33 year losses for these tendons. For the tendons that are detensioned and retensioned, the new concrete will not have experienced any significant shrinkage immediately after the tendons are retensioned.

$Stress_{v.shrink.33.unmod} := Stress_{v.shrink.60.unmod}$ $\qquad Stress_{v.shrink.33.unmod} = 2900\,psi$

$Stress_{v.shrink.33.mod} := 0$

Wire Relaxation

The wire relaxation losses are interpolated from Reference 7, Appendix F, Page F2. These losses are applicable for the tendons that are unadjusted during SGR. The tendons that are detensioned and retensioned will not experience any significant relaxation immediately after they are retensioned.

$Force_{v.relax.33.unmod} = 48.0\,kip$

Converting this load into a stress loss in the tendon:

$Stress_{v.relax.33.unmod} := \dfrac{Force_{v.relax.33.unmod}}{A_t}$ $\qquad Stress_{v.relax.33.unmod} = 4937\,psi$

$Stress_{v.relax.33.mod} := 0$

Creep

The tendon tension loss due to creep of the existing concrete can be calculated using the basic creep at 33 years. This calculation was performed in Section 5.3. The creep loss in the tendons that are unadjusted is equal to this value. The tendons that are detensioned and retensioned do not experience any significant creep immediately after retensioning.

$Stress_{v.creep.33.unmod} = 5466\,psi$

$Stress_{v.creep.33.mod} := 0$

Total Loss at 33 Years

The total tendon stress loss after 33 years is:

$Stress_{v.total.33.unmod} := Stress_{v.eshort.33.unmod} + Stress_{v.shrink.33.unmod} + Stress_{v.relax.33.unmod} + Stress_{v.creep.33.unmod}$

$Stress_{v.total.33.unmod} = 16577\,psi$

$Stress_{v.total.33.mod} := Stress_{v.eshort.33.mod} + Stress_{v.shrink.33.mod} + Stress_{v.relax.33.mod} + Stress_{v.creep.33.mod}$

$Stress_{v.total.33.mod} = 3274\,psi$

Converting the stress lost into a force per tendon that is lost:

$Force_{v.total.33.unmod} := Stress_{v.total.33.unmod} \cdot A_t$ $\qquad Force_{v.total.33.unmod} = 161.2\,kip$

$Force_{v.total.33.mod} := Stress_{v.total.33.mod} \cdot A_t$ $\qquad Force_{v.total.33.mod} = 31.8\,kip$

where

$A_t = 9.723\,in^2$

The design tension per tendon is (see Section 5.2 for original calculation):

$Force_{design} = 1635\,kip$

The remaining tension in the vertical tendons at 33 years is:

$Tension_{v.33.unmod} := Force_{design} - Force_{v.total.33.unmod}$ $\quad Tension_{v.33.unmod} = 1473.8\,kip$

$Tension_{v.33.mod} := Force_{design} - Force_{v.total.33.mod}$ $\quad Tension_{v.33.mod} = 1603.2\,kip$

5.7 Horizontal Tendons - After SGR Completion

After the Steam Generator Replacement Project is completed, the containment will only have experienced 33 years of its 60 year life. The tendon losses are less at this time compared to the losses after 60 years. The total losses after SGR completion are calculated

Elastic Shortening

The elastic shortening losses are not time dependent. The elastic shortening losses after 33 years will be equal to the elastic shortening losses calculated for 60 years in Section 5.3. These losses are applicable to both tendons that are detensioned and retensioned and those that are not.

$Stress_{h.eshort.33.unmod} := Stress_{h.eshort.60.unmod}$ $\quad Stress_{h.eshort.33.unmod} = 6407\ psi$

$Stress_{h.eshort.33.mod} := Stress_{h.eshort.60.mod}$ $\quad Stress_{h.eshort.33.mod} = 6407\ psi$

$Stress_{h.eshort.33.mod.SGR} := Stress_{h.eshort.60.mod.SGR}$ $\quad Stress_{h.eshort.33.mod.SGR} = 6407\ psi$

Concrete Shrinkage

For the existing concrete, the concrete shrinkage losses will be essentially independent of time after 33 years. Therefore, the concrete shrinkage losses calculated for the 60 year end of life in Section 5.3 are appropriate for the 33 year losses for these tendons. For the tendons that are detensioned and retensioned, the new concrete will not have experienced any significant shrinkage immediately after the tendons are retensioned.

$Stress_{h.shrink.33.unmod} := Stress_{h.shrink.60.unmod}$ $\quad Stress_{h.shrink.33.unmod} = 2900\ psi$

$Stress_{h.shrink.33.mod} := 0$

$Stress_{h.shrink.33.mod.SGR} := 0$

Wire Relaxation

The wire relaxation losses are interpolated from Reference 7, Appendix F, Page F2. These losses are applicable for the tendons that are unadjusted during SGR. The tendons that are detensioned and retensioned will not experience any significant relaxation immediately after they are retensioned.

$Force_{h.relax.33.unmod} = 47.8\ kip$

Converting this load into a stress loss in the tendon:

$Stress_{h.relax.33.unmod} := \frac{Force_{h.relax.33.unmod}}{A_t}$ $\qquad Stress_{h.relax.33.unmod} = 4916\,psi$

$Stress_{h.relax.33.mod} := 0$

$Stress_{h.relax.33.mod.SGR} := 0$

Creep

The tendon tension loss due to creep of the existing concrete can be calculated using the basic creep at 33 years. This calculation was performed in Section 5.3. The creep loss in the tendons that are unadjusted is equal to this value. The tendons that are detensioned and retensioned do not experience any significant creep immediately after retensioning.

$Stress_{h.creep.33.unmod} = 10122\,psi$

$Stress_{h.creep.33.mod} := 0$

$Stress_{h.creep.33.mod.SGR} := 0$

Total Loss at 33 Years

The total tendon stress loss after 33 years is:

$Stress_{h.total.33.unmod} := Stress_{h.eshort.33.unmod} + Stress_{h.shrink.33.unmod} + Stress_{h.relax.33.unmod} + Stress_{h.creep.33.unmod}$

$Stress_{h.total.33.unmod} = 24345\,psi$

$Stress_{h.total.33.mod} := Stress_{h.eshort.33.mod} + Stress_{h.shrink.33.mod} + Stress_{h.relax.33.mod} + Stress_{h.creep.33.mod}$

$Stress_{h.total.33.mod} = 6407\,psi$

$Stress_{h.total.33.mod.SGR} := Stress_{h.eshort.33.mod.SGR} + Stress_{h.shrink.33.mod.SGR} + Stress_{h.relax.33.mod.SGR} + Stress_{h.creep.33.mod.SGR}$

$Stress_{h.total.33.mod.SGR} = 6407\,psi$

Converting the stress lost into a force per tendon that is lost:

$Force_{h.total.33.unmod} := Stress_{h.total.33.unmod} \cdot A_t$ $\quad Force_{h.total.33.unmod} = 236.7\,kip$

$Force_{h.total.33.mod} := Stress_{h.total.33.mod} \cdot A_t$ $\quad Force_{h.total.33.mod} = 62.3\,kip$

$Force_{h.total.33.mod.SGR} := Stress_{h.total.33.mod.SGR} \cdot A_t$ $\quad Force_{h.total.33.mod.SGR} = 62.3\,kip$

where

$A_t = 9.723\,in^2$

The design tension per tendon is (see Section 5.2 for original calculation):

$Force_{design} = 1635\,kip$

The remaining tension in the horizontal tendons at 33 years is:

$Tension_{h.33.unmod} := Force_{design} - Force_{h.total.33.unmod}$ $\quad Tension_{h.33.unmod} = 1398.3\,kip$

$Tension_{h.33.mod} := Force_{design} - Force_{h.total.33.mod}$ $\quad Tension_{h.33.mod} = 1572.7\,kip$

$Tension_{h.33.mod.SGR} := Force_{design} - Force_{h.total.33.mod.SGR}$ $\quad Tension_{h.33.mod.SGR} = 1572.7\,kip$

6.0 REFERENCES

1. Crystal River Unit 3, "Design Basis Document for the Containment," Revision 7.

2. Gilbert Associates Inc. Calculation 1.01.19.

3. Gilbert Associates Inc. Calculation 1.01.7.

4. Roark, Raymond J. and Warren C. Young, *Formulas for Stress and Strain*, McGraw Hill, 5th Ed., 1975.

5. Progress Energy Drawing 421-347, Sheet 1, "Reactor Building Temporary Access Opening for SGR Vertical & Horizontal Tendon Positions," Rev. 0.

6. S&ME, Inc., "Phase III Test Report Mix Acceptance Testing for Crystal River Unit 3 Steam Generator Replacement Project," June 19, 2009.

7. Florida Power Calculation S-95-0082, "6th Tendon Surveillance - Generation of Tendon Force Curves," Revision 3.

8. Progress Energy Calculation S06-0004, "Containment Shell Analysis for Steam Generator Replacement - Properties of New Concrete for Access Opening and Number of Hoop and Vertical Tendons to be De-tensioned," Revision 0.

9. Florida Power Corporation Drawing SC-421-031, "Reactor Building Exterior Wall - Concrete Outline," Rev. 4.

10. Crystal River Unit 3 Final Safety Analysis Report, Rev. 31.3.

11. Gilbert Associates Inc. Calculation 1.01.8.

12. American Concrete Institute Standard ACI 209R-92, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures," 1992.

13. Prescon DWG 5EX7-003, Sheet P9, "Hoop Tendon Placement 60°-120°, EL. 94'-5 3/4" - 143'-9 3/4"", Rev. 4.

14. Progress Energy Calculation S08-0008, "Containment Shell Analysis for Steam Generator Replacement - Evaluation of Restored Shell at 60-year Design Life," Revision 1.

15. Gilbert Associates Drawing SC-421-003, "Reactor Building Foundation Mat Concrete Outline," Revision 1.

16. MPR Calculation 0102-0135-02, "Concrete Modulus of Elasticity and Minimum Compressive Strength."

17. Relative Humidity Data from January 2004 to September 2006 at Brooksville, FL, University of Florida IFAS Extension, Florida Automated Weather Network, fawn.ifas.ufl.edu.

18. Not Used

19. Not Used

20. Florida Power Corporation Drawing S-425-007, "IWE/IWL Inspection Hoop Tendon "53" Layout," Revision 1.

21. Florida Power Corporation Drawing S-425-006, "IWE/IWL Inspection Hoop Tendon "42" Layout," Revision 1.

22. USNRC Regulatory Guide 1.35.1, "Determining Prestressing Forces for Inspection of Prestressed Concrete Containments," July 1990.

23. Email from J. Holliday (Progress Energy) to K. Gantz (MPR), Subj: FW: Temperature Effect on Creep, January 7, 2010, 6:14 AM (Provided as Attachment A).

24. Prescon Drawing 5EX7-003, Sheet P9, "Hoop Tendon Placement 60°-120° El. 94'-5 3/4" - 143' - 9 3/4"," Revision 4.

A

Reference 23

From: Holliday, John [John.Holliday@pgnmail.com]
Sent: Thursday, January 07, 2010 6:14 AM
To: Bird, Edward; Gantz, Kevin
Cc: Dyksterhouse, Don; Knott, Ronald
Subject: FW: Temperature Effect on Creep

Gentlemen,

Attached e-mail is from Prof. Domingo Carreira the Chairman of the sub-committee that prepared ACI 209 and specifically authored the section that addresses the effects of temperature on creep, he is also the individual who designed the concrete mix for CR3 SGR. Based on his observations I believe we can exclude operating temperature as a factor in determining the creep ratio.

Regards,

John Holliday

From: CHRIS.A.SWARD@sargentlundy.com [mailto:CHRIS.A.SWARD@sargentlundy.com]
Sent: Wednesday, January 06, 2010 1:10 PM
To: Holliday, John
Subject: Fw: Temperature Effect on Creep

John,

Domingo's reply follows. I think some of his discussion relates only to the patch concrete but in general I think he provides enough basis for not applying a temperature adjustment.

Chris Sward
Project Manager
Sargent & Lundy
312-269-7426

----- Forwarded by CHRIS A SWARD/Sargentlundy on 01/06/2010 12:08 PM -----

From:	DOMINGO CARREIRA <domingocarreira@sbcglobal.net>
To:	CHRIS.A.SWARD@sargentlundy.com
Date:	01/05/2010 11:21 PM
Subject:	Re: Temperature Effect on Creep

Chris:

Testing may recollection is a little bit risky, however the question is on a subject that I am familiar with, plus it was a good idea to send me the CR3 report to refresh what we did in 2007.

As you well said, ACI 209 Report discuss briefly the subject of the temperature effects on creep and shrinkage and gives some estimates but no factor is given to quantify it. I must confess that I personally wrote this portion of the report at the request of the late Jim Rhodes.

The same limitation on the effects of temperature on creep and shrinkage occurs with the other 3 methods of predictions given in the latest revision of ACI 209-2R recently published. The reason is the same for the four methods in ACI 209-2R, we don't have enough information to evaluate it and to propose an acceptable coefficient for correction. In addition, we say in the introduction of ACI 209-2R that a departure of + or - 30% from actual test data could be expected when using the proposed our methods. This sad admission was approved by the authors of the other 3 methods in ACI 209-3R, that is, Bazant, Gadner, and Muller. Branson the author of the original 209 method is no longer a member of this committee, he retired some years ago.

In the case of CR3 concrete replacement I am of the opinion that temperatures higher than 70 F will not be of concern for the following reasons:

1. Despite the temperature of the concrete during operating conditions as well as the exterior temperature in Florida will be higher than 70F, this higher temperature will not increase significally concrete creep and shrinkage, since their values from the standard testing temperature are very low compared with the majority of the concrete on which the prediction methods are based on.

2. The operating temperature will be by far lower than the initial accelerated autogenous curing temperature from the cement heat of hydration. This high autogenous temperature is not present in the standard testing methods for creep and shrinkage.

3. Most of the effect of high operating temperatures on creep and shrinkage is caused by the driving out of the concrete the water uncombined with cement. Approximately

the mass of water corresponding to 20% of the mass of cement will combine with it. That is, a w/c ratio of 0.20 will be chemically combined with the cement. The remaining of the mixing water may evaporate from the concrete This problem is drastically reduced by the fact that in our concrete the free evaporable water is low compared with most concretes. Also, and most important, by the very high volume-to-surface ratio of the walls (48 inches) compared with that of the test specimens (3 inches), and by the use of fly ash and silica fume that will combine chemically with some portion of the evaporable water that will not chemically combine with cement.

4. The high modulus of elasticity and the high initial strength of the CR3 concrete mixture are conditions that help to reduce the effects of temperatures higher than the testing temperatures. We know that some of the high strength concretes have lower creep and shrinkage than normal strength concretes because of the lesser free water in these concretes.

5. The higher operating temperatures will mostly affect the top portion of the containment away from the replacement concrete.

I could continue elaborating on this subject, but I think that the given reasons make sense..

I will return to Chicago from California tomorrow January 6, 2010 and could visit you the coming Thursday or Friday.

My best wishes in this 2010, Domingo

From: "CHRIS.A.SWARD@sargentlundy.com" <CHRIS.A.SWARD@sargentlundy.com>
To: Domingo Carreira <carreira@iit.edu>; domingocarreira@sbcglobal.net
Sent: Tue, January 5, 2010 10:55:08 AM
Subject: Temperature Effect on Creep

Domingo,
Happy New Year.

I need to test your recollection. The attached study was included with one of the calcs that we did for the CR3 containment analysis. Part of the study works through the computation of effective modulus based on creep. The creep coefficient computation (following ACI-209R) applies a number of adjustments for nonstandard conditions. ACI 209R discusses temperature as a factor although it does not provide a specific adjustment factor. Our temperature during operation will be somewhat above the standard 70 degF. Do you recall why we did not include a temperature adjustment?

Chris Sward
Project Manager
Sargent & Lundy
312-269-7426

20 pages

MPR	MPR Associates, Inc. 320 King Street Alexandria, VA 22314

CALCULATION TITLE PAGE

Client: Progress Energy	Page 1 of 20
Project: CR3 Containment Delamination	Task No. 0102-0906-0135
Title: Finite Element Model Description	Calculation No. 0102-0135-04

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
[signature] 1-21-2010 Peter Barrett	Kevin Gantz 1-21-2010 Kevin Gantz	E. B. Bird 1-21-2010 Edward Bird	0

QUALITY ASSURANCE DOCUMENT

This document has been prepared, checked, and reviewed/approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B, as specified in the MPR Quality Assurance Manual.

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0102-0135-04	[signature]	Kevin [signature]	

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

Table of Contents

1.0 INTRODUCTION

1.1 Background

A project is underway at Progress Energy's Crystal River Unit 3 (CR3) site to replace the steam generators. As part of that project, 10 vertical and 17 horizontal tendons were detensioned and an opening was cut into the concrete containment above the equipment hatch. As this opening was being cut, cracking in the concrete wall was identified around the full periphery of the opening in the cylindrical plane of the wall. The cracking is located at the approximate radius of the circumferential tendon conduits, and is indicative of a delaminated condition. Progress Energy plans to remove the delaminated concrete and replace it.

1.2 Purpose

This calculation documents an ANSYS finite element model of the Crystal River Unit 3 (CR3) Containment Building. The model was developed to analyze containment restoration and design basis loading conditions. Limited results from the model are provided for benchmarking. Results of repair and design basis analyses performed with the model, including the detensioning sequence, are documented elsewhere.

1.3 Reactor Building Description

Reference 1, Chapter 5.2, provides the following description of the Crystal River Containment. The CR3 Reactor Building is a concrete structure with a cylindrical wall, a flat foundation mat, and a shallow dome roof. The foundation slab is reinforced with conventional mild-steel reinforcing. The cylindrical wall is prestressed with a post-tensioning system in the vertical and horizontal (hoop) directions. The dome roof is prestressed utilizing a three-way post-tensioning system. The inside surface of the reactor building is lined with a carbon steel liner to ensure a high degree of leak tightness during operating and accident conditions. Nominal liner plate thickness is 3/8 inch for the cylinder and dome and 1/4 inch for the base. (Note that the liner plate is thicker around the equipment hatch.)

The foundation mat is 12-1/2 feet thick with a 2 foot thick concrete slab above the bottom liner plate. The cylindrical portion of the containment building has an inside diameter of 130 feet, wall thickness of 3 feet 6 inches, and a height of 157 feet from the top of the foundation mat to the spring line. The shallow dome roof has a major radius of 110 feet, a transition radius of 20 feet 6 inches, and a thickness of 3 feet.

2.0 SUMMARY OF RESULTS AND CONCLUSIONS

This calculation documents the development of the CR3 Containment finite element model for restoration and design basis analyses. The benchmarking results provided in Section 5 show a favorable comparison between the finite element membrane stresses and a hand calculation of membrane stresses for the intact containment.

3.0 METHODOLOGY

A three-dimensional finite element model is developed for the CR3 containment restoration and design basis analyses. The model includes linear-elastic material behavior with the exception of the steel liner which is modeled as elastic-plastic. The effects of concrete creep on prestress are represented in the finite element model by a reduction of tendon tension through time (Reference 7). Concrete creep strains are not considered in this calculation.

3.1 Finite Element Model Description

The finite element model of the Crystal River 3 Containment for restoration and design basis analyses includes the following features:

- The model represents a symmetric portion of the building (180°) with the symmetry plane passing through the center of the steam generator replacement opening and center of the equipment hatch.
- The hoop and vertical tendons are modeled explicitly.
- The equipment hatch is modeled with a simplified representation.
- The model has the ability to remove individual tendons (hoop or vertical) and has the ability to vary an individual tendon's force (hoop or vertical).
- The prestress from the dome tendons is modeled using equivalent forces.
- The delaminated portion of concrete on the containment wall is explicitly modeled as well as the concrete that is still intact.

The following finite element types are used in the model:

1. 3-D, 8 node brick elements are used to model the concrete building.

2. 1-D truss elements are used to model the tendons.

3. 3-D Shell elements are used to model the steel liner.

4. 1-D spring elements are used to link the boundary between the concrete added to fill the steam generator opening and the containment wall as well as the boundary between the delaminated concrete and the intact concrete in the plane of the cylindrical wall. The

stiffness of these elements is varied to represent the delamination or continuous bond of the intact and repaired building

5. Surface-to-surface contact elements are used to model the delamination stage in the containment wall. Contact elements are also used to bond the SGR plug to the existing concrete.

These elements are discussed in more detail below.

3.1.1 Containment Wall

Brick elements are used to model the containment wall since they can predict a nonlinear through-thickness stress distribution that cannot be captured using conventional shell modeling. Using the element birth and death features of ANSYS, these brick elements can accurately represent the incompatibility of the stress-free concrete used for repairs and the pre-loaded building deformation pattern.

The cylindrical portion of the wall is modeled as 42-inch thick concrete, with the exception of the wall that contains the opening for the steam generator replacement. This portion of the wall is modeled in two separate sections, a 10-inch thick delaminated portion on the outside surface of the wall, and the remaining intact 32-inch thick portion of the wall. The portion of the wall that is modeled as delaminated is the area bounded laterally by the two adjacent buttresses, and vertically by the transition to a 42-inch thick wall above the equipment hatch and a horizontal line at elevation 240 ft (approximately 10 feet below the bottom of the ring girder). This rectangular area surrounds the opening used for steam generator replacement and is somewhat greater than the actual delaminated area. 1-D springs are added to the interface surfaces of the delamination to either free the delamination or bond the delamination to the intact concrete, depending on the intent of the analysis. For the load steps including delamination, very soft springs eliminate tensile load transfer across this boundary.

The area in the containment wall that was removed to form an opening for steam generator replacement is modeled using independent elements which have coincident nodes with the edges of the containment. Prior to removal of the section, the model uses stiff springs to bond the elements to the containment wall. Element birth and death is used to kill the elements in the opening simulating the plug being cut. The plug region remains in the model but carries no stiffness or loads and when replaced appears as stress and strain-free material. After the tendons around the opening are detensioned and the new concrete is installed, the springs at the interface are eliminated (set to a negligibly small stiffness) and contact elements are used to bond the interface surfaces. A similar technique is applied for the delaminated concrete.

Brick element edges are aligned with the tendons such that the tendon (truss) element nodes are coincident with the containment (brick) concrete element nodes. These coincident nodes allow for direct coupling between the concrete and tendon elements in the two directions normal to the tendon. The truss elements are described in more detail below.

Figure 3-1 shows the 180° model. The buttresses are modeled with brick elements to capture their eccentric stiffness and to provide tendon attachment points. The basic dimensions of the containment model are presented in Section 4.1. The personnel hatch and other localized geometry, with the exception of the equipment hatch, were not modeled since they are remote from the steam generator opening. A scoping submodeling analysis of the equipment hatch showed that the hatch modeling shown below is adequate for performing repair and design basis analyses. The regions remote from the opening are unaffected by the steam generator replacement; their presence will not affect the global model results near the SGR opening and delamination.

3.1.2 Ring Girder and Dome

In the finite element model, the ring girder and dome are represented by uniform areas swept about the vertical axis of the containment. This representation is exact for the dome and nearly exact for the ring girder. The dome and ring girder elements are joined by constraint equations rather than by shared nodes. The dome delamination and repair are considered to have a negligible effect on the purpose of this calculation and therefore are not represented in the finite element model. All of the dome tendons are considered to be fully tensioned.

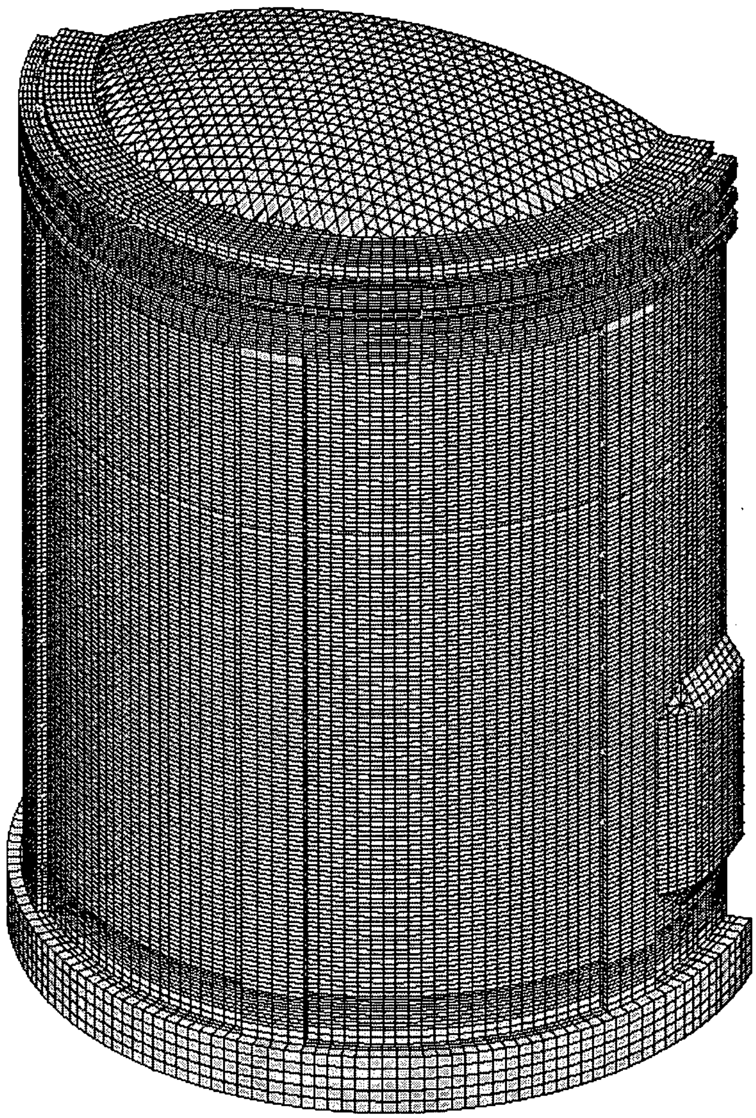


Figure 3-1 Finite Element Model of CR3 Containment Building

3.1.3 Tendons

Truss elements are used to model the vertical and hoop tendons to provide flexibility in evaluating variations in tendon loads (de-tensioning and re-tensioning) during the repair process. Hoop tendon truss element nodes are defined at coincident locations of the brick elements of the containment wall where load transfer is required between hoop tendons and the containment wall. Vertical tendons are each modeled as a single truss element with nodes at the top of the ring girder and at the bottom of the basemat. Rigid beam elements are used at the buttresses for the hoop tendons, and at the top of the ring girder and bottom of the basemat for the vertical tendons to connect the ends of the tendons to the containment. This modeling distributes the tendon support loads to the concrete brick elements without modeling the anchorages explicitly. Coupling in the radial and vertical directions between the tendon elements and the containment wall is used to transfer load between the hoop tendons and the containment wall. The axial degrees of freedom of the tendons are fixed, but are not tied to the containment wall. The fixed axial displacement allows for an initial strain to be used to define the tendon forces in these elements. Forces are derived directly from the stresses and tendon areas. However since the building deformation effects the stress, the strain required to define the tendon forces requires an iterative approach to ensure the proper tendon force is applied. Thus, each element is given a different initial strain to produce the current tendon loads. Tendon de-tensioning and future re-tensioning is performed by scaling these strains.

Table 4-2 provides basic tendon spacing. There are 144 evenly spaced vertical tendons (2.5 degree spacing). There are 94 tendon hoops, each hoop consisting of three individual tendons. The hoop tendons are arranged in pairs. The two tendons in the pair are separated by 12.75 inches (typically) whereas pairs are typically separated by 38.12 inches (Reference 12).

Tendons are initially tensioned to 80% of Guaranteed Ultimate Tensile Strength (GUTS) and then the load is reduced to 70% of GUTS. For horizontal tendons, this procedure results in a tendon force curve that is best represented by a uniform tendon tension along the length of the tendon. Consequently, a uniform tension was applied to the horizontal tendons (Reference 9). The tension applied accounts for loss of tension through time (Reference 7).

Vertical tendons only transfer load between the tendon and containment wall at the anchorages. The vertical tendon loads are defined using initial strains similar to the hoop tendons. The strains are adjusted via an iterative approach to account for the building stiffness. During tendon de-tensioning, adjacent tendons that are not de-tensioned automatically capture the additional forces caused by load re-distribution. The re-distribution of load also occurs during the de-tensioning of hoop tendons.

Tendon material and structural properties are defined below. Figure 3-2 illustrates the vertical and hoop tendons in the model.

The dome tendons are modeled in a similar manner as the vertical and hoop tendons, but since there is no detensioning required, the dome tendons are removed in the final model with prestress applied to the dome using equivalent forces. The dome tendons are modeled with an independent truss element mesh with coincident nodes aligned with the dome brick elements. In the process of constructing the model, these independent nodes are constrained in all directions and the tendon preload is applied using initial strains as described above. Reaction forces are calculated at all of the common nodes, and these forces are explicitly applied to the dome elements. The dome tendon truss elements are then removed. The dome tendon ring girder forces are distributed to the concrete elements via stiff beams. Modeling the dome tendons explicitly is not necessary since these tendons will not be detensioned. Dome tendon forces are adjusted to account for loss of tendon tension due to aging phenomenon (e.g. concrete creep) in a manner analogous to the process for the hoop and vertical tendons (Reference 7).

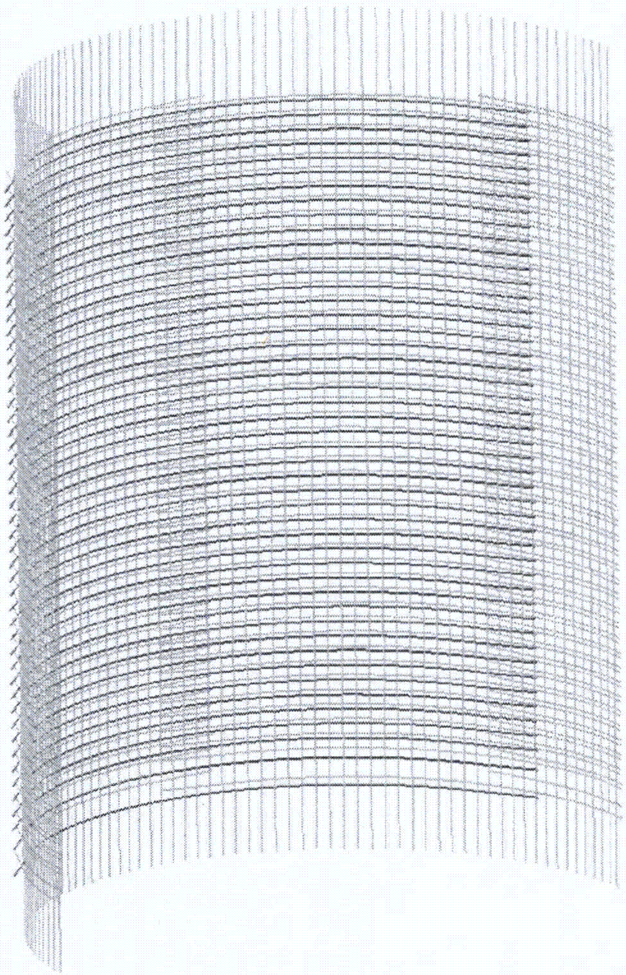


Figure 3-2 Hoop and Vertical Tendons for the 180˚ Model

3.1.4 Liner

The liner is included in the model to account for the structural interaction between it and the concrete containment. The liner plate is modeled as a single layer of four-node shell elements on the inside face of the containment building. The liner is modeled as ⅜-inch thick on the inside surface of the cylindrical portion and dome and ¼-inch thick on the bottom surface of containment (Reference 2, page 34). The liner plate thickness is increased to 1.125 inches around the equipment hatch.

3.2 Boundary Conditions

Displacement boundary conditions are defined to prevent rigid body motion of the containment building and to simulate the reflected portion of the building modeled with the symmetry plane. The vertical support of the building is modeled as an elastic foundation.

Symmetry boundary conditions are applied to constrain all nodes at the centerline of the model to have zero displacement in the normal (global z) direction. For the tendon nodes that have been rotated into a cylindrical coordinate system the symmetry constraint is applied to the local hoop or y direction.

A single point at the center of the foundation is constrained in the lateral "x" direction to prevent rigid body motion. This does not prevent rocking type motion that would occur in the building and the reaction force at this node is negligible.

Vertical support of the building is achieved using an elastic foundation. The elastic foundation stiffness is defined using a layer of surface effect elements placed under the basemat. The foundation stiffness defined in the model is 395 lbs per cubic inch (680 kips per cubic foot) (Reference 1, Figure 5-20).

4.0 DESIGN INPUT

The design input used to develop the finite element model is provided below.

4.1 Geometry

The key dimensions used to model the CR3 containment are listed in Tables 4-1 and 4-2.

Table 4-1 Key Containment Concrete Dimensions

Dimension	Value	Reference
Containment Concrete ID	130 ft 0.75 in	Reference 10
Containment Wall Thickness (excluding buttresses)	3 ft 6 in	Reference 10
Basemat Thickness	12 ft 6 in	Reference 11
Basemat OD	147 ft 0.75 in	Reference 10
Dome Radius of Curvature (Cyl. To Dome Transition)	20 ft 6.375 in	Reference 10
Dome Radius of Curvature (Dome Middle)	110 ft 0.375 in	Reference 10
Dome Thickness	3 ft	Reference 10

Dimension	Value	Reference
Ring Girder Vertical Thickness	16 ft 4 in	Reference 10
Ring Girder OD	141 ft 8.75 in	Reference 10
Height (Top of Basemat to Springline)	157 ft	Reference 10
Buttress Wall Thickness	5 ft 10 in	Reference 10
Buttress Height (Top of Basemat to Bottom of Ring Girder)	158 ft 2 in	Reference 10
Steam Generator Opening Height	27 ft	Reference 12
Steam Generator Opening Width	25 ft	Reference 12
Top of Basemat to Bottom of Opening	90 ft	References 11 and 12
Top of Basemat to Equipment Hatch Centerline	39 ft	Reference 10
Equipment Hatch Opening IR[1]	11 ft 4.5 in	Reference 10
Equipment Hatch Centerline Vert. Distance to 3.5 ft Thick Cyl. Wall	25 ft 10 in	Reference 10
Transition Radius of Curvature from Cyl. To Basemat	20 ft 0.375 in	Reference 10
Slab Thickness	2 ft	Reference 10

Note 1: The equipment hatch is modeled as a square opening with an equivalent area of the circular opening prescribed in the table.

Table 4-2 Miscellaneous Component Dimensions

Dimension	Value	Reference
Hoop Conduit Placement Radius[1]	67 ft 8.375 in	Reference 2, Page 14
Vertical Conduit Placement Radius	67 ft 3.375 in	Reference 2, Page 14
Tendon total area (163 wires)	9.723 in^2	Reference 2, Page 6
Nominal Liner Thickness, Excluding Base	0.375 in	Reference 10
Liner Thickness Near Equipment Hatch	1.125 in	Estimated from Reference 10
Base Liner Thickness	0.25 in	Reference 10
Number of Vertical Tendons	144	Reference 2, Page 14
Number of Tendon Hoops	94	Reference 2, Page 14

Dimension	Value	Reference
Number of Tendons per Hoop	3	Reference 2, Page 14
Total Number of Hoop Tendons	282	Calculated from Reference 2, Page 15
Number of Prestressed Dome Tendons	123	Reference 2, Page 14

Note 1: The hoop conduit placement radius is listed as 67 ft 8.625 on Prescon DWG P10-A. The difference in placement radius between the DBD (Reference 2) and the Prescon drawing is less than 1% of the total wall thickness and is less than 5% of the conduit diameter. The difference in results for the global model is judged to be insignificant.

4.2 Material Properties

The linear elastic material properties used in the finite element model are elastic modulus, density and Poisson's ratio. There is a unique elastic modulus applied to concrete that has existed for the entire life of the plant and for concrete that is used to replace the delamination and the SGR opening. Concrete properties are listed below.

Elastic modulus

Existing Concrete	4.03×10^6 psi	Reference 3, page 4
Replacement Concrete	5.12×10^6 psi	Reference 3, page 4

Poisson's Ratio

All Concrete	0.2	Reference 2, page 3

Density

All Concrete	150 lb/ft^3	Reference 2, page 3

Thermal Expansion Coefficient

All Concrete	4.25×10^{-6} in/in/°F	Reference 6, Table 2.2.38

The liner is made of ASTM A283 Grade C carbon steel with a minimum yield strength of 30.0 ksi (Reference 2, page 34). The tendon wire in all post-tensioning conduit is ASTM A421-65 steel with a yield strength of 240 ksi (Reference 2, page 5). The typical density, stiffness, and Poisson's ratio of steel are used for these materials, taken from Reference 4, Table 38. The coefficient of thermal expansion is taken from Reference 5, Table TE-1 and is only applied to the liner.

Elastic modulus	29×10^6 psi	Reference 4, Table 38
Poisson's ratio	0.27	Reference 4, Table 38
Density	0.283 lb/in^3	Reference 4, Table 38
Thermal Expansion Coefficient (Avg. from 70°F to 281°F)	6.83×10^{-6} in/in/°F	Reference 5, Table TE-1
Minimum Yield Strength		
Liner	30 ksi	Reference 2 page 34
Tendon Wire	240 ksi	Reference 2, page 5

The yield strength of the liner is incorporated directly into the liner material properties in the model so that if it becomes overstressed, the liner will yield and relieve itself of load. The yield point of the material is modeled as 1.2 times the minimum yield strength (Reference 2, page 26).

5.0 Model Benchmarking Results

To benchmark the finite element model, stress results for the intact containment model considering 95% of the deadweight plus tendon preload (1474 kips for the vertical tendons and 1398 kips for the hoop tendons) are compared to hand calculations. The linearized hoop and vertical membrane stresses were obtained at the SGR opening mid-height elevation. Figures 5-1 and 5-2 show color contour plots of hoop and vertical stress respectively. The linearized stresses are tabulated below.

Hoop membrane stress: 1630 psi

Vertical membrane stress: 977 psi

A hand calculation of hoop and vertical stress is provided below for comparison. The hand calculated hoop stress is 1560 psi; the hand calculated vertical stress is 957 psi. The hand calculated hoop stress is within 5% of the finite element result; the hand calculated vertical stress is within 3% of the finite element result.

$h := 157 \cdot ft$ Containment height

$r_i := 65 \cdot ft + \frac{3}{8} \cdot in$ Containment concrete inside radius (Reference 8)

$r_i = 65.031 ft$

$r_o := r_i + 42 \cdot in$ Containment concrete outside radius (Reference 8)

$r_o = 68.531 ft$

$t_b := 28 \cdot in$ Buttress thickness (Reference 8)

$L_b := 12 \cdot ft + 4.125 \cdot in$ Average buttress width (Reference 8)

$N_b := 6$ Number of buttresses (Reference 8)

$N_v := 144$ Number of vertical tendons (Reference 2, page 14)

$N_h := 94$ Number of hoop tendons (282 total / 3 per loop = 94 loops, Reference 2, page 14)

$T_v := 1474000 \cdot lbf$ Vertical tendon tension (Reference 7, page 5, unadjusted tendon)

$T_h := 1398000 \cdot lbf$ Hoop tendon tension (Reference 7, page 5, unadjusted tendon)

$\rho_c := 150 \cdot \frac{lbf}{ft^3}$ Concrete density (Reference 2, page 3)

$t_{dome} := 3 \cdot ft$ Dome thickness (Reference 2, page 1)

$h_{rg} := 16 \cdot ft + 4 \cdot in$ Ring girder height (Table 4-1, above)

$h_{sgro} := 90 \cdot ft + \frac{27}{2} \cdot ft$ Mid-height of the SGR opening (Table 4-1, above)

$h_{sgro} = 103.5 ft$

$t_{liner} := \frac{3}{8} \cdot in$ Liner thickness (Reference 10)

$E_c := 4.03 \cdot 10^6 \cdot psi$ Concrete elastic modulus (Reference 3, page 4)

$E_l := 29 \cdot 10^6 \cdot psi$ Liner elastic modulus (Reference 4, Table 38)

The approximate concrete area of a vertical section through the full height of the containment wall is calculated below:

$a_h := h \cdot (r_o - r_i)$ $a_h = 549.5 ft^2$

The approximate steel liner area of a vertical section through the full height of the containment wall is calculated below:

$a_{lh} := h \cdot t_{liner}$ $\qquad a_{lh} = 4.906\,ft^2$

The average hoop stress in the containment wall is calculated below considering the effect of the liner:

$$\sigma_h := \frac{N_h \cdot T_h}{a_h} \cdot \frac{a_h \cdot E_c}{a_h \cdot E_c + a_{lh} \cdot E_l} \qquad \sigma_h = 1560\,psi$$

The area of a horizontal section through the containment is calculated below. The area contribution of the buttresses is included. The area contribution of the vertical conduits is not deducted because the conduits are not represented in the finite element model.

$a_a := \pi \cdot \left(r_o^{\,2} - r_i^{\,2}\right) + N_b \cdot L_b \cdot t_b$ $\qquad a_a = 1641\,ft^2$

The average vertical stress due to tendon tension is calculated below considering the effect of the liner:

The approximate steel liner area of a vertical section through the full height of the containment wall is calculated below:

$a_{lv} := 2 \cdot \pi \cdot r_i \cdot t_{liner}$ $\qquad a_{lv} = 12.769\,ft^2$

$$\sigma_a := \frac{N_v \cdot T_v}{a_a} \cdot \frac{a_a \cdot E_c}{a_a \cdot E_c + a_{lv} \cdot E_l} \qquad \sigma_a = 850\,psi$$

The deadweight of the concrete above the mid-height of the SGR opening is estimated below. The buttress is approximated by a rectangular section, the dome is approximated by a flat disc and the ring girder is approximated as a cylindrical section. (Note that the deadweight is a small contribution to the vertical stress. Consequently, these approximations are considered acceptable.)

$W_{shell} := \rho_c \cdot \left(h - h_{sgro}\right) \cdot a_a$ $\qquad W_{shell} = 13.17 \times 10^6\,lbf$

$W_{rg} := \rho_c \cdot \pi \cdot h_{rg} \cdot \left[\left(r_o + t_b\right)^2 - r_i^{\,2}\right]$ $\qquad W_{rg} = 6.102 \times 10^6\,lbf$

$W_{dome} := \rho_c \cdot t_{dome} \cdot \pi \cdot r_i^{\,2}$ $\qquad W_{dome} = 5.979 \times 10^6\,lbf$

The average vertical stress due to deadweight of the concrete above the mid-height of the SGR opening is estimated below

$$\sigma_{dw} := \frac{W_{shell} + W_{rg} + W_{dome}}{a_a} \qquad \sigma_{dw} = 107\,psi$$

The total vertical stress due to tendon tension and deadweight at the SGR opening mid-height is calculated below:

$\sigma_{a\_tot} := \sigma_a + \sigma_{dw}$ $\qquad \sigma_{a\_tot} = 957\,psi$

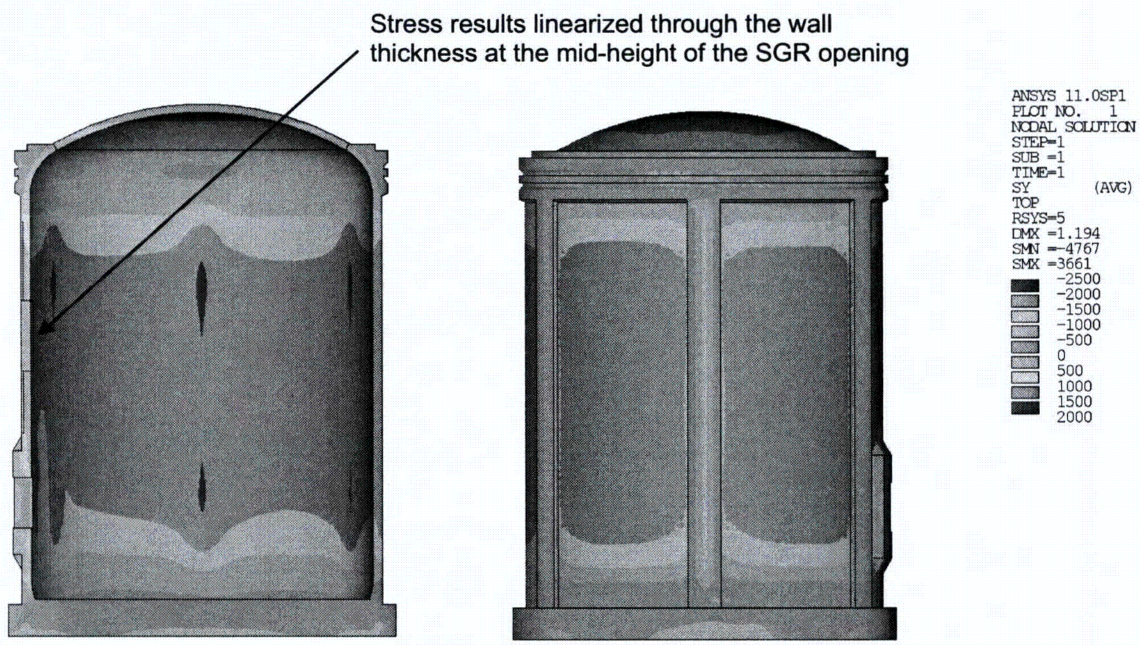


Figure 5-1 Hoop Stress

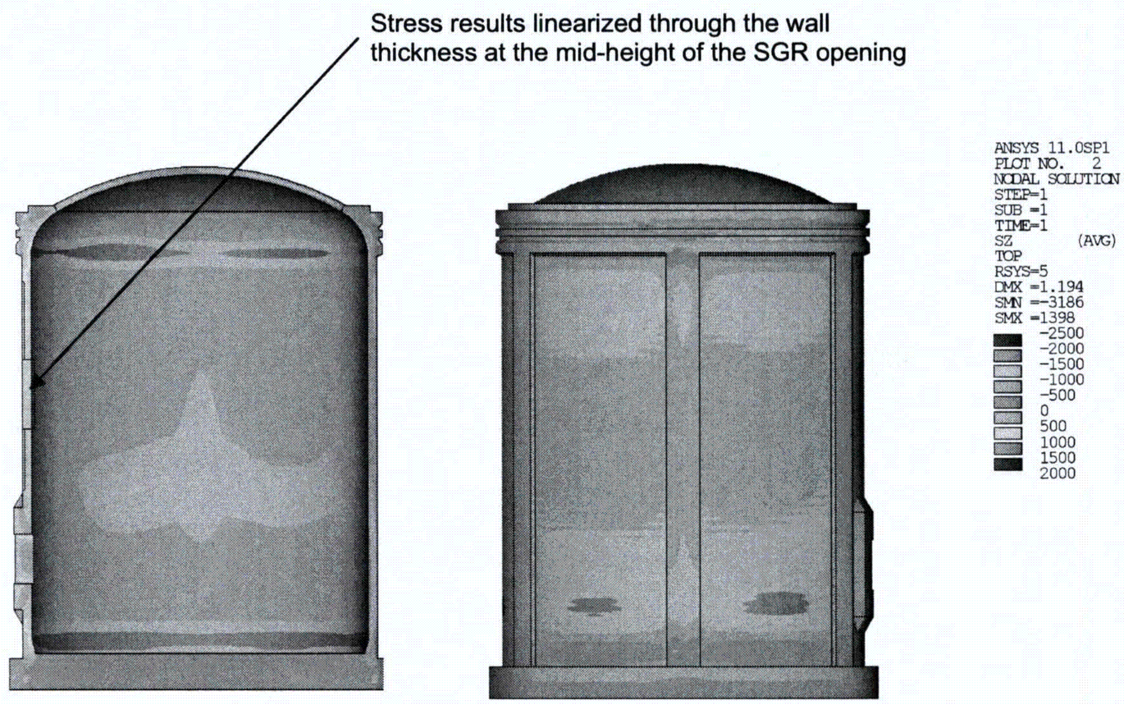


Figure 5-2 Vertical Stress

6.0 ASSUMPTIONS

1. The tendons are assumed to be symmetric about the 150 degree azimuth through the center of the SGR opening. This assumption is reasonable because of the staggered design of the hoop tendons, the load application they apply to the building is nearly uniform radial compression which would make the loading symmetric about the centerline of each buttress. For the intact building cases, the response predicted in the finite element model is the same between each buttress set. Since the hatch and SGR opening are centered between buttresses 3 and 4, symmetry can be applied via the centerline of the model in this area.

7.0 COMPUTER CODES

This analysis was performed with the ANSYS general purpose finite element program, Version 11.0 SP1. The analysis was performed on a Sun v40z server running the Suse Linux 9.0 operating system. The ANSYS installation verification is documented in QA-110-1.

8.0 REFERENCES

1. Final Safety Analysis Report, Progress Energy Florida, Crystal River 3, Revision 31.3.

2. Progress Energy, "Design Basis Document for the Containment," Revision 7.

3. MPR Calculation 0102-0135-02, Rev. 0, "Concrete Modulus of Elasticity and Minimum Compressive Strength."

4. Roarke, Raymond J. and Warren C. Young, *Formulas for Stress and Strain*, 5th Ed., McGraw-Hill, 1975.

5. ASME Boiler and Pressure Vessel Code, Section II, Part D – Properties, 1992 Edition.

6. National Cooperative Highway Research Program, *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, March 2004.

7. MPR Calculation 0102-0135-03, Rev. 0, "Tendon Tension Calculation."

8. FPC DWG SC-421-031, Rev. 4, "Reactor Building, Exterior Wall – Concrete Outline."

9. CR3-LI-537934-31-SE-007, Revision B, Attachment C, January 6, 2010, DRAFT Follow-Up Input to Technical Issues Discussed at 3rd Party Review Meeting at MPR on December 8 & 9, 2009.

10. Drawing No. SC-421-031, "Reactor Building Exterior Wall Concrete Outline," Revision 4.

11. Drawing No. SC-421-003, "Reactor Building Foundation Mat Concrete Outline," Revision 4.

12. Drawing No. CR3 DWG 421-347, "Reactor Building Temporary Access Opening for SGR Vertical & Horizontal Tendon Positions," Revision 0.

13. Computer output file 0102-0135-04-1 and 0102-0135-04-2.

15 pages

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

CALCULATION TITLE PAGE

Client: Progress Energy	Page 1 of 15
Project: CR3 Containment Delamination	Task No. 0102-0906-0135
Title: Conduit Local Stress Analysis	Calculation No. 0102-0135-05

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
E. B. Bird 1-20-2010 Edward Bird	Erin Tindall 1-20-2010 Erin Tindall	Robert Keating 1-20-2010 Robert Keating	0

QUALITY ASSURANCE DOCUMENT

This document has been prepared, checked, and reviewed/approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B, as specified in the MPR Quality Assurance Manual.

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0102-0135-05	E. B. Snd	EC Tell	

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

Table of Contents

1.0 PURPOSE

1.1 Background

A project is underway at Progress Energy's Crystal River Unit 3 (CR3) site to replace the steam generators. As part of that project, 10 vertical and 17 horizontal tendons were detensioned and an opening was cut into the concrete containment above the equipment hatch. As this opening was being cut, cracking in the concrete wall was identified around the full periphery of the opening in the cylindrical plane of the wall. The cracking is located at the radius of the circumferential tensioning tendons, and is indicative of a delaminated condition. Progress Energy plans to remove the delaminated concrete and replace it.

1.2 Purpose

The concrete repair and restoration of the steam generator opening may require detensioning additional tendons. The purpose of this calculation is to determine if the absence of either the vertical or horizontal compressive load results in a more limiting stress condition around the tendon conduits than the case with both vertical and horizontal compression applied. If a more limiting stress condition is predicted for the case with either vertical load only or hoop load only, this calculation will provide a basis for the detensioning sequence.

A local axisymmetric finite element analysis of the hoop tendon conduits was performed to evaluate the principal stress magnitude and orientation around the hoop conduits for three combinations of vertical and hoop compression. The three cases are:

- Both vertical and hoop tendons tensioned
- Vertical tendons only tensioned
- Horizontal tendons only tensioned.

2.0 SUMMARY OF RESULTS AND CONCLUSIONS

Figures 7-1, 7-2 and 7-3 show color contour plots of maximum principal stress (S1) in the concrete for the three post-tension loading conditions evaluated. The maximum principal stress for the three cases is listed below:

- Horizontal + Vertical Tendon Load: 1,041 psi
- Vertical Tendon Load Only: 919 psi
- Horizontal Tendon Load Only: 237 psi

The results show that with either vertical only or horizontal only tendon loads, the maximum principal stress is less than the case with both loads applied simultaneously. Therefore, this calculation does not provide a basis for the detensioning sequence.

3.0 METHODOLOGY

An axisymmetric finite element model of the local geometry around the hoop tendons was developed with the Ansys finite element program. The axis of symmetry for the model is the vertical centerline of the containment. The model represents an un-delaminated section of the containment wall. Linear-elastic, static structural analyses were performed for three loading conditions.

Figure 3-1 shows the axisymmetric model developed for the local stress analysis. The model represents a vertical slice through the containment wall between vertical tendons and includes the liner and two conduits.

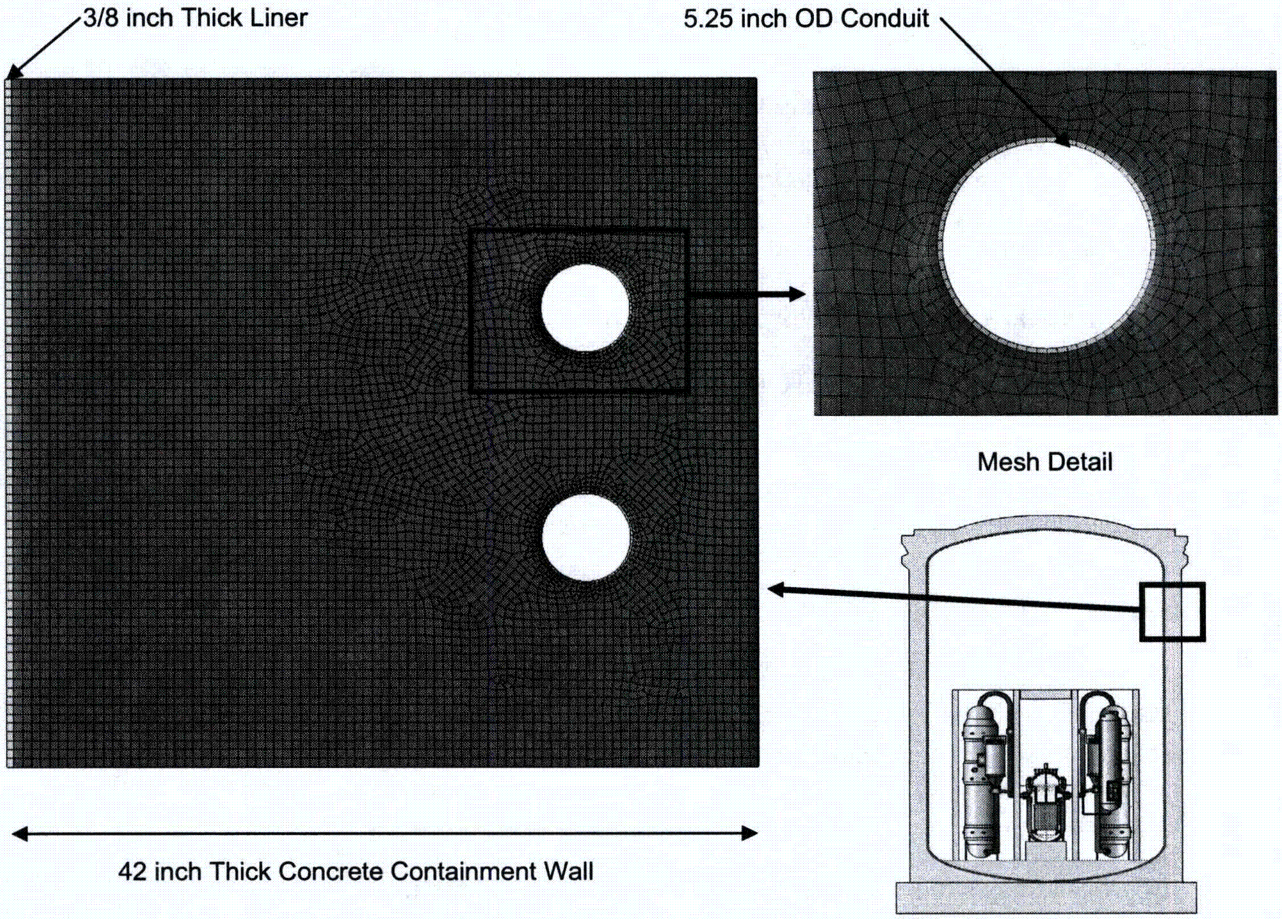


Figure 3-1 Hoop Conduit Axisymmetric Finite Element Model

4.0 DESIGN INPUTS

4.1 Geometry

The basic geometric parameters used for the model are listed in Table 4-1.

Table 4-1. Local Model Dimensions

Dimension	Value	Reference
Containment Liner Inside Radius	65 ft	Reference 1a and Reference 2, pg 35
Containment Wall Thickness	42 in	Reference 1a
Hoop Conduit OD	5.25 in	Reference 2, Page 4
Hoop Conduit ID	5.125 in	Assumption 1
Hoop Conduit Spacing	13 in	Reference 2, Page 14
Hoop Conduit Placement Radius	67 ft 8.625 in	Reference 1b
Liner Thickness, Excluding Base	0.375 in	Reference 1a

The model is 39 inches high, which represents the nominal distance between tendon pairs.

4.2 Material Properties

The linear elastic material properties used in the conduit local stress analysis are elastic modulus, density and Poisson's ratio. The values used for concrete are listed below:

Elastic Modulus:	4.03×10^{6} psi	Reference 3, page 4 (uncracked)
Density:	150 lb/ft^3	Reference 2, page 3
Poisson's ratio:	0.2	Reference 2, page 3

The liner is made of ASTM A283 Grade C carbon steel with a yield strength of 30.0 ksi (Reference 2 page 34). Typical values for the elastic modulus, density and Poisson's ratio are taken from Reference 4, Table 38.

Elastic Modulus:	29×10^{6} psi
Density:	0.283 lb/in^3
Poisson's ratio:	0.27

4.3 Boundary Conditions

The boundary conditions applied to the model include displacement restraints and applied forces that represent post-tension loads only. As shown in Figure 4-1, along the lower edge of the model, displacements of the concrete and liner normal to the edge are restrained. At the upper edge of the model, the concrete and liner displacements normal to the edge are coupled to one another such that all nodes have the same vertical displacement. This condition forces the upper edge of the model to remain horizontal and represents a symmetry condition across the edge. A pressure corresponding to the vertical compression load was applied at the upper edge.

Three hoop tendons, each spanning 120 degrees, form a complete 360 degree circle around the containment. In the axisymmetric model, at each tendon conduit, the tendon load is represented by the total (360 degree) radial load. For the case with vertical load only, both hoop tendons in the model are detensioned. The hoop tendon load and vertical pressure are calculated below.

Note that because the liner is explicitly included in the model with steel material properties, the prestress load is shared between the steel liner and concrete wall.

$r_i := 65 \cdot ft + \frac{3}{8} \cdot in$ Containment concrete inside radius (Reference 1a)

$r_i = 65.031\,ft$

$r_o := r_i + 42 \cdot in$ Containment concrete outside radius (Reference 1a)

$r_o = 68.531\,ft$

$t_b := 28 \cdot in$ Buttress thickness (Reference 1a)

$L_b := 12 \cdot ft$ Buttress length (Reference 1a)

$N_b := 6$ Number of buttresses (Reference 1a)

$N_v := 144$ Number of vertical tendons (Reference 2, page 14)

$d_c := 5.25 \cdot in$ Tendon conduit outside diameter (Reference 2, page 4)

$T_v := 1474000 \cdot lbf$ Vertical tendon tension (Reference 5, page 5, unadjusted tendon at the end of the SGR project, 33 years)

$T_h := 1398000 \cdot lbf$ Hoop tendon tension (Reference 5, page 5, unadjusted tendon at the end of the SGR project, 33 years)

The vertical tendon load is reacted by the cross section area of the containment wall and buttresses less the area of the vertical tendon conduits.

$$a_a := \pi \cdot \left(r_o^2 - r_i^2\right) + N_b \cdot L_b \cdot t_b - N_v \cdot \frac{\pi}{4} \cdot d_c^2 \qquad a_a = 1615\,ft^2$$

$$\sigma_a := \frac{N_v \cdot T_v}{a_a} \qquad \sigma_a = 913\,psi$$

Each hoop tendon has a tension of T_h and exerts a unit radial force of T_h / r on the containment. The Ansys code requires that the radial load be applied on a 360 degree basis. The total radial load is then (T_h / r) x 2 pi r = 2 pi T_h.

$$F_{hoop} := 2 \cdot \pi \cdot T_h \qquad F_{hoop} = 8.784 \times 10^6\,lbf$$

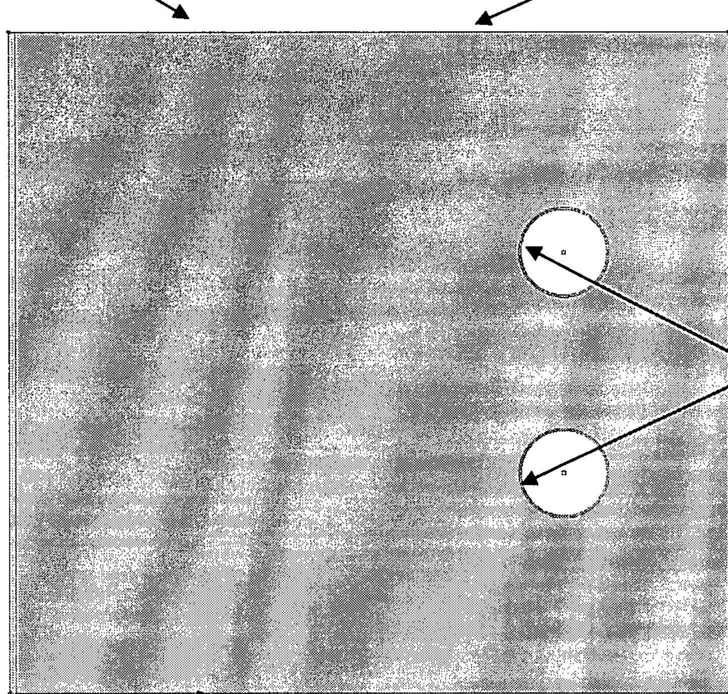


Figure 4-1 Local Conduit Model Boundary Conditions

5.0 ASSUMPTIONS

1. The DBD provides both a minimum wall thickness of 1/16 inch for the hoop conduits and an inside diameter of 5 inch which leads to a thickness of 1/8 inch (Reference 2, page 4). The conduit wall thickness used in the analysis is 1/16 inch.

6.0 Computer Codes

This analysis was performed with the ANSYS general purpose finite element program, Version 11.0 SP1. The analysis was performed on a Sun v40z server running the Suse Linux 9.0 operating system. The ANSYS installation verification is documented in QA-110-1.

7.0 Results

Figures 7-1, 7-2 and 7-3 show color contour plots of maximum principal stress (S1) in the concrete for the three post-tension loading conditions evaluated. Positive (+) stress values are tensile. The maximum principal stress for the three cases is listed below:

Horizontal + Vertical Tendon Load:	1,041 psi
Vertical Tendon Load Only:	919 psi
Horizontal Tendon Load Only:	237 psi

The results show that with either vertical only or horizontal only tendon loads, the maximum principal stress is less than the case with both loads applied simultaneously.

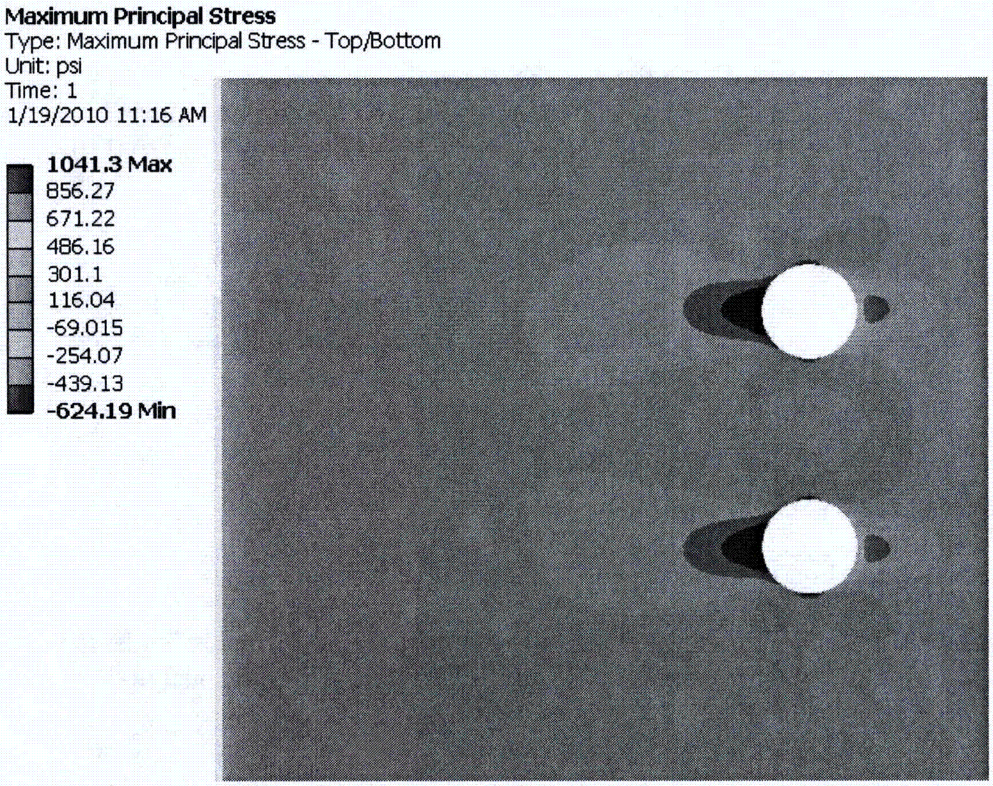


Figure 7-1. Concrete Maximum Principal Tensile Stress – Vertical + Horizontal

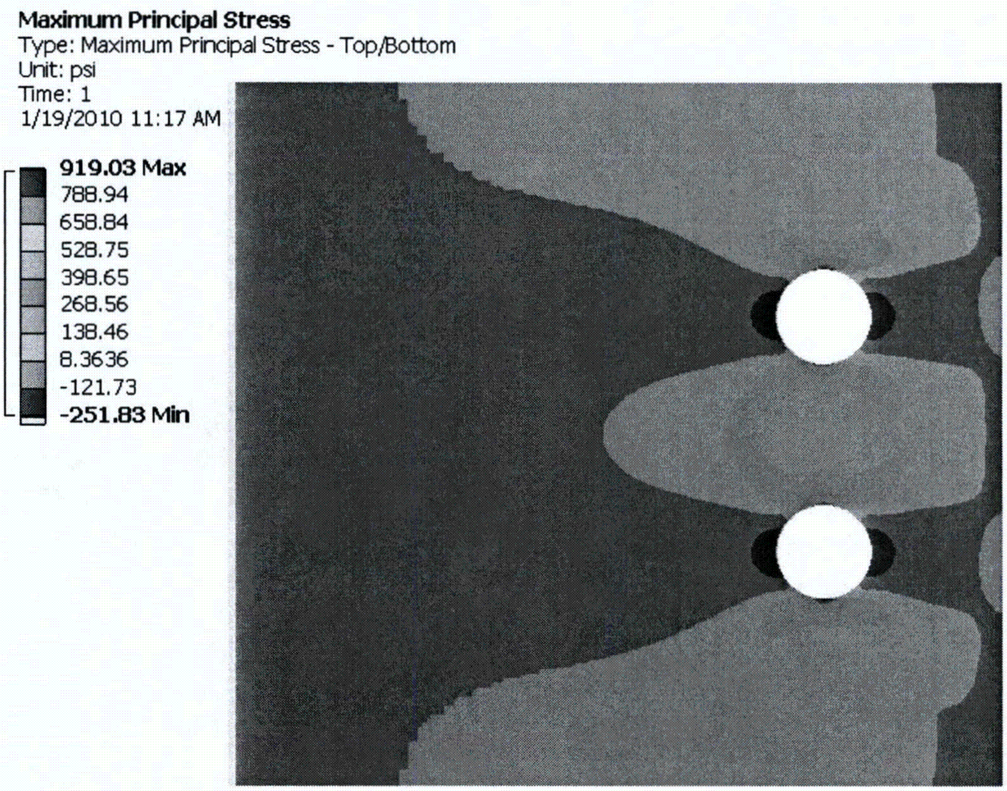


Figure 7-2. Concrete Maximum Principal Tensile Stress – Vertical Only

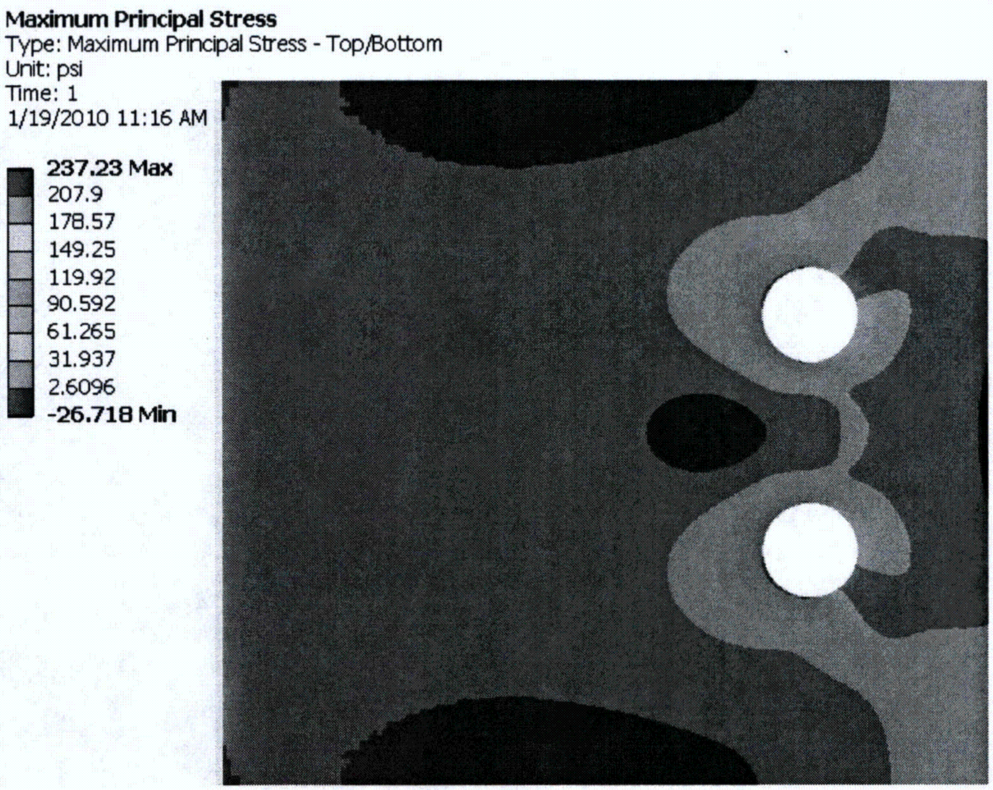


Figure 7-3. Concrete Maximum Principal Tensile Stress – Horizontal only

8.0 REFERENCES

1. Drawings:

    a. FPC DWG SC-421-031, Rev. 4, "Reactor Building, Exterior Wall – Concrete Outline.

    b. Prescon Drawing P10-A, Rev. 1, "Horizontal Tendon Detail Between 120° – 180°."

2. Progress Energy, "Design Basis Document for the Containment," Revision 7.

3. MPR Calculation 0102-0135-02, Rev. 0, "Concrete Modulus of Elasticity and Specified Compressive Strength."

4. Roarke, Raymond J. and Warren C. Young, "Formulas for Stress and Strain," 5th Ed., McGraw-Hill, 1975.

5. MPR Calculation 0102-0135-03, Rev. 0, "Tendon Tension Calculation."

6. Computer output files 0102-0135-05-1, 0102-0135-05-2 and 0102-0135-05-3.

25 pages

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

CALCULATION TITLE PAGE

Client: Progress Energy	Page 1 of 23 plus Attachment
Project: CR3 Containment Calculations	Task No. 0102-0906-0135
Title: Seismic, Wind, and Tornado Evaluation and Delamination Depth Evaluation for Detensioned State	Calculation No. 0102-0135-08

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
J. L. Hibbard J. L. Hibbard 1-23-2010	M. Oghbaei M. Oghbaei 1-23-2010	E. B. Bird E. Bird 1-23-2010	0

QUALITY ASSURANCE DOCUMENT

This document has been prepared, checked, and reviewed/approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B, as specified in the MPR Quality Assurance Manual.

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0102-0135-08	J. L. Hibbard	M. Oghbaei	

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

Table of Contents

1.0 PURPOSE

This calculation evaluates the containment building for three design basis loads due to natural phenomena that might occur while the containment building is detensioned for repair. The load cases are: 1) deadweight and Safe Shutdown Earthquake (SSE), 2) deadweight and wind, and 3) deadweight and tornado. The containment is evaluated for membrane plus bending stress at two sections through the containment: 1) the bottom of the containment at Elevation 93 ft, and 2) at the bottom of the SGR (Steam Generator Replacement) opening at Elevation 183 ft. For the evaluation at the bottom of the SGR opening, the containment is assumed to have no concrete between Buttresses 3 and 4 between Elevations 183 feet and 210 feet. These are the bottom and top elevations respectively, of the SGR opening.

2.0 SUMMARY

Membrane plus bending stress in the containment shell at two sections for two load cases are provided in the table below. The deadweight plus wind load case is bounded by the results for the deadweight plus tornado load case.

$$T_s = \begin{bmatrix} \text{"Load"} & \text{"Section"} & \text{"M + B"} & \text{"Stress"} & \text{"Result"} \\ \text{"Case"} & \text{""} & \text{"Stress"} & \text{"Limit"} & \text{""} \\ \text{""} & \text{""} & \text{"psi"} & \text{""} & \text{""} \\ \text{"Deadweight \& SSE"} & \begin{pmatrix} \text{"Bottom of Cont."} \\ \text{"Bottom of SGR Opening"} \end{pmatrix} & \begin{pmatrix} 139 \\ 19 \end{pmatrix} & 600 & \begin{pmatrix} \text{"No Failure"} \\ \text{"No Failure"} \end{pmatrix} \\ \text{"Deadweight \& Tornado"} & \begin{pmatrix} \text{"Bottom of Cont."} \\ \text{"Bottom of SGR Opening"} \end{pmatrix} & \begin{pmatrix} -103 \\ -95 \end{pmatrix} & 600 & \begin{pmatrix} \text{"No Failure"} \\ \text{"No Failure"} \end{pmatrix} \end{bmatrix}$$

Notes:

1. Column with heading M + B is the membrane plus bending stress. Plus is tensile and minus is compressive.
2. SSE is Safe Shutdown Earthquake.
3. SGR is Steam Generator Replacement
4. The stress limit prevents a tensile failure per Reference 4. It is conservative to compare a compressive stress to a tensile stress limit.
5. The section at the bottom of containment is at Elev. $E_{sect_1} = 93\,ft$. The section at the bottom of the SGR opening is at Elev. $E_{sect_2} = 183\,ft$.

Conclusions from these evaluations are:

- The containment building is not expected to fail catastrophically while the building is detensioned for repairs due to the following load combinations: 1) deadweight and SSE, 2) deadweight and wind, and 3) deadweight and tornado.

- Delamination depths greater than nominal will not result in a catastrophic failure of the containment building for the load cases listed above. The basis for this conclusion is the analysis result at the section at the bottom of the SGR opening. This section is assumed to have no concrete between Buttresses 3 and 4 for the height of the SGR opening. This configuration bounds a case in which the delamination depth is greater than nominal. Delamination depths greater than nominal above and below the SGR opening are considered acceptable based on judgement. The basis is that the SGR opening with a width of 25 feet and extending the full thickness of the containment wall will bound any thinned sections above or below the opening.

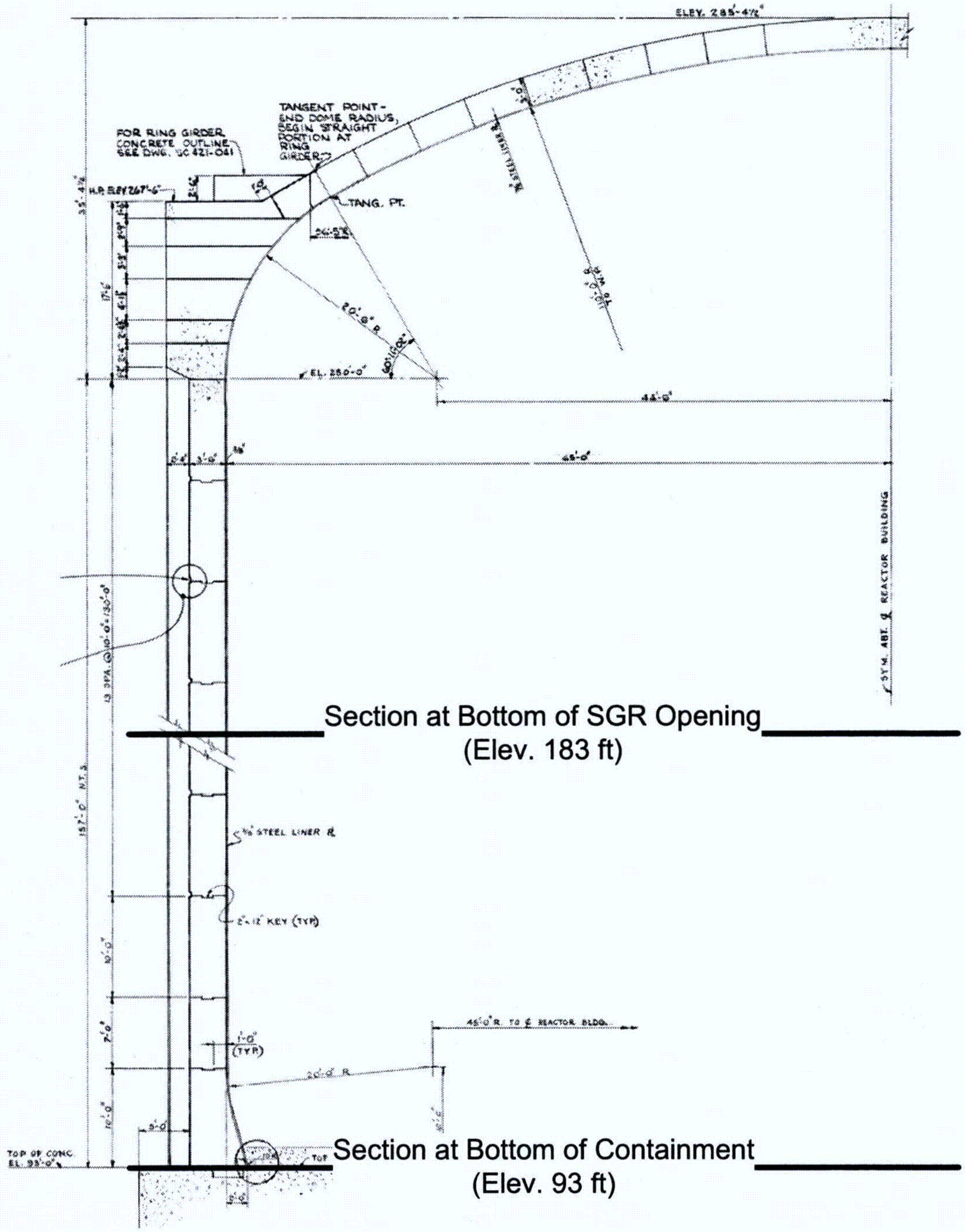
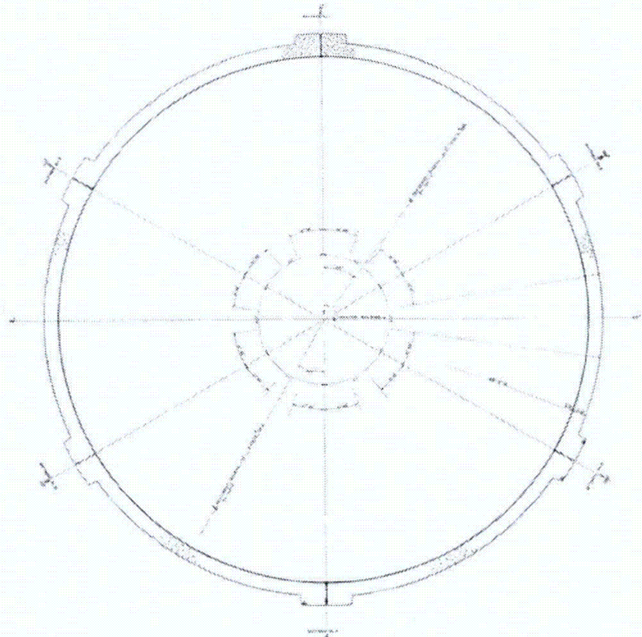
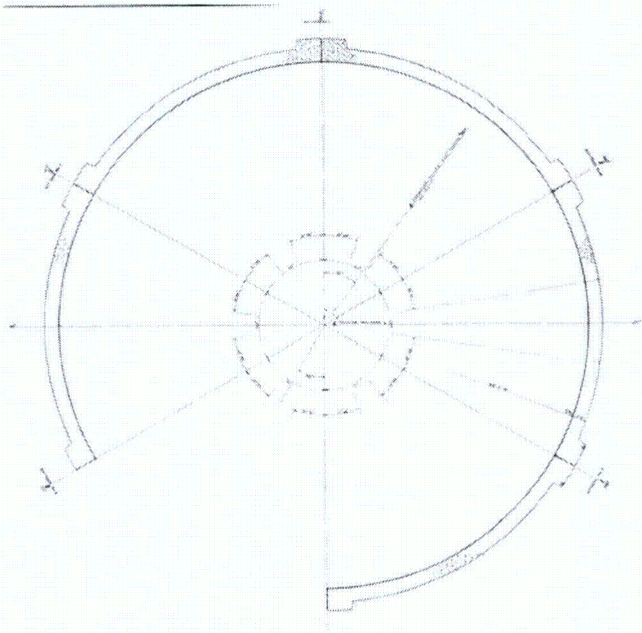


Figure 1. Containment Building



Section at Bottom of Containment (Approximate)



Section at Bottom of SGR Opening (Approximate)

Figure 2. Sections

MPR MPR Associates, Inc. 320 King Street Alexandria VA 22314	Prepared By: J. L. Hibbard Checked By: M. Oghbaei	Calculation No.: 0102-0135-08 Revision No.: 0 Page No.: 8

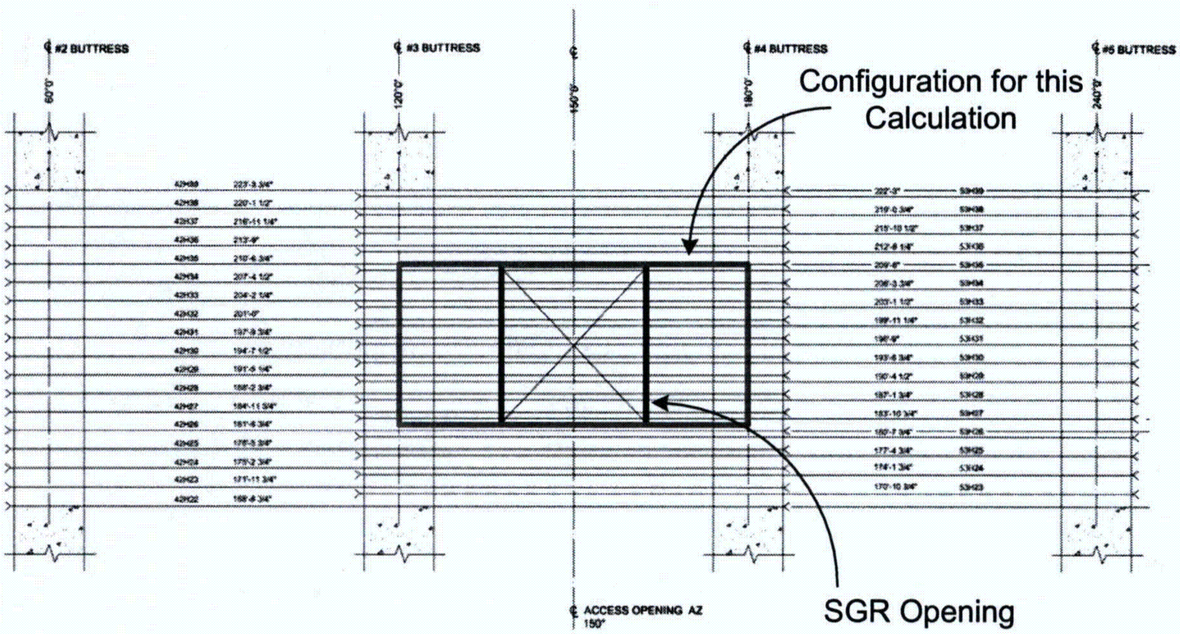


Figure 3. Configuration of Containment for Section at SGR Opening

3.0 BACKGROUND

A project is underway at Progress Energy's Crystal River Unit 3 site to replace the steam generators. As part of that project, an opening has been cut into the concrete containment above the equipment hatch. As this opening was being cut, cracking in the concrete containment wall was identified. The crack is around the full periphery of the opening and is in the plane of the wall. The cracking is located at the radius of the circumferential tensioning tendons, and is indicative of a delaminated condition.

4.0 ASSUMPTIONS

4.1 Unverified Assumptions

None.

4.2 Other Assumptions

1. It is assumed that the thickness of the ring girder is $t_{rg} = 8.83\ ft$. This is a reasonable estimate of the concrete in the ring girder considered as an equivalent rectangular section (see Ref. 2.1). The thickness is used to calculate the mass of the ring girder. A comparison was made of the mass of the ring girder and dome determined in this calculation to the mass calculated by the finite element model used in this project. There was good agreement between the mass calculation in this calculation with that from the finite element model.

5.0 APPROACH

This calculation is an approximate evaluation to assess the potential for a catastrophic failure of the containment when the containment is detensioned for repair. Approximate analysis techniques are used. The analysis considers effects that are considered to be important to the assessment. This is a bounding evaluation rather than a comprehensive evaluation. Effects that are considered to have less than a 20% effect on the final answer are not considered. This is justified based on the large margin to failure in the results.

This calculation considers three load cases: 1) deadweight and SSE, 2) deadweight and wind, and 3) deadweight and tornado. A best estimate is used for the deadweight load. The SSE, wind, and tornado loads are the design basis loads as defined by the FSAR (Reference 3). No load factors are used in the analysis. This is appropriate for a catastrophic failure assessment.

The static coefficient method for seismic analysis specified in Reference 7, Section 6.3 is used. The static coefficient method applies a factor of 1.5 to peak response acceleration to account for potential closely spaced modes. The peak seismic response is from the ground acceleration spectrum from Reference 1. The seismic assessment considers horizontal acceleration and a simultaneous vertical acceleration in the up direction. The vertical up acceleration increases the tensile stress due to the horizontal acceleration, which is a conservative approach.

The analysis calculates the mass of the containment for deadweight and for seismic using the intact configuration of the containment. The effects of removing concrete for the delamination and removing the concrete for the SGR opening are not significant within the framework of this approximate analysis. The mass is based on cylinders and does not include the mass of the buttresses (the buttress mass is less than 1% of the total mass).

The acceptance criterion is that the containment wall membrane plus bending stress be less than the tensile failure stress criterion established in Reference 4 ($\sigma_{ten} = 600\,psi$). The containment wall membrane plus bending stress is a near uniform tensile stress across the containment wall thickness at the extreme tension fiber. Use of a tensile stress criterion is appropriate.

The analysis calculates membrane plus bending stress at two sections through the containment as shown on Figures 1 and 2.

- The first section is at the bottom of the containment at elevation 93 feet. The nominal containment wall thickness is 3.5 feet. At elevation 93 feet, the containment wall is thicker than the nominal thickness. For conservatism and simplicity, the nominal containment wall thickness is used for the evaluation at this section.

- The second section is at the bottom of the SGR opening at Elevation $E_{SGR.b} = 183\,ft$. The SGR opening dimensions are $h_{SGR} = 27\,ft$ high by $w_{SGR} = 25\,ft$ wide (Reference 2.2). The analysis assumes a configuration for the containment in which there is no concrete for an angular extent of $\alpha = 60 \cdot deg$ for the height of the SGR opening. Figure 3 shows the configuration used for the analysis. For reference, the angular extent of the SGR opening is $\alpha_{SGR} = 20.9 \cdot deg$.

Some vertical and hoop tendons will be detensioned for the repair. Detensioning vertical tendons reduces the containment resistance to an overturning moment such as might occur in a seismic, wind, or tornado event. The vertical tendons strengthen the containment in the longitudinal direction and keep the containment concrete in longitudinal compression. Without all the vertical tendons, the capacity of the containment to resist an overturning moment is reduced. This calculation uses the conservative approach that all vertical tendons are detensioned.

The containment building is reinforced with a significant amount of vertical rebar at the 93 foot elevation. This rebar connects the containment shell to the basemat. This calculation takes no credit for this rebar.

The center of gravity of the dome and ring girder are offset from the neutral axis for the analysis at the section at the SGR opening. The moment created by the offset increases the compressive stress due to deadweight at the SGR opening. No credit is taken for this effect in the analysis.

6.0 CALCULATION

6.1 Design Inputs

Containment Cylinder

$t_{cyl} \equiv 42 \cdot in$		Containment wall thickness; Ref. 2.1
$t_b \equiv 2 \cdot ft + 4 \cdot in$	$t_b = 28 \cdot in$	Buttress additional thickness beyond thickness of cylinder; Ref. 2.1
$t_{liner} \equiv 0.375 \cdot in$		Liner thickness; Ref. 2.1
$id_{cyl} \equiv 2 \cdot (65 \cdot ft + t_{liner})$	$id_{cyl} = 130.06\,ft$	Inside diameter of containment concrete wall; Ref. 2.1
$od_{cyl} \equiv id_{cyl} + 2 \cdot t_{cyl}$	$od_{cyl} = 137.06\,ft$	Outside diameter of containment; Ref. 2.1
$E_{cyl.b} := 93 \cdot ft$		Elevation of bottom of containment cylinder; Ref. 2.1
$E_{cyl.t} := 250 \cdot ft$		Elevation of top of containment cylinder; Ref. 2.1
$\alpha \equiv 60 \cdot deg$		Angle between Buttresses 3 and 4; Ref. 2.1 and discussion in Section 5.0

SGR Opening

$E_{SGR.b} \equiv 183 \cdot ft$		Elevation of bottom of SGR opening; Ref. 2.2
$E_{SGR.t} \equiv 210 \cdot ft$		Elevation of top of SGR opening; Ref. 2.2
$w_{SGR} \equiv 25 \cdot ft$		Width of SGR opening; Ref. 2.2
$\alpha_{SGR} \equiv \dfrac{w_{SGR}}{od_{cyl} \div 2}$	$\alpha_{SGR} = 20.9 \cdot deg$	Angular extent of SRG opening
$h_{SGR} \equiv E_{SGR.t} - E_{SGR.b}$	$h_{SGR} = 27\,ft$	Height of SGR opening

Ring Girder

$od_{rg} := od_{cyl} + 2 \cdot t_b$	$od_{rg} = 141.73\,ft$	Outside diameter of ring girder; Ref. 2.1
$t_{rg} \equiv t_{cyl} + t_b + 3 \cdot ft$	$t_{rg} = 106 \cdot in$	Estimate of ring girder thickness for mass calculation; Ref. 2.1 and Assumption 4.2.1
$id_{rg} := od_{rg} - 2 \cdot t_{rg}$	$id_{rg} = 124.06\,ft$	Inside diameter of ring girder
$L_{rg} := 17.5 \cdot ft$		Height of ring girder; Ref. 2.1

Dome

$t_{dome} := 3 \cdot ft$		Dome thickness; Ref. 2.1
$L_{dome} := (35 \cdot ft + 4.5 \cdot in) - L_{rg}$	$L_{dome} = 17.88\,ft$	Height of dome; Ref. 2.1

Concrete

$\rho_c := 144 \cdot \frac{lb}{ft^3}$		Concrete density; Ref. 6
$\sigma_{ten} \equiv 600 \cdot psi$		Concrete tensile strength; Ref. 4

Seismic

$a_h := 1.5 \cdot 2 \cdot 0.135 \cdot g$	$a_h = 0.405 \cdot g$	SSE static equivalent acceleration; the peak in the OBE ground response spectra is from Pages 97 and 98 of Attachment E to Ref. 1 at 2% damping; damping for the reactor building shell is from Ref. 3, Section 5.2.4.1.2, Page 36; SSE is a factor of 2 times OBE based on Ref. 3, Section 5.2.1.2.9; the 1.5 factor accounts for potential closely spaced modes per Ref. 7, Section 6.3
$a_v := \frac{2}{3} \cdot a_h$	$a_v = 0.27 \cdot g$	SSE vertical ground acceleration; Ref. 3, Section 5.2.1.2.9

Wind

$v_{wind} := 179 \cdot mph$

Wind speed for design basis accident; Ref. 3, Section 5.2.1.2.5

Tornado

$v_{tornado} := 300 \cdot mph$

Tornado wind speed for design basis accident; Ref. 3, Section 5.2.1.2.6

$P_{ext} := 3 \cdot psi$

Tornado internal to external pressure drop for design basis accident; Ref. 3, Section 5.2.1.2.6

Air

$\rho_{air} := 0.071 \cdot \frac{lb}{ft^3}$

Density of air; the air temperature to obtain density is 100F for simplicity; Ref. 9, Table A-3

$\mu_{air} := 1.285 \cdot 10^{-5} \cdot \frac{lb}{ft \cdot sec}$

Viscosity of air; the air temperature to obtain viscosity is 100F for simplicity; Ref. 9, Table A-3

Misc.

$C_{d.E6} := 0.38$

Drag coefficient for a cylinder at Reynolds Number greater than 10^6; Ref. 8, Figure 5-78

$C_{d.E5} := 1.2$

Drag coefficient for a cylinder at Reynolds Number of 10^5; Ref. 8, Figure 5-78

6.2 Deadweight Stress

Stress will be calculated at two sections at elevations:

$$E_{sect} := \begin{pmatrix} E_{cyl.b} \\ E_{SGR.b} \end{pmatrix} \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The length of the containment cylinder above each section for the analysis is:

$$L_{cyl} := E_{cyl.t} - E_{sect}$$

$$L_{cyl} = \begin{pmatrix} 157 \\ 67 \end{pmatrix} ft \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The mass of dome, ring girder, and cylinder are:

$$mass_{1,i} := \rho_c \cdot \begin{bmatrix} t_{dome} \cdot \frac{\pi}{4} \cdot id_{cyl}^{\,2} \\ L_{rg} \cdot \frac{\pi}{4} \cdot \left(od_{rg}^{\,2} - id_{rg}^{\,2}\right) \\ L_{cyl_i} \cdot \frac{\pi}{4} \cdot \left(od_{cyl}^{\,2} - id_{cyl}^{\,2}\right) \end{bmatrix} \qquad id := \begin{pmatrix} \text{"dome"} \\ \text{"ring girder"} \\ \text{"cylinder"} \end{pmatrix}$$

The mass of the dome is calculated with a simplified approach in which the dome is a circular plate.

$$E_{sect}^{\,T} = (93 \quad 183)\, ft$$

$$mass = \begin{bmatrix} \begin{pmatrix} 5.74 \times 10^6 \\ 9.29 \times 10^6 \\ 3.32 \times 10^7 \end{pmatrix} & \begin{pmatrix} 5.74 \times 10^6 \\ 9.29 \times 10^6 \\ 1.42 \times 10^7 \end{pmatrix} \end{bmatrix} lb \qquad id = \begin{pmatrix} \text{"dome"} \\ \text{"ring girder"} \\ \text{"cylinder"} \end{pmatrix}$$

The total mass is:

$$mass_{tot_i} := \sum mass_{1,i} \qquad mass_{tot} = \begin{pmatrix} 4.82 \times 10^7 \\ 2.92 \times 10^7 \end{pmatrix} lb \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The cross section area at the two sections is:

$$A_{c_1} := \frac{\pi}{4} \cdot \left(od_{cyl}^{\,2} - id_{cyl}^{\,2} \right)$$

$$r_{mean} := \frac{od_{cyl} + id_{cyl}}{4} \qquad r_{mean} = 66.78\,ft$$

$$A_{c_2} := A_{c_1} - \alpha \cdot r_{mean} \cdot t_{cyl}$$

$$A_c = \begin{pmatrix} 2.11 \times 10^5 \\ 1.76 \times 10^5 \end{pmatrix} \cdot in^2 \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The compressive stress is:

$$\sigma_{dw} := -\frac{mass_{tot} \cdot 1 \cdot g}{A_c} \qquad \sigma_{dw} = \begin{pmatrix} -228.1 \\ -165.7 \end{pmatrix} psi \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

6.3 Seismic and Deadweight Stress

Horizontal

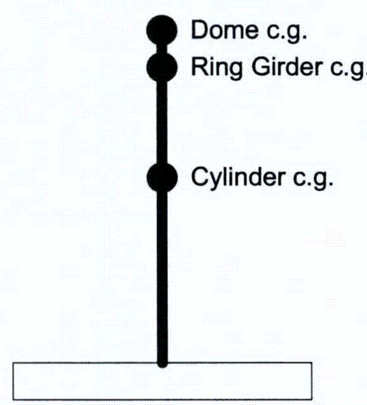
The length from the mass cg to the elevation for the section is:

$$L_{cg_{1,i}} := \begin{pmatrix} L_{cyl_i} + L_{rg} + L_{dome} \div 2 \\ L_{cyl_i} + L_{rg} \div 2 \\ L_{cyl_i} \div 2 \end{pmatrix}$$

$$E_{sect}^{T} = (93 \quad 183)\,ft$$

$$L_{cg} = \left[\begin{pmatrix} 183.44 \\ 165.75 \\ 78.5 \end{pmatrix} \begin{pmatrix} 93.44 \\ 75.75 \\ 33.5 \end{pmatrix}\right] ft \qquad id = \begin{pmatrix} \text{"dome"} \\ \text{"ring girder"} \\ \text{"cylinder"} \end{pmatrix}$$

The moment due to horizontal seismic is:



$$M_{s_i} := a_h \cdot \sum_{j=1}^{3} \left[\left(mass_{1,i}\right)_j \cdot \left(L_{cg_{1,i}}\right)_j \right]$$

$$M_s = \begin{pmatrix} 2.11 \times 10^{9} \\ 6.95 \times 10^{8} \end{pmatrix} \cdot ft \cdot lbf \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The moment of inertia for the intact containment is:

$$I_{cyl} := \frac{\pi}{64} \cdot \left(od_{cyl}^{4} - id_{cyl}^{4} \right) \qquad I_{cyl} = 6.8 \times 10^{10} \cdot in^{4}$$

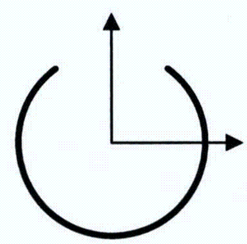
The moment of inertia for the C shaped segment of containment about the containment centroid is calculated below. The neutral axis of the C shaped segment is defined as:

$$\int y\,dA = 0$$

basic statics, no reference required

Define a function to calculate the integral.

$$f(y_{na}) := 2\cdot\int_{-\frac{\pi}{2}}^{\frac{\pi}{2}-\frac{\alpha}{2}} r_{mean}\cdot t_{cyl}\cdot\left(r_{mean}\cdot\sin(\theta) - y_{na}\right)\,d\theta$$



where $y = r_{mean}\cdot\sin(\theta) - y_{na}$

$dA = r_{mean}\cdot t_{cyl}\cdot d\theta$

The neutral axis is:

$$y_{na} := \left| \begin{array}{l} y_{guess} \leftarrow 0 \\ root\left(f(y_{guess}), y_{guess}\right) \end{array}\right.$$

$y_{na} = -12.75\,ft$

Verify the solution:

$f(y_{na}) = 9.93\times 10^{-10}\cdot in^3$ which is approximately zero.

The moment of inertia about the containment centroid is:

$$I_{centroid} = \int y^2\,dA$$

Ref. 5, Formula j 100

$$I_{centroid} := 2\cdot\int_{-\frac{\pi}{2}}^{\frac{\pi}{2}-\frac{\alpha}{2}} r_{mean}\cdot t_{cyl}\cdot\left(r_{mean}\cdot\sin(\theta)\right)^2\,d\theta$$

$I_{centroid} = 4.72\times 10^{10}\cdot in^4$

where $y = r_{mean}\cdot\sin(\theta)$

$dA = r_{mean}\cdot t_{cyl}\cdot d\theta$

The moment of inertia about the neutral axis is:

$$I_C := I_{centroid} + A_{c_2} \cdot y_{na}^2 \qquad I_C = 5.14 \times 10^{10} \cdot in^4$$ Ref. 5, Formula p 19

where $A_{c_2} = 176232 \cdot in^2$

The distance from the neutral axis to the extreme tension fiber is:

$$c_{C.max} := max\left(\left(\begin{array}{c} \left|-\frac{od_{cyl}}{2} - y_{na}\right| \\ \left|\frac{od_{cyl}}{2} \cdot sin\left(\frac{\pi}{2} - \frac{\alpha}{2}\right) - y_{na}\right| \end{array}\right)\right)$$

$$c_{C.max} = 72.1\ ft$$

where $\left|-\frac{od_{cyl}}{2} - y_{na}\right| = 56 \cdot ft$ $\qquad \left|\frac{od_{cyl}}{2} \cdot sin\left(\frac{\pi}{2} - \frac{\alpha}{2}\right) - y_{na}\right| = 72.1\ ft$

The moments of inertia for the two sections are:

$$I_{sect} := \begin{pmatrix} I_{cyl} \\ I_C \end{pmatrix} \qquad I_{sect} = \begin{pmatrix} 6.8 \times 10^{10} \\ 5.14 \times 10^{10} \end{pmatrix} \cdot in^4 \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The distances to the extreme tension fiber are:

$$c_{sect} := \begin{pmatrix} od_{cyl} \div 2 \\ c_{C.max} \end{pmatrix} \qquad c_{sect} = \begin{pmatrix} 68.53 \\ 72.1 \end{pmatrix} ft \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The bending stress is:

$$\sigma_{s.h} := \overrightarrow{\frac{M_s \cdot c_{sect}}{I_{sect}}} \qquad \sigma_{s.h} = \begin{pmatrix} 305.83 \\ 140.42 \end{pmatrix} psi \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

Vertical

The vertical seismic stress is:

$$\sigma_{s.v} := -\left(\sigma_{dw} \cdot \frac{a_v}{1 \cdot g}\right) \qquad \sigma_{s.v} = \begin{pmatrix} 61.58 \\ 44.74 \end{pmatrix} psi \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

Deadweight and Seismic Stress

The deadweight and seismic stress is:

$$\sigma_{dw.s} := \sigma_{s.h} + \sigma_{s.v} + \sigma_{dw} \qquad \sigma_{dw.s} = \begin{pmatrix} 139.3 \\ 19.5 \end{pmatrix} psi \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

Compare the stress to the concrete tensile strength.

$$check_{1_i} := if\left(\sigma_{dw.s_i} \le \sigma_{ten}, ok, nok\right) \qquad check_1 = \begin{pmatrix} \text{"No Failure"} \\ \text{"No Failure"} \end{pmatrix} \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

6.4 Tornado and Deadweight Stress

The drag on the containment building is (Ref. 8, y-axis of Figure 5-78):

$$F_d = C_d \cdot \frac{1}{2} \cdot \rho_{air} \cdot A_p \cdot v_{tornado}^2$$

where	C_d	=	drag coefficient
	ρ_{air}	=	air density
	A_p	=	projected area
	$v_{tornado}$	=	air velocity due to tornado

The projected area of the containment including the projection of the buttresses and ring girder is:

$$A_p := \overrightarrow{\left[od_{rg} \cdot (L_{cyl} + L_{rg} + L_{dome})\right]} \qquad A_p = \begin{pmatrix} 3.93 \times 10^6 \\ 2.09 \times 10^6 \end{pmatrix} \cdot in^2 \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The drag coefficient is a function of the Reynolds Number.

$$Re := \frac{\rho_{air} \cdot v_{tornado} \cdot od_{rg}}{\mu_{air}} \qquad Re = 3.45 \times 10^8$$

The drag coefficient for a cylinder at Reynolds Number greater than 10^6 is $C_{d.E6} = 0.38$. For conservatism, use a drag coefficient of $C_{d.E5} = 1.2$ at a Reynolds Number of about 10^5. The drag load is:

$$F_d := C_{d.E5} \cdot \frac{1}{2} \cdot \rho_{air} \cdot A_p \cdot v_{tornado}^2 \qquad F_d = \begin{pmatrix} 6.99 \times 10^6 \\ 3.72 \times 10^6 \end{pmatrix} lbf \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The bending moment due to the tornado is:

$$M_{tornado} := \overrightarrow{\left[F_d \cdot \frac{1}{2} \cdot (L_{cyl} + L_{rg} + L_{dome})\right]}$$

$$M_{tornado} = \begin{pmatrix} 6.72 \times 10^8 \\ 1.9 \times 10^8 \end{pmatrix} \cdot ft \cdot lbf \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

The bending stress is:

$$\sigma_{tornado} := \overrightarrow{\frac{M_{tornado} \cdot c_{sect}}{I_{sect}}} \qquad \sigma_{tornado} = \begin{pmatrix} 97.63 \\ 38.49 \end{pmatrix} psi \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

Coincident with the tornado wind is a local depressurization. The internal to external pressure drop across the containment wall is $P_{ext} = 3\,psi$. The longitudinal stress in the containment due to the pressure is:

$$\sigma_{ext} := \overrightarrow{\frac{P_{ext} \cdot \frac{\pi}{4} \cdot id_{cyl}^{\,2}}{A_c}} \qquad \sigma_{ext} = \begin{pmatrix} 27.14 \\ 32.57 \end{pmatrix} psi \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

Deadweight and Tornado Stress

The deadweight and tornado stress is:

$$\sigma_{dw.t} := \overrightarrow{\left(\sigma_{tornado} + \sigma_{ext} + \sigma_{dw}\right)} \qquad \sigma_{dw.t} = \begin{pmatrix} -103.3 \\ -94.6 \end{pmatrix} psi \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

Compare the stress to the concrete tensile strength.

$$check2_i := if\left(\sigma_{dw.t_i} \le \sigma_{ten}, ok, nok\right) \qquad check_2 = \begin{pmatrix} \text{"No Failure"} \\ \text{"No Failure"} \end{pmatrix} \qquad E_{sect} = \begin{pmatrix} 93 \\ 183 \end{pmatrix} ft$$

7.0 REFERENCES

1. Progress Energy Specification SP-5209, "CR-3 Seismic Qualification," Revision 0.

2. Progress Energy Drawings:

2.1 No. SC-421-031, "Reactor Building Exterior Wall Concrete Outline," Revision 4.
2.2 No. 421-347, "Reactor Building Temporary Access Opening for SGR Vertical & Horizontal Tendon Positions," Revision 0.

3. Florida Power FSAR, Containment System & Other Special Structures, Revision 31.3.

4. Letter from WJE (Mr. J. Fraczek) to Progress Energy (Mr. D. Dyksterhouse), Subject: CR3 Containment Limiting Tensile Stress, WJE No. 2009.4690, January 11, 2010.

5. K. Gieck, "Engineering Formulas," McGraw-Hill Book Company, 3rd Edition, 1979.

6. Email from Mr. J. Holliday (PE) to Mr. K. Gantz (MPR), 12-30-2009, 10:35 AM, Subject: Concrete Density.

7. Institute of Electrical and Electronics Engineers, Inc. (IEEE) Standard 344-1987, "IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations."

8. Perry & Chilton, "Chemical Engineers' Handbook," McGraw-Hill, 5th Edition.

9. F. Kreith, "Principles of Heat Transfer," International Textbook Company, 1964.

ATTACHMENT

Email from Mr. J. Holliday (PE) to Mr. K. Gantz (MPR), 12-30-2009, 10:35 AM, Subject: Concrete Density.

Hibbard, Jim

From: Holliday, John [John.Holliday@pgnmail.com]
Sent: Wednesday, December 30, 2009 10:35 AM
To: Gantz, Kevin; Knott, Ronald
Cc: Hibbard, Jim; Dyksterhouse, Don
Subject: RE: Concrete Density

Kevin,
The reference will be EC 75218, RB Delamination Repair Phase 2- Detensioning

The unit weight is 144 lbs cu ft.

From: Gantz, Kevin [mailto:kgantz@mpr.com]
Sent: Wednesday, December 30, 2009 10:01 AM
To: Knott, Ronald; Holliday, John
Cc: Hibbard, Jim
Subject: RE: Concrete Density

John and Ron,

I don't think there was ever a follow-up sent to this email. Could you provide us with the reference. I did not see it in S00-0047.

Kevin

-----Original Message-----
From: Knott, Ronald [mailto:Ronald.Knott@pgnmail.com]
Sent: Wednesday, December 16, 2009 10:15 AM
To: Holliday, John
Cc: Gantz, Kevin
Subject: FW: Concrete Density

John,
Can you direct Kevin to the density reference. I don't know where the original data came from for density. I was only quoting what I heard in the meeting. I assumed it was in the S00-0047 attachments.

From: Gantz, Kevin [mailto:kgantz@mpr.com]
Sent: Tuesday, December 15, 2009 6:22 PM
To: Knott, Ronald
Cc: Dyksterhouse, Don; Holliday, John; Bird, Edward; Butler, Patrick
Subject: Concrete Density

Ron,

During our previous meeting you received some original information on the concrete density. I remember you saying later that the concrete density was 144 or 145 pcf. Do you have a reference or an actual number so that I can make sure I have the correct modulus calculated?

Thanks,

Kevin